TRANSPORTATION RESEARCH RECORD 1055

Low-Volume Rural Roads

TRB
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1986

Transportation Research Record 1055
Price $8.60
Editor: Naomi Kassabian
Compositor: Lucinda Reeder
Layout: Lucinda Reeder

mode
1 highway transportation

subject areas
17 energy and environment
21 facilities design
22 hydrology and hydraulics

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Low-volume rural roads.

(Transportation research record, ISSN 0361-1981 ; 1055)
1. Rural roads. 2. Low-volume roads. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1055 380.5 s 86-16273
[TE229] [625.7]
ISBN 0-309-03971-1

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Contents

On the Optimum Level of Effort for Evaluating Low-Volume Rural Road Projects in Developing Countries

CHARLES G. VANDERVOORT

ABSTRACT

Reviews of several rural road projects in developing countries financed by the United States Agency for International Development have revealed that the level of effort in the evaluation of rural road projects has varied widely. The cost of evaluating projects can be significant, but if the higher cost results in better selection of projects, the possible increase in net value of the road program may make a more intensive evaluation worthwhile. A model based on probability theory and parameters believed appropriate for current low-volume road projects, where the benefits flow mostly from increased agricultural activity, are used to show that project benefits would increase if more effort were devoted to evaluation. Cursory evaluations such as "windshield surveys," though perhaps valuable as a screening tool and inexpensive, do not appear to have the accuracy required to maximize the net value of the road program. It appears that for the road projects considered, the so-called rapid rural appraisal techniques, costing about $400/km, are close to the optimum, striking a proper balance between cost of evaluation and benefits achieved. It is only roads with a high cost of construction for which the more elaborate evaluation methods involving in-depth surveys are required. It was found that compared with the benefits foregone by not enough analysis, the cost of the evaluation is small; thus it is better to err on the side of too much analysis than not enough.

The recent series of ex-post impact evaluations (4) by the U.S. Agency for International Development (AID) have shown that the level of effort applied to the economic justification of their road projects has varied widely. The evaluation of the Jamaica Feeder Roads project, an example of a high level of effort, involved massive data gathering in the zone of influence of each road and detailed analyses of these data on computers. At the other extreme, the Liberia Rural Access Roads project involved only a simple reconnaissance lasting a few weeks and followed by a brief analysis by a transportation expert. The difference in cost between these two approaches was, of course, large. The question is whether the return from a higher level of investment in economic evaluation procedures is worth the cost and what the best level of effort might be.

The purpose of this paper is to present a model that assists in establishing the optimum level of effort for the economic justification of a low-volume (say, less than 20 vehicles per day) road project. In this model, the cost of the evaluation effort is balanced against the benefits to be gained from the increased reliability of the evaluation. This reliability is defined as the probability that economically feasible road projects are accepted and infeasible ones are rejected.

The model is then applied to yield useful guidance on the level of effort that should be expended on selection of low-volume feeder road projects. For example, it turns out that, for the average AID feeder road project in which benefits are primarily determined by the additional value of agricultural production and other economic activity induced by the road project, the optimum level of the evaluation effort should be that of a rapid rural appraisal (see section on Estimation of the Parameters for definition) costing about $400 to $600/km of road. Furthermore, the model demonstrates that once the optimum level of effort has been reached, increases in the level of effort will result in only a slight reduction in the net value (benefits minus construction cost minus evaluation cost) of the road program. However, decreasing the level of effort from the optimum will result in a sharp dropoff of the net value of the road program. Thus, it is better to err on the high side of investing in evaluation; that is, it is better to spend too much on evaluation than not enough.

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THE MODEL

The model developed in this section relates the net value of the road construction program with the level of effort devoted to the selection of the roads. Typically, about 30 candidate road projects would be proposed for the rural roads construction program, of which perhaps 20 are economically feasible. The net value of this program is calculated by adding the net present value (NPV) of the benefits of each road that is constructed (and because of imperfect evaluation techniques, some infeasible road projects will probably be included in the construction program) and subtracting the cost of the evaluation of all the projects. The NPV of the benefits for a road project is the value of the incremental agricultural production plus the savings in vehicle operating costs plus the savings in maintenance cost plus other benefits (such as those from increased ease and more convenient passenger travel) minus the construction cost of the project. The NPV can be calculated by using standard economic analysis techniques such as those described by Carnemark et al. (2).

The NPV of the benefit for economically feasible road projects is, of course, higher than that for

infeasible ones. An opportunity is lost, therefore, if, because of unreliable evaluation procedures, an economically infeasible road project is selected or a feasible one is rejected. However, selection involves economic analysis and data collection efforts that are not cheap, and there is a possibility that the gains from more reliable evaluation are nullified by the high costs of the evaluation. It is this trade-off that is analyzed in the model described here.

It should be pointed out briefly that the term "economic evaluation" refers to more than the calculation of costs and benefits and subsequent production of a list showing which road projects are economically feasible. In this paper the term refers to the broader role that economic evaluation plays in increasing benefits and reducing costs. For example, through effective interaction with the rest of the design team, economic evaluation assists in increasing the benefits by identifying constraints (other than access) that diminish the beneficial impact of a road project and by proposing effective measures to eliminate these constraints. At the same time, economic evaluation can reduce costs and increase benefits by determining the proper mix between labor and capital during construction, exploring alternative alignments, or establishing the proper mix between road design standards and maintenance effort.

GENERAL DERIVATION

A brief description of the model is presented. It is assumed that the project has a fixed construction budget and that the roads to be constructed are selected from a long list of candidates. To keep the model simple, it will be assumed that all roads are of equal length and have equal construction cost. Dropping this assumption presents no analytical problem but would unnecessarily complicate the model.

The road projects are evaluated one by one until the road program budget is exhausted; this will happen when the budget equals the cost of construction plus the evaluation of the roads. The list usually contains both "good" and "bad" projects. (To save space, the term "good" will be used for an economically feasible project and "bad" for an economically infeasible one.) Some candidate road lists will be of high quality, containing a large number of good projects. High quality may result, for example, if projects are screened before being included on the list. Or, because local governments are often much aware of the transport needs of their communities, lists composed at the local level are sometimes of high quality.

Depending on the reliability of the evaluation procedures, a good project will have a certain probability of being correctly identified as good. Similarly, a bad project will also have a certain probability of being mistakenly identified as good. The reliability of the selection procedure will of course depend on the level of effort devoted to it and on the inherent difficulty of evaluating the projects. Road projects that are, for example, located in a remote area of a country that has been little studied and where few data are available will be more difficult to evaluate than those located in areas that have been well studied.

Using probability theory, equations are derived that determine the number of good and bad projects that are constructed and, as a function of the quality of the candidate road list, the reliability of the evaluation procedures and the number of road projects that are evaluated. Then, with the budget constraint and the cost of constructing and evaluating a project, an equation is derived that gives the total number of projects that are evaluated.

$$NC = (W - W1) + R1 \tag{1}$$

$$W = N * (1 - PG) \tag{2}$$

$$W1 = B1 * W = B1 * N * (1 - PG) \tag{3}$$

$$W - W1 = N * [(1 - B1) * (1 - PG) + G1 * PG] \tag{4}$$

$$R1 = G1 * R = G1 * N * PG \tag{5}$$

where

- NC = total number of projects constructed,
- W = number of bad projects evaluated,
- W1 = number of bad projects evaluated as bad,
- R = number of good projects evaluated,
- R1 = number of good projects evaluated as good,
- N = total number of projects evaluated,
- PG = probability that a candidate project is good,
- B1 = probability that a bad project will be evaluated as bad, and
- G1 = probability that a good project will be evaluated as good.

Thus, NC can be expressed as follows:

$$NC = N * [(1 - B1) * (1 - PG) + G1 * PG] \tag{6}$$

By introducing the budget constraint and the cost of constructing and evaluating the projects, the equation for the total number of projects evaluated (N) can be derived. The cost of constructing the good projects plus the cost of constructing the bad ones plus the cost of evaluating the projects (B) is

$$B = N * [G1 * PG * CC + (1 - B1) * (1 - PG) * CC + EC] \tag{7}$$

where

- B = budget available for the road program ($),
- CC = construction cost of a project ($), and
- EC = cost of evaluating a project ($).

Solving for N,

$$N = B/\{[(1 - B1) * (1 - PG) + G1 * PG] * CC + EC\} \tag{8}$$

The number of projects eliminated by the economic evaluation (NE) is

$$NE = N * [PG * (1 - G1) + B1 * (1 - PG)] \tag{9}$$

The number of economically feasible projects constructed (NG) is

$$NG = N * PG * G1 \tag{10}$$

And the number of economically infeasible projects constructed (NB) is

$$NB = N * (1 - PG) * (1 - B1) \tag{11}$$

It can be seen that the total number of projects evaluated as given in Equation 8 is the sum of the good projects constructed (Equation 10) and the bad projects constructed (Equation 11). The total value of the road program is the sum of the benefits added by each project minus the construction cost of the good and bad projects and the evaluation costs of all the projects.

The value of the benefits added by a good project (VG) and that of the benefits of a bad one (VB) can

be expressed as the product of the project's benefit/cost ratio and the construction cost. This follows simply from the definition of the benefit/cost ratio. For example, if the construction cost for a project is $700,000 and the benefit/cost ratio is 1.2, the benefits of the project can be calculated as 1.2 x $700,000 = $840,000. For an economically infeasible project with a benefit/cost ratio of 0.8, the benefits would be 0.8 x $700,000 = $560,000. Assuming that the good projects have an average benefit/cost ratio of GS and the bad ones have a ratio of BS, the net value of the road program (VAL) is

$$VAL = (GS * CC * NG) + (BS * CC * NB) - (NC * CC) - (N * EC) \quad (12)$$

The first and second terms of the foregoing equation represent the sum of the benefits of the good and bad projects. The third term gives the total construction cost of the projects, and the fourth term gives the cost of evaluating the projects, including those that were not constructed because they did not pass the evaluation.

Calculation of the net value of the road program requires estimation of a complex set of parameters, which will be discussed in the next section.

ESTIMATION OF THE PARAMETERS

To exercise the model and to enable the drawing of some broad conclusions regarding the optimum level of effort for evaluating rural road projects certain parameters must be estimated: the probability that a project is good (PG), the probabilities that a good project is identified as good (G1) and that a bad project is identified as Bad (B1), and the average economic return (benefit) of an economically feasible and an economically infeasible project.

It must be mentioned, however, that ex-post evaluations of feeder road projects are rare, and information on the results of the evaluations is even more scarce. The author knows of only a few evaluations that generated data useful to the estimation of these parameters. These sparse data do not instill much faith in the precision of these estimates. For this reason, care was taken to draw only those conclusions from the model that are not sensitive to the precision of the parameter estimates. The model, of course, was useful in examining this sensitivity.

Probability That a Project Is Good

The probability that a project is good (PG) depends on the amount of background work that went into the preparation of the list of road candidates. Usually this list is prepared by the host government, and its preparation may or may not involve some screening of the projects. A few historic projects will be examined to get an indication of what the range of PG might be.

In a recent road project in Haiti, for example, the candidate road list simply contained projects that were deemed desirable; no attempt had been made to apply quantitative criteria to screen the projects from the point of view of economic feasibility. For this project, ex-post evaluation revealed that about 450 of the 600 km of candidate road projects were economically feasible, and the ratio of 450/600 = 0.75 may be taken as an indication of the probability that a candidate road project is economically feasible. It is not an exact indicator because the ex-post evaluation techniques used to establish the feasibility were not perfect; nevertheless, this procedure provides a useful indicator of PG for the Haiti project.

Another example is a rural road project in the Dominican Republic. The candidate road projects had been screened with cursory data on traffic levels and agricultural potential. Ex-post evaluation indicated that an average of 81 percent of the candidate road projects were found to be economically feasible.

For an Asian Development rural road project in the Philippines, the candidate road project list was compiled from recommendations submitted by local government officials. Though the screening was not quantitative, it was based on judgment by persons familiar with the transport needs in their regions. Ex-post evaluation procedures established that 84 percent of the road projects were indeed economically feasible.

These three examples and other cases not cited here indicate that, depending on the degree of screening, the range for the parameter PG would be 0.75 to 0.85. It would be possible to have lower values of PG for cases in which the roads are in an area with especially low potential (such as areas in the Sahel in Africa) and where screening is applied infrequently or not at all. But it would be surprising if PG fell below 0.6. For the upper limit of PG a value of 0.9 could be considered reasonable. Thus, the range for PG is estimated to fall between 0.6 and 0.9.

Evaluation Efficiency

The values of B1 and G1 are a function of the level of effort devoted to the evaluation. These levels of effort can range from no evaluation, which is comparable to selecting projects by flipping a coin--heads, the project is good, tails, the project is bad--to comprehensive in-depth surveys to collect the necessary data on agricultural and other activities in the zone of influence of the project.

Statistical theory suggests that the reliability of the evaluation process will increase as the level of effort devoted to the evaluation increases but that this relationship is governed by the law of diminishing returns. Thus, at low levels of effort it will be easy to achieve large gains in reliability, but at the higher levels an increase in effort may produce only a small increase in reliability. The relationship is also a monotonically increasing one in that an increase in the level of effort will never result in a decrease in reliability. Finally, the reliability should asymptotically approach the value of perfection (100 percent accuracy) as the level of effort increases beyond bound.

There are a number of mathematical functions that have been found useful in science and industry to depict such a reliability function. The one selected for this paper is a simple one (the data do not warrant more sophistication) and is defined as follows:

$$R = A - 0.5 * [1 - \exp(-M * K)] \quad (13)$$

where

- R = reliability or probability of correctly classifying a project, expressed as either B1 or G1;
- K = level of effort devoted to the evaluation ($/km of road);
- A = initial reliability at a zero level of effort; and
- M = parameter specifying the efficiency of the evaluation procedures.

In theory, the level of effort may assume any value between zero and infinity. In this paper, however, the discussion will be limited to the four levels of effort that have traditionally been applied to road projects. Ex-post evaluation of a number of past road projects is used to establish the relationship between reliability and the level of effort and to estimate the parameters A and M.

The lowest level of evaluation effort, as mentioned earlier, is simply none. In this case, projects may be selected at random, by flipping a coin, for example. This selection process is repeated until the road construction budget is exhausted. At this practically zero level of effort the reliability of correctly classifying a project as good or bad will be 50 percent, and the intercept A of the reliability curve is therefore 0.50.

Given that the value of A is 0.5, it can be shown that M, the efficiency parameter, can be expressed in terms of the level of effort (K) and the reliability (Y) as

$$M = \log \{[(1 - Y)/0.5]/-K\} \quad (14)$$

In this equation, it is stated that if the level of effort K that went into the evaluation and the reliability Y that was achieved are known, the efficiency can be calculated. In estimating the reliability curve for G1, for example, if the level of effort expended on the evaluation was \$1,000/km of road and the ex-post evaluation showed that a reliability of 0.86 was achieved, the value of M for the G1 curve would be 0.00127. If for that same level of effort of \$1,000/km it was possible to achieve a reliability of 0.96, the value of M would increase to 0.0023. For the reliability of the B1 curve, the parameter M can in principle be estimated the same way. If for the level of effort of \$1,000/km a reliability of 0.86 could be achieved for B1, the value of M would also be 0.00127. Carrying out the value of M to five decimal places is not an attempt to achieve spurious precision; rather, it is necessary because of the great sensitivity of R to M.

M has to be carefully distinguished from the other parameter, K, also found in the curve for evaluation reliability and that relates to the level of effort devoted to the evaluation. K is measured in dollars per kilometer, or the cost to evaluate 1 km of road, and is proportional to the size of the evaluation team and their time spent. As will be described later, this cost is about \$40/km for the windshield survey. M is a variable that gives the increase in reliability that can be expected from an increase in level of effort K. The higher the value of M, the more rapidly reliability will increase with level of effort.

For example, if M = 0.001 and the level of effort is increased from \$400/km to \$425/km, the reliability will increase from 0.665 to 0.673, an increase of 1.2 percent. But for M = 0.002 and for the same increase in K, the reliability will increase from 0.775 to 0.786, an increase of 1.4 percent.

M can be considered proportional to the skill of the evaluation team and the amount of readily available information on the road and its zone of influence. The value of M will be high for an evaluation in which the evaluation team is well trained and experienced in rural road evaluations and has available a number of studies and surveys pertaining to the road. Conversely, for an evaluation in which the team is unskilled and there are few reliable data on the road and the surrounding region, the value of M will be low. Typically, as indicated in the section on the estimation of the parameters, the average efficiency of evaluation teams used for estimating the value of M is around 0.002, though there may have been instances in which the efficiency dropped to 0.001.

The lowest value for M is zero. This value implies that the evaluation team is totally incompetent and that its classification of projects is no better than that achieved by flipping a coin. In theory, there is no upper limit to the value of M. In practice, however, it does not appear that M could exceed the value of 0.04; this value implies that, even for the lowest meaningful level of effort, the windshield survey, the team could correctly classify about 90 percent of the roads. Though this efficiency is high, it could conceivably be reached by well-trained evaluation teams that have the benefit of earlier studies of the road and its zone of influence.

It was not possible to determine a reasonable range for the reliability of classifying a bad project as bad. Projects classified as bad were, of course, not constructed, and for the projects investigated in this study, the data on the bad ones had been discarded. It would be reasonable to assume, however, that the efficiency for the process of classifying good projects as good would be similar to that for classifying bad projects as bad. In this paper, therefore, it will be assumed that M is the same for both processes.

To review briefly, at this point the value of A is known, and the intercept of the reliability curve for B1 and G1 is 0.5. It has been assumed that the value for the efficiency (M) is the same for both reliability curves. It remains to estimate the value of M. To do this, some historic projects will be reviewed, and the level of effort (K) that went into the evaluation will be calculated and the reliability (R) that was achieved will be estimated.

The nature and the cost of the various levels of effort, such as the windshield survey, rapid rural assessment, and the in-depth survey, that have been applied to past rural road project evaluations will be discussed first. This will be followed by the estimation of the reliability. By combining the reliability and level of effort it will be possible to estimate the value for the efficiency (M).

Windshield Survey

In the windshield survey, the information for evaluating rural roads is collected by a quick visit to the candidate projects by a team of engineers and economists to obtain an impression of the actual or potential level of economic activity along the road and of the costs of road construction. No attempt is made to quantify the extent of cultivated areas for various crops, the density of population, or the location of sources of borrow for construction. Such a survey is cheap and rapid and, if done by a competent team, a distinct improvement over no evaluation. This approach was applied to the Liberia Rural Access Roads II project.

The time required to survey a project consisting of, say, 500 km of feeder roads would take the two-person team about 2 weeks. Allowing 1 week for office work and 1 week for contingencies, the approximate cost of the windshield survey would be about \$20,000. This assumes that the work is done by expatriates and includes the per diem cost of a jeep plus driver and the cost of airfare from the United States to a less developed country in Africa. The average cost of the windshield survey would be about \$40/km of surveyed road.

Few ex-post evaluations have been carried out for projects that used a windshield survey, but their reliability is estimated to be somewhat less than 60 percent. This estimate is supported by an ex-post evaluation of a rural road project in the Dominican

Republic in which it was found that the ex-ante windshield survey had correctly evaluated 6 out of 11 roads. This would make the reliability of the windshield survey about 0.55 and, assuming that the cost of the windshield survey was $40/km of road, the value for the efficiency parameter M would be 0.0026.

Rapid Rural Assessment

In the rapid rural assessment, a small multidisciplinary team of exports attempts to quantify the costs and benefits of the road project by extensive use of direct field observations, aerial surveys, interviews of key persons, including, of course, farmers of small properties, and the use of key indicators as proxies for economic variables, such as the quality of housing as a proxy for income (3). In the rapid rural assessment, in-depth surveys of production, income, and so forth, based on scientifically designed sampling plans and requiring detailed questionnaire surveys, are avoided.

In the calculation of the cost of this type of survey, it is assumed that the team consists of seven persons (team leader, two agronomists, two engineers, one sociologist, and one economist) and that two jeeps are used. About 60 days would be spent in the field and 40 days in the office. For a 500-km feeder-road project, the total cost of the evaluation would be about $200,000, again including per diem costs and domestic and international transportation, which makes the cost per kilometer about $400.

Though there have been a number of rapid rural appraisals in the recent past, the author knows of no case in which their accuracy has been evaluated. It is estimated, however, that their reliability would be between 70 and 80 percent and, assuming a cost of $400/km, the value of M would fall between 0.0013 and 0.0023.

In-Depth Survey

The in-depth survey represents the most intensive level of effort. A large multidisciplinary team spends a long time in the field collecting detailed data on crop types, cultivated areas, soil characteristics, yields, and the other information required to calculate the value added by the road project. Household interviews, for example, would be made to gather information on rural travel patterns. This type of survey would enable the most accurate determination of the economic feasibility of the road projects.

The team would consist of the same personnel as those for the rapid rural appraisal, with the addition of a statistician and 10 interviewers, and two more jeeps with drivers would be needed. They would spend about 5 months in the field and 3 months in the office, and the cost for the 500-km road survey would be about $500,000. On a per-kilometer basis the cost would be about $1,000.

In a comparative evaluation of selected highway projects performed by the World Bank and documented in an internal memo in 1974, it was found that of 15 road projects that had been identified as economically feasible in the ex-ante evaluation, three in retrospect turned out to be infeasible. These projects accounted for 14 percent of the investment. Furthermore, five of the projects (17 percent of the investment) had, in retrospect, a marginal rate of return. Hence, between 14 and 31 percent of the investment was in subnormal or marginal projects, and it may therefore be concluded that the gross reliability of the selection process was between 69 and 86 percent. This modest reliability is not, however, all due to weaknesses in the economic evaluation procedures. A major factor accounting for the infeasibility of some of the projects in this program was the large cost overruns caused by poor implementation of construction. Assuming that half of the unreliability was due to these cost overruns, the actual reliability of the evaluation procedures would be between 85 and 95 percent.

The level of effort devoted to the World Bank project evaluations was not documented, but it was estimated to fall between that of a rapid rural appraisal and an in-depth survey, at a cost of about $700/km. By applying the equation, it can be calculated that the value of M falls between 0.0017 and 0.0028. Another example is the series of ex-post evaluations of eight loans for rural roads carried out by the Inter-American Development Bank in 1980. Between 11 and 12 of the 14 road projects were correctly classified, giving a reliability between 0.79 and 0.86. As for the previous case, the level of effort devoted to the ex-ante evaluations is estimated at about $700/km and M therefore ranges between 0.00124 and 0.00182.

Value Added

The benefit/cost ratio of an economically feasible road project will, of course, be higher than that for an infeasible one. However, its average benefit/cost ratio will depend on a large number of factors, of which the two most important are the economic potential of the area within which the project is located and the condition of the road before improvement. Thus, for road projects that consist of upgrading an animal and pedestrian track in an agriculturally rich area that is only now being developed, the average benefit/cost ratio of the economically feasible projects may be quite high and may exceed 3.0. On the other hand, if the project consists of rehabilitating neglected roads in an area that has been under development for some time, and many of today's projects in the developing countries fall within this category, the average benefit/cost ratio of the economically feasible projects may fall between 2 and 3. Finally, a project consisting of improving a low-potential road located in, for example, the Sahel area of Africa may yield an average benefit/cost ratio of only about 1.5.

From a review of a number of road projects in South America and Asia that were completed during this decade, it was found that, on the average, an economically feasible project had a benefit/cost ratio of about 2.25, and the economically infeasible ones had a benefit/cost ratio of about 0.5. The projects consisted of rehabilitating roads that had seriously deteriorated because of lack of maintenance, and the lack of access resulting from the poor road conditions had supressed the development of agriculture in the regions served by the roads. Such road projects are common now in the developing countries, and in the application of the model in the next section, these roads will be taken to represent the nominal case.

APPLICATION OF THE MODEL

In this section the model will be used to develop an understanding of what the appropriate level of effort should be for feeder-road evaluations. Figure 1 shows the total value of the road construction program as a function of the level of effort devoted to the evaluation and as predicted by the model. The parameters used by the model in developing Figure 1 assume the nominal values derived in the section on estima-

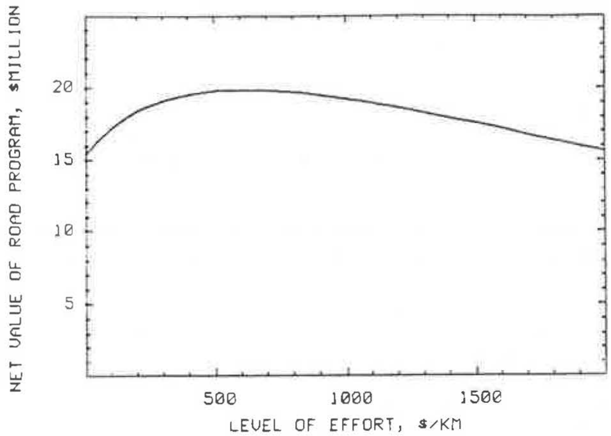

FIGURE 1 Nominal case.

tion of the parameters. These values, it will be recalled, are believed appropriate for current projects involving the rehabilitation of feeder roads that, through neglect of maintenance, are in bad condition and therefore carry little or no mechanized vehicle traffic. These values are as follows:

1. The probability that a road project on the candidate list is economically feasible (PG) is 0.7.
2. The efficiency (M) of the evaluation procedure is 0.002.
3. The average benefit/cost ratio of an economically feasible project (GS) is 2.25.
4. The average benefit/cost ratio of an economically infeasible project (BS) is 0.5.
5. The average construction cost of a project (the project is assumed to be 20 km long) (CC) is $10,000/km, and the total budget for road construction is $21 million.

The effect of deviations from these nominal values will be explored later.

As shown in Figure 1, the net value of the road program even with a zero level of evaluation effort is about $15 million. (As discussed earlier, the net value of the road program is equal to the total benefits generated by the program minus the construction cost and the evaluation cost.) As the level of effort increases, the net value of the program rises rapidly because the infeasible projects are being weeded out until a maximum of about $20 million is reached at a level of effort around $600/km. This is the optimum level of effort, and the reliability of the evaluation effort at that point is about 0.85. As the evaluation effort increases beyond that point, still more infeasible projects are being eliminated, but the additional cost of the evaluation starts to offset the gain in benefits from the larger proportion of feasible projects. Thus, the net value of the program gradually diminishes. And, at a level of effort of about $2,000/km, at which the reliability of the evaluation process should be about 0.99, the net value of the road program is again slightly above $15 million.

A numerical example is useful to illustrate the shape of the curve in Figure 1. Because the proportion of good projects on the candidate road list for the base case is 0.7, the zero level of effort (such as simply picking every other project on the list or choosing the projects by tossing a coin) will result in a set of constructed projects in which 70 percent are economically feasible and 30 percent are infeasible. Because the cost of evaluation was zero, the whole road budget ($21 million) can be used for construction at $200,000 per project. Thus, 105 projects can be constructed, of which 73.5 (70 percent) are economically feasible and 31.5 (30 percent) are infeasible. The benefits generated by the economically feasible projects will be 2.25 times their construction cost, $450,000 per project, or $33 million in total (2.25 x $200,000 x 73.5). The infeasible projects will contribute benefits of only 0.5 times their construction cost, $110,000 per project, or $3.15 million in total (0.5 x $200,000 x 31.5). The total net return of the road program will therefore be $33 million + $3.15 million - $21 million = $15.2 million as shown in Figure 1.

With high evaluation levels of effort of $2,000/km of road, the reliability of the selection effort will be practically perfect, and only economically feasible projects will be constructed. However, the cost of evaluation will be high. For every seven good projects that are evaluated and constructed, three infeasible ones are evaluated and eliminated. In effect, for every economically feasible project that is constructed, 1 3/7 of a project must be evaluated at a cost of $2,000/km. The effective construction cost of the economically feasible projects is therefore

$200,000 + 20 x 1 3/7 x $2,000 = $257,143.

Also, the number of economically feasible roads constructed is

$21 million/$257,143 = 81.66.

The net value of the road program therefore is

2.25 x $200,000 x 81.66 - $21 million = $15.7 million.

As shown in Figure 1, the value of the road program rises rapidly as the level of effort increases, until the optimum of about $19.8 million is reached. After that, the value diminishes gradually because of the excessive cost of evaluation. It is important to note that the curve is steeper on the left-hand side (the side of reduced level of effort) than it is on the right-hand side (the side of increased level of effort). This would indicate that, under the usual uncertainty faced when a project is planned, it is better to err on the side of too much effort on the evaluation. For example, as shown in Figure 1, the optimum level of effort is about $600/km of road. A decrease in this level of effort of $400 would reduce the net value of the program to $18,331 million, a reduction of 7.5 percent. However, increasing the level of effort by $400/km would reduce the value of the road program to $19.17 million, a reduction of only 3.3 percent.

Figure 2 shows that with more efficient evaluation techniques (M = 0.005) so that a higher reliability of classifying a candidate road is achieved at a given cost, the optimum level of evaluation effort can be reduced to $400/km. (For the nominal case, as discussed previously, the optimum level of effort was $600/km.) In addition, more efficient evaluation techniques also increase the maximum possible value of the road project to about $22.6 million; this is about 14 percent above the $19.8 million value for the nominal case. It can also be seen that, as for the nominal case, the curve is steeper to the left of the optimum level of evaluation effort than it is to the right. Again, this means that it pays to err on the side of too much effort on the evaluation rather than too little. For example, decreasing the level of effort by $400/km from the optimum would reduce the value of the road program to about $15 million, a reduction of about 33 percent. Increasing

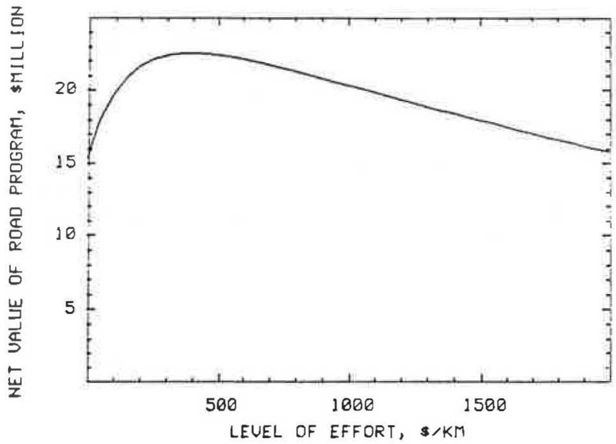

FIGURE 2 More efficient evaluation.

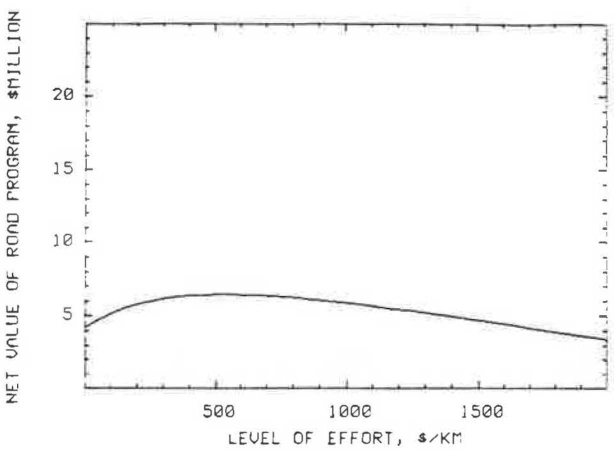

FIGURE 3 Low-potential projects.

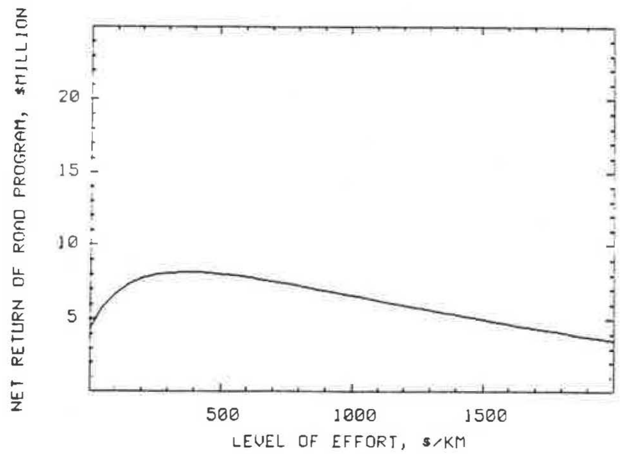

FIGURE 4 Efficient evaluation of low-potential projects.

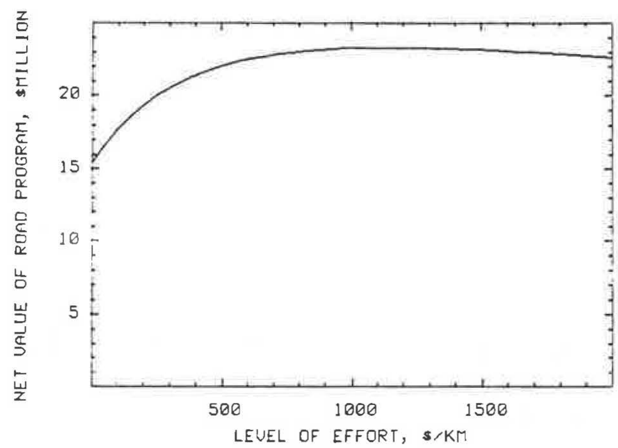

FIGURE 5 High-cost projects.

the level of effort by $400/km would reduce the value of the road program to about $21 million, a reduction of only 6 percent.

Figure 3 is indicative of low-potential projects, such as those in sparsely populated areas with low agricultural potential; the average benefit/cost ratio of the economically feasible projects in this case was assumed to be 1.5. The figure shows that the optimum level of evaluation effort is about $500/km. As expected for such projects, however, the maximum value of the road program is only $6.5 million, far below the $20 million or so that can be realized from the more productive projects.

The reason for the lower level of effort for low-potential projects (assuming the same construction cost and evaluation efficiency) is the proportionally lower return of these projects compared with the cost of evaluation and construction.

Again, it can be seen that it is better to err on the high side of evaluation effort than on the low side. To illustrate, a reduction of evaluation effort by $400/km will reduce the value of the road program to $5.2 million, a reduction of 20 percent. But an increase in effort of $400/km will reduce the value of the road program to $6 million, a reduction of only about 6 percent.

Figure 4 shows that, when low-potential projects are involved, improving the efficiency of the evaluation (M = 0.005) will substantially increase the net value of the road program (from $6.5 to $8.1 million, a 25 percent increase). The new optimum level of effort is $400/km, or $100 less than for the case with less efficient evaluation.

It will be recalled that increasing the evaluation efficiency by the same amount for the nominal projects increased the net value of the road program by only 14 percent. Thus, it can tentatively be concluded (clearly, more research is warranted in this area) that improving the skills of evaluation teams becomes even more important when low-potential projects are involved.

Figure 5 shows the case for the high-cost projects; these are projects in which the improvement cost is $35,000/km of road. This would be an unusually high cost for a feeder-road improvement project and is presented only to illustrate the impact of higher construction costs on the optimum level of effort. The optimum level of effort for such high-cost projects would be about $1,100/km, a substantial increase from the $600/km for the nominal case. But the net value of the road program is even less sensitive to the optimum level of effort than it was for the previous cases. To illustrate, a decrease of $400/km in the level of effort would cause the net return to drop from $23.3 million for the optimum level of effort to $22.8 million, a reduction of only 2 percent. Increasing the level of effort by $400/km would reduce the net value to $23.1 million, a reduction of less than 1 percent. In fact, the curve is so flat around the optimum that a level of

effort corresponding to the nominal case, $600/km, would reduce the net value of the road program by only about 3.5 percent.

In summary, it has been seen that, around the optimum, the net value of a road program is remarkably insensitive to the level of effort. In general, for road improvement projects that are of fairly low cost, such as around $10,000/km, the optimum level of effort will fall between $400/km and $600/km; this is on or slightly above the rapid-rural-appraisal level of effort. For more expensive road projects, such as those costing around $35,000/km, the optimum level of effort is about $1,100/km and comparable to an in-depth survey of level of effort. However, for the high-cost projects, the value of the road program is so insensitive to the level of effort that the use of rapid rural appraisal techniques would result in only a minor reduction in the net value of the road program.

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The views and judgments expressed in this paper are those of the author and do not necessarily reflect those of the U.S. Department of Transportation.

Publication of this paper sponsored by Committee on Low Volume Roads.

Recent Developments in Rural Road Design in Australia

C. J. HOBAN

ABSTRACT

Two-lane roads make up the bulk of the rural road system in Australia and carry most of the travel between major cities. A number of developments in the geometric design of these roads are discussed, with particular reference to the contributions made by the Australian Road Research Board. Some of the major changes have been a greater emphasis on alignment consistency, the growing use of auxiliary lanes, and the move toward partial sealing of shoulders. Some details of new design guidelines are presented. Partial shoulder sealing was introduced primarily to reduce maintenance costs but has since been found to have safety and operational benefits. A survey of shoulder use has provided information on the probability of meeting stopped vehicles on the roadside and given some recommendations on shoulder and rest area design. Traffic simulation has been used to evaluate alternative road improvement strategies, including alignment changes and the use of auxiliary lanes. The TRARR simulation model is now being used by several state road authorities for planning and investigation studies. A consideration of accidents and road geometry is an underlying theme of the research on all of these topics.

Approaches to the geometric design of rural roads in Australia have undergone a number of changes in recent years. The emphasis has shifted from the rigid application of design standards to a greater awareness of the specific objectives for a given project and the alternative methods for achieving these. Many design standards have been critically reviewed, and particular attention has been paid to the cost-effectiveness of alternative road improvement options.

A number of these changes are discussed, with particular reference to the contributions made by the Australian Road Research Board (ARRB) and the continuing research in this area. For simplicity, only the geometric aspects of road design for isolated road sections away from intersections and towns are considered.

Australian Road Research Board, 500 Burwood Highway, Vermont South, Victoria 3133, Australia.

TABLE 1 Comparative Statistics for Australia and North America, 1979

Country	Area (x 10^6 km^2)	Population (x 10^6)	Roads (x 10^6 km)	Freeways (x 10^3 km)	Automobiles (no./km)	Mileage/1,000 Population (km)
Australia	7.7	14.5	0.87	1.0	6.8	60.0
United States	9.4	226.4	6.25	82.0	18.7	27.6
Canada	9.9	23.6	0.89	4.9	10.8	37.7

BACKGROUND

Australia has a land area almost as large as that of the United States (excluding Alaska and Hawaii) but with a population of only 15 million. A large proportion of the population lives within a few large cities in the Southeast. Outside the cities, the majority of travel occurs on two-lane roads, which thus represent the principal means of long-distance regional and intercity travel and include most of the National Highway network. Some comparative statistics are presented in Table 1.

During the 1960s and 1970s, road design practice in Australia was largely based on well-established high geometric standards. For major two-lane rural highways, these called for wide cross sections and high design speeds for vertical and horizontal alignment. As traffic volumes grew, these roads were converted to four-lane freeways with full access control.

In the mid-1970s, however, it was becoming apparent that only a small proportion of required road improvements could be undertaken with available road funds. This was leading to a network with some sections of high-standard road and a growing backlog of substandard road lengths.

With this background, road practitioners and researchers took a renewed interest in the objectives of rural road designs and the most cost-effective means of achieving those objectives. This interest culminated in 1980 with the Workshop on the Economics of Road Design Standards (1) and the publication of the Interim Guide to the Geometric Design of Rural Roads (2) by the National Association of Australian State Road Authorities (NAASRA). In both these publications a need for changes to the prevailing philosophies of rural road design was recognized. The process of review and change is continuing, and a revised version of the NAASRA Interim Guide will be published in 1986.

ALIGNMENT CONSISTENCY

The alignment of a road--that is, its vertical and horizontal profile--can affect traffic speeds and safety. In a major study of speeds on curves, McLean (3) demonstrated that horizontal curves had a strong effect on traffic speeds, whereas vertical curves had very little. Further, he found that speed reductions on horizontal curves depended on the overall alignment of the road as well as the specific curve radius, as shown in Figure 1. McLean used the term "speed environment," defined as the 85th-percentile speed on level tangent sections, to reflect the driver's perception of the overall speed standard of a road.

The speed environment perceived by the driver is often different from the design speed conceived by the engineer. Consider, for example, a road designed to 85 km/hr but with a long straight between curved sections. The straight may give the driver an impression of a higher design speed, and he may be caught unaware by the sharpness of the next curve. McLean found that drivers were consistently exceeding the safe speeds for curves in such situations.

Accident research studies have also highlighted the role of unexpected sharp curves. The U.K. Transport and Road Research Laboratory (4) reported that accident rates on sharp curves on high standard alignments were over five times as high as those for similar curves on more winding roads, as shown in Table 2.

These concepts were incorporated into the Interim Guide to the Geometric Design of Rural Roads (2). The new design procedure called for greater attention

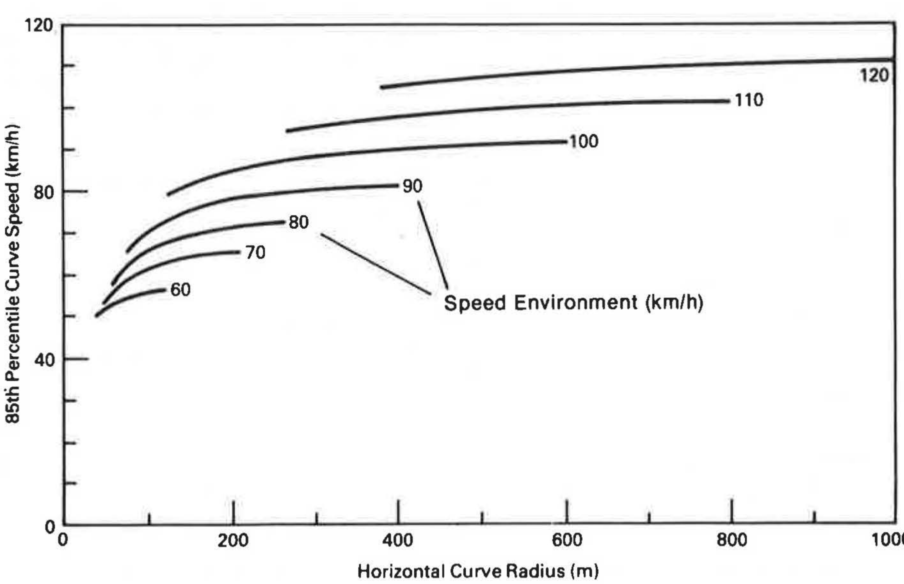


FIGURE 1 Curve speed-prediction equations *(3)*.

TABLE 2 Nonintersection Injury Accident Rates on Straights and Curves *(4)*

Avg Curvature (degrees/mi)	Straight[a]		Bend: Radius 5,000-2,000 ft		Bend: Radius 2,000-1,000 ft		Bend: Radius < 1,000 ft		Total	
	AMVM	NOA	AMVM	NOA	AMVM	NOA	AMVM	NOA	AMVM	NOA
0-40	1.2	284	1.2	33	1.0	4	8.6	18	1.3	339
40-80	0.9	142	0.9	37	0.9	23	1.5	14	0.9	216
80-120	0.7	69	0.5	11	0.9	16	1.6	24	0.8	120
> 120	0.4	15	0.5	3	1.0	19	1.2	19	0.7	56
Total	1.0	510	0.9	84	1.0	62	1.8	75	1.0	731

Note: AMVM = accidents per million vehicle miles; NOA = number of accidents.

[a]Bends with radius more than 5,000 ft included.

to consistency of alignment from the driver's point of view, and recommended that

1. On long straights, the design speed be taken as the speed environment;
2. Design speeds on successive road elements not normally differ by more than 10 km/hr and definitely not more than 15 km/hr;
3. Where larger changes are unavoidable, more gradual change be accomplished with a sequence of horizontal curves; and
4. Special attention be paid to curves at the end of long straight road sections.

In the current review of the guide it is recognized that it will sometimes be impossible to achieve consistent road geometry. In these situations, the use of traffic management techniques such as warning signs or speed zoning is recommended to alert drivers to an unexpected change in geometric standard.

AUXILIARY LANES

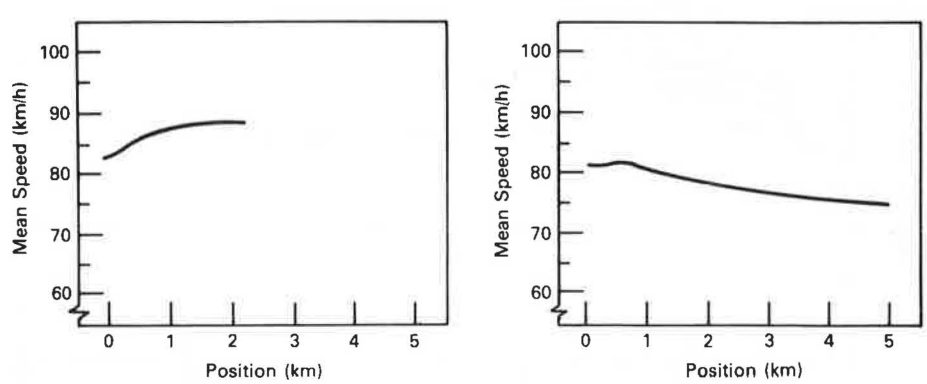
A major change in rural road design practice in Australia in recent years has been the growing use of auxiliary lanes. These were initially provided on long steep grades to overcome delays and bottlenecks caused by slow trucks. They are now being used at a wide range of locations to break up bunched vehicles and improve the quality of service on a road. The term "auxiliary lane" is used here to include overtaking lanes, climbing lanes, descending lanes, and short passing bays.

When an additional lane is provided over a short length of road to increase overtaking opportunities, benefits should be experienced for some distance upstream and downstream of the added lane. If advance notice is given to drivers, say, 1, 2, and 3 km before the overtaking lane, their overtaking behavior over this upstream section is likely to be more relaxed and safe, because marginal overtaking maneuvers are less likely to be attempted. Because the overtaking lane breaks up bunched traffic, downstream traffic should experience higher average speeds and an improved quality of service. The benefits of the short extra lane can thus extend for a number of kilometers downstream.

To investigate traffic behavior on an overtaking lane, the author (5) measured changes in traffic performance near transitions between two- and four-lane rural roads. The transitions were in fairly level terrain, and some results are presented in Figure 2. On entry to the four-lane road section, it is shown that speed continued to improve over a distance of 2.2 km, but half of the observed benefits occurred within the first 400 m. The deterioration in traffic speed on return to the two-lane road (at a second site many kilometers downstream) was considerably more gradual.

A number of studies in Australia have used traffic simulation to estimate the benefits in traffic operations that could be expected from the provision of auxiliary lanes. An example is discussed in the section on simulation of road improvement strategies. In each case, the studies indicated that substantial benefits in traffic operations could be achieved at quite low costs. A two-lane road with regular auxiliary lanes in effect provided an intermediate level of service between two- and four-lane roads. The term "two-and-a-half-lane road" was coined to describe this type of road.

A major review of auxiliary-lane research and design practice in Australia and Canada has recently been completed (6), which highlighted many areas of similarity between the two countries, although there are some differences in lengths, signing, and barrier



(a) Two Lanes to Four Lanes **(b) Four Lanes to Two Lanes**

FIGURE 2 Changes in traffic speed on transitions between two-lane and four-lane rural roads *(5)*.

lining practice. Also, Australian research on this topic was compared with results from Canada (7) and the United States (8,9).

GUIDELINES FOR AUXILIARY-LANE DESIGN

In the current review of the Australian design guide (2), the section on auxiliary lanes is being substantially expanded and modified to incorporate the results of recent research and experience. Some key provisions of the draft guidelines are described in the following paragraphs.

Justification

The justification for an auxiliary lane is based on an evaluation of a significant length of road rather than on an isolated location. The basic evaluation assesses the need for overtaking opportunities and depends on traffic volume, the percentage of slow vehicles in the traffic stream, and the availability of overtaking opportunities in the surrounding terrain.

TABLE 3 Recommended Minimum Volume Guidelines for Overtaking Lanes

Overtaking Opportunities over Preceding 5 km[a]		Current-Year Design-Hour Volume[c] by Percentage of Slow Vehicles[d]		
Description	Percent of Road Providing Overtaking[b]	5	10	20
Excellent	70-100	850	750	650
Good	30-70	680	600	520
Moderate	10-30	500	450	400
Occasional	5-10	340	300	260
Restricted	0-5	230	200	170
Very restricted	0	140	120	100

[a]Depending on road length being evaluated, this distance could range from 3 to 10 km.
[b]These percentages are based on a much more demanding criterion than the 1,500 ft used in the Highway Capacity Manual *(10)*. See NAASRA Interim Guide *(2)*.
[c]Vehicles per hour, both directions.
[d]Including light trucks and cars towing trailers, caravans, and boats.

Draft volume guidelines are presented in Table 3. These are appropriate for short low-cost auxiliary lanes at spacings of 10 to 15 km or more. Climbing lanes, descending lanes, and passing bays are then regarded as special cases that may justify auxiliary lanes at lower traffic volumes. Where grades are steep and long enough to reduce a design truck speed to 40 km/hr, for example, the volume guidelines may be reduced by specified reduction factors.

Length

Table 4 presents a range of auxiliary lane lengths appropriate for both grades and level terrain. As a general rule, it is considered more cost-effective to construct two short auxiliary lanes several kilometers apart than one long one in excess of the normal maximum length. If long bunches occur at a given location, the provision of several auxiliary lanes at regular spacings should break them up before they become very extensive.

On grades, auxiliary lane length is often constrained by the choice of appropriate starting and termination points. These constraints can lead to long or expensive climbing-lane proposals. Because the volume guidelines in Table 3 are based on benefit-cost analysis, they may not be applicable to more expensive proposals. When climbing-lane proposals exceed 1,200 m in length or have construction costs well above those for surrounding terrain, consideration may be given to the use of partial climbing lanes or, in extreme cases, short passing bays. Alternatively, if the grade delays are not excessive, it may be appropriate to construct a lower-cost auxiliary lane away from the grade.

TABLE 4 Auxiliary Lane Lengths

Design Speed[a] (km/hr)	Total Taper (m)	Auxiliary Lane Length, Including Taper (m) Minimum	Recommended	Normal Maximum
50	125	200	350	450
60	150	250	400	550
70	175	300	500	650
80	200	400	600	850
90	225	500	700	1,000
100	250	600	800	1,200

[a]For the section on which the auxiliary lane is constructed.

Location

The choice between grade and nongrade locations should take account of relative costs, delays on the grade, and the nature of traffic demand on the road. If significant bunching occurs for several kilometers along a route, an auxiliary lane at any location is likely to produce substantial benefits. In other cases, the major problem could be truck crawl speeds at specific locations, and the solution may need to be provided where the problem occurs.

Spacing

On a road with no auxiliary lanes, it is most cost-effective initially to place auxiliary lanes well apart rather than to have several close together. A spacing of 10 to 15 km or more is appropriate for this approach. As traffic volumes grow, or where a larger improvement in traffic operations is required, additional auxiliary lanes may be provided at spacings as close as 3 to 5 km.

Approach to Design

Auxiliary lanes may be provided at regular spacings to upgrade traffic operations over a long section of road. Locations should be chosen where possible to minimize construction costs, and a mixture of climbing and overtaking lanes is often appropriate. Some important considerations are the following:

1. Starting and termination points should be clearly visible to drivers, and tapers should be adequate to allow smooth lane changing.
2. Merges should, where possible, be located so as to minimize speed differences between fast and slow vehicles.
3. Advance notice is desirable for several kilometers upstream of an auxiliary lane.
4. For traffic in the opposing direction to that of an auxiliary lane, barrier lines to restrict overtaking should generally follow normal practice for two-lane roads. (This would often allow the opposing traffic to make some use of the auxiliary lane for overtaking.) The use of more restrictive barrier lining practice may be appropriate in some circumstances (such as high volumes), but should not be too widespread.

When auxiliary lanes are not constructed on grades, they should be located so as to appear appropriate to drivers. Sites with curved alignment and restricted sight distance are generally preferable to straight sections with long sight distance. Curves with lower safe speeds should be avoided, however, because these are not appropriate locations for overtaking.

SHOULDER DESIGN

At the request of the NAASRA Road Design Committee, ARRB has recently investigated many aspects of shoulder design for two-lane rural roads. The project considered the effects of shoulder width and sealing, and incorporated five main tasks:

1. A literature review of the effects on both traffic operations and safety;
2. A more detailed investigation of the effects on road accidents;
3. A field study of vehicle lateral placement on the road and its variation with lane width, shoulder width, and shoulder type;
4. A survey of state road authority divisions to determine current practice and attitudes regarding shoulder construction, maintenance, and use by vehicles; and
5. A field survey of the extent and nature of use of road shoulders by stationary vehicles.

Literature Review

Armour and McLean (11) found that much of the literature on road shoulders was over 20 years old and that the effect of shoulder width on accidents was still unclear. Sealed shoulders were found to have better safety records than gravel shoulders for a wide range of traffic volumes and shoulder widths. The accident reductions achieved through shoulder improvements were mainly in run-off-road and opposing-direction accidents. The role of the shoulder in providing a stand-clear area for stopped vehicles did not appear to produce significant safety benefits.

On the basis of overseas research, Armour and McLean (11) argued that narrow sealed shoulders may be appropriate for many Australian highways. The benefits would include reduced maintenance costs, a reduction in loss-of-control accidents associated with gravel shoulders, and the possibility that slow-moving vehicles could pull to the left to be overtaken. Armour and McLean suggested that, because nondiscretionary stops are fairly rare on rural roads, continuous wide shoulders may not be necessary on all roads.

Shoulders and Accidents

A more detailed study of shoulders and accidents was reported by Armour (12), who used the fatal-accident report forms completed by engineers of the Department of Main Roads (DMR) in New South Wales and the computerized road inventory developed for the NAASRA Roads Study (13).

Armour (12) considered fatal accidents on undivided rural highways with a 100 km/hr speed limit (the normal state limit) from 1980 to 1982. The frequency of accidents on roads with various shoulder types was compared with the frequency of travel on these road sections, using the road inventory information. Armour found that the DMR forms covered only 55 percent of fatal accidents reported by the police for state highways over that period, but provided an unbiased sample of all accidents. She therefore expressed her results as "relative accident rates," assuming a rate of 1.0 for straight roads with unsealed shoulders.

TABLE 5 Relative Accident Rates by Shoulder Type and Road Geometry *(12)*

Geometry	Shoulder Type: Unsealed	Sealed	All
Horizontal			
Straight	1.0	0.3	0.7
Curved	5.9	1.5	3.5
Vertical			
Flat	1.1	0.4	0.8
Grade	5.6	1.2	3.0
All	1.8	0.6	-

The results of this study are presented in Table 5. Overall, the relative accident rates were 1.8 on roads with unsealed shoulders and 0.6 on roads with sealed shoulders. This indicates that accidents occurred three times as often with unsealed shoulders than with sealed shoulders. When grades and curves are isolated, the difference is even more pronounced. That is, the accident rate on curves and grades with unsealed shoulders was roughly four times that on similar road sections with sealed shoulders.

Investigating these relationships further, Armour (12) noted that roads with sealed shoulders may have been reconstructed more recently, and thus incorporated better road geometry, pavement, or surroundings than the roads with unsealed shoulders. An assessment of the frequency of various types of curves and grades, however, revealed similar results for both road types.

A closer examination of accident report forms, however, showed that loss of control on the gravel shoulder was a contributing factor to 17 percent of all accidents and 50 percent of run-off-road accidents. This examination also showed that parked vehicles and vehicles overtaking on the left were not a significant accident factor.

Vehicle Lateral Placement

In the third stage of this project, Armour (14) reported on a field study of the effects of road cross section on vehicle lateral placement. Vehicles were observed on the road at 19 rural highway sites, including 11 in the state of Victoria and 6 in Queensland. The sites included a range of shoulder types, shoulder widths, lane widths, and delineation treatments. The major findings of this study were as follows:

1. The main factors affecting lateral placement at the sites were shoulder type and lane width;
2. A lateral shift of 0.15 m away from the center line was produced by either increasing lane width from 3.2 to 3.7 m or sealing or partly sealing the shoulders;
3. There was a significant variation in lateral placement between vehicles alone on the road and those meeting other vehicles; that is, vehicles moved farther from the center line when opposing vehicles were in the vicinity;
4. The effect of unsealed shoulder width was unclear;
5. No effect of edge lines on lateral placement was detected; however, all studies were conducted in

daylight and night behavior with regard to edge lines may be different; and

6. No difference was found in the lateral placement data from the two states.

Vehicle lateral placement is sometimes regarded as a measure of driver comfort and safety. If this is the case, the provision of wider lanes (up to 3.7 m) or sealed shoulders will increase the spacing between opposing vehicles and improve comfort and safety. The sealed shoulders investigated in this study had widths of 1.2 to 1.5 m.

Current Shoulder Design Practice and Attitudes

With the support of the NAASRA Road Design Committee, a questionnaire survey was circulated to engineers of each state road authority in Australia. A total of 94 questionnaires was circulated to all district, divisional, and regional engineers and head office sections responsible for road design. Sixty-nine of these were returned by July 1984, although some did not contain replies to all questions.

The results of this survey (15) provide a useful summary of current practice and attitudes to road shoulders. Some interesting results were the following:

1. On existing rural roads, both wide and narrow shoulders are used, and the majority are unsealed. On new construction, however, about half of the replies indicated that sealed shoulders are provided on over 50 percent of new work.

2. Of those divisions providing sealed shoulders, half provided them at volumes below 500 vehicles/day and 94 percent provided them at volumes below 3,000 vehicles/day. Sixty-eight percent of divisions always provide full-strength pavement under sealed shoulders.

3. Two questions asked for general opinions on construction and maintenance costs. The majority of respondents believed that sealed shoulders only slightly increase construction costs and substantially reduce maintenance costs. Those respondents actually using sealed shoulders particularly supported these views.

4. There was general agreement that although sealed shoulders would be used by moving vehicles, this was not desirable and should not be encouraged. It was stated that sealed shoulders should be designed for traditional shoulder functions and should not be used as an alternative to providing extra traffic lanes.

5. The most common delineation treatment with both shoulder types was edge lining. About 28 percent of respondents believed that edge lines reduced maintenance costs of unsealed shoulders; of divisions using edge lines with unsealed shoulders, 49 percent believed that they reduced maintenance costs. A number of respondents commented that edge lines had safety benefits.

SHOULDER USE

The final component of this project consisted of a field survey of vehicles stopped on the road shoulder. This survey has recently been undertaken in six regions of Queensland, with considerable assistance from the Main Roads Department.

The survey involved drivers traveling over predetermined routes with known traffic volumes and noting all vehicles stopped by the roadside in rural areas. If the driver was present, a short questionnaire was administered to determine whether the stop was discretionary or unavoidable, how far the vehicle might have traveled before stopping, and the driver's knowledge of nearby rest areas. Information on the placement of the vehicle relative to the traffic lane and shoulder was also recorded.

Details of the survey procedure and results are given by Charlesworth (16). Some of the key findings were as follows:

1. A total of 283 stopped vehicles was recorded in almost 6,000 km surveyed. This gave an average of one stationary vehicle met per 21 km traveled. The frequency of stationary vehicles increased with traffic volume.

2. The distribution of reasons for stopping is shown in Table 6. Sixty-five percent of stopped vehicles were parked completely off the road carriageway, that is, clear of both the traffic lane and shoulder. The majority of these were service vehicles (e.g., telephone or gas) or those parked outside houses, farms, and schools. The frequency of stopped vehicles met on the carriageway was one every 62 km. Most of these vehicles were on the shoulder. Only three were fully on the traffic lane and five straddled lane and shoulder.

3. Table 7 shows a clear relationship between the proportion of vehicles off the carriageway and the duration of stop. Vehicles stopped for a long time were most likely to be off the carriageway.

4. Roads with fewer rest areas had more discretionary stops but a similar number of breakdown stops relative to roads with more frequent rest areas.

5. Discretionary stops not related to specific locations accounted for 33 percent of vehicles stopped on the carriageway. The results of this study indicate that better publicity about rest areas may reduce this number.

6. A disabled vehicle was met on the carriageway, on average, every 180 km driven. Forty-seven percent of disabled vehicles could have been driven further.

Charlesworth (16) reported two measures of the interaction between moving traffic and stopped vehicles. These were vehicles met per kilometer of travel

TABLE 6 Distribution of Reasons for Stopping (16)

Reason for Stopping	Location of Vehicle: Off Carriageway	On Carriageway	Total
Service	56	4	60
Outside property	60	12	73
Breakdown	20	23	43
Leisure or refreshments	17	18	35
Other discretionary	3	14	17
Other location (specific)	14	13	27
Unknown	15	13	28
Total	185	97	283

TABLE 7 Percentage of Vehicles off Carriageway and Estimated Duration for Different Types of Stops (16)

Type of Stop	Percent of Vehicles off Carriageway	Estimated Duration of Stop	Sample Size
Service	93	5.5 hr	20
Outside house	88	4.0 hr	9
Leisure	51	25 min	35
Breakdown	53	16 min	28
Location (specific)	52	14 min	5
Outside business	18	11 min	5
Other discretionary	18	7 min	14

by one driver and vehicles met per kilometer per hour at a given traffic volume. She recommended that road designers give particular attention to shoulder and roadside design at locations where drivers are likely to stop. Improving the frequency and attractiveness of, and information about, rest areas was also recommended as a way of reducing discretionary stops on the roadside.

SIMULATION OF ALTERNATIVE ROAD IMPROVEMENT STRATEGIES

A number of studies have been undertaken at ARRB to investigate alternative strategies for rural road improvement. McLean (17) used a single-vehicle simulation model to examine the cost implications of various design speed standards in hilly terrain. Briefly, his findings were as follows:

1. With the cost parameters typically employed for economic assessment of road projects in Australia, travel time costs for cars were about three times those for fuel consumption, whereas these two cost components were about equal for trucks.
2. In hilly terrain, a design speed increase of 20 km/hr in the range 50 to 90 km/hr represents
   - A doubling of earthworks,
   - A 10 percent decrease in car travel time,
   - A 7 percent decrease in truck travel time,
   - A 0 to 10 percent decrease in car fuel consumption, and
   - A 2 to 5 percent decrease in truck fuel consumption.
3. In terms of trade-offs between standards for horizontal and vertical alignment, trucks gain relatively more from improved vertical geometry, whereas cars gain relatively more from improved horizontal geometry.

More recently, a detailed microscopic simulation model has been used to evaluate road improvement alternatives. The TRARR rural traffic simulation model (18) was developed at ARRB, and copies have been provided to over a dozen organizations both inside and outside Australia. Several Australian state road authorities are using the model for planning future road programs and evaluating alternative design standards. Some specific applications are described by Robinson (19), Hoban (20), and Cox (21).

Some typical results from one case study (21) are presented in Table 8. This study compared several options for upgrading a 9-km section of road in rolling to hilly terrain with a substandard alignment relative to adjacent road sections. Although the existing alignment had an original design speed of 80 km/hr, its average highway speed (10) over all road elements was 99 km/hr. A proposed realignment to 110 km/hr was costed at $2.25 million in 1981 Australian prices. To investigate a wide range of options, the simulation study considered overtaking lanes, duplication to four lanes, and partial duplication on both the existing and proposed alignments. Simulated traffic speeds on these options are illustrated in Figure 3.

The results in Table 8 show that overtaking lanes were found to be highly cost-effective and could be economically justified at traffic volumes of about 1,000 vehicles/day on this road. The options of realignment and duplication were considerably more expensive and could only be justified at much higher volumes. The results of this and similar studies have provided a major input to the development of Australian design guidelines for auxiliary lanes.

The simulation studies found that the provision of overtaking opportunities is generally more important to improved traffic operations than a high standard of road geometry. In discussion, however, the importance of alignment consistency was stressed (see section on auxiliary-lane design). It was suggested that localized alignment improvements could have great benefits for both traffic operations and safety.

ROAD SAFETY CONSIDERATIONS

The safety implications of geometric design alternatives have also been investigated. A detailed dis-

TABLE 8 Benefit/Cost Results for Nine Road Proposals

Proposal	Cost ($000s)	B/C Ratio (AADT = 4,000)	AADT for B/C = 1.0
Existing alignment			
E1: existing two-lane road	-	-	-
E2: two overtaking lanes	168	5.2	950
E3: four overtaking lanes	336	4.4	1,160
E4: partial duplication	549	2.5	2,000
E5: full duplication	1,710	1.4	3,030
New alignment			
R1: two-lane road	2,250	0.3	-[a]
R2: two overtaking lanes	2,418	0.6	6,430
R3: four overtaking lanes	2,586	0.8	4,920
R4: partial duplication	2,836	0.7	5,180
R5: full duplication	4,050	0.7	5,040

Note: AADT = annual average daily traffic (vehicles per day); B/C = benefit/cost.
[a]AADT > 8,000 vehicles/day, outside the range of this study.

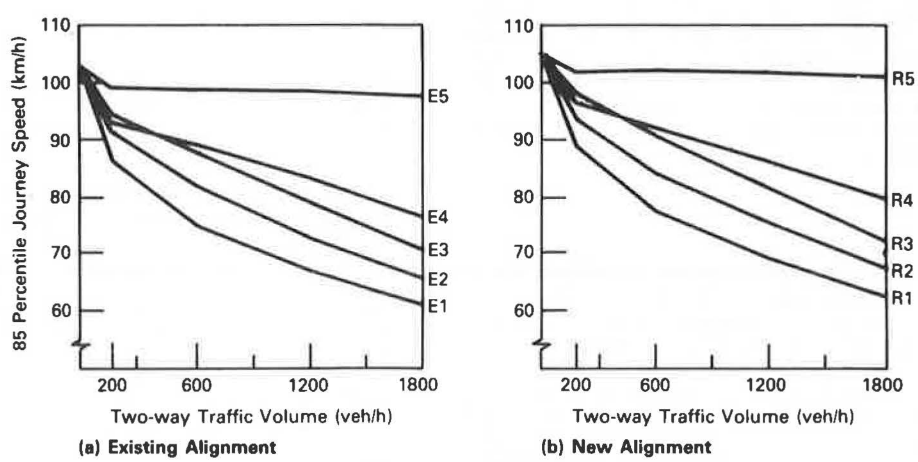


FIGURE 3 Simulated 85th-percentile speeds for 10 road options.

cussion of this topic is beyond the scope of this paper, but useful Australian reviews have been made (22-24). These have drawn heavily from U.S. research studies (25-28) and reinforce many of the recommendations already made in this paper. They indicate, for example, that road safety may be improved by actions such as

1. Realignment of road sections with sharp curves and steep grades, especially where these are below the standard of adjacent road sections;
2. Increasing lane width, at least up to 3.4 m, or providing additional lanes;
3. Sealing road shoulders and increasing shoulder width;
4. Adding auxiliary lanes;
5. Controlling access to a road;
6. Providing gentle batter slopes and removing fixed objects or installing guard fence to protect motorists from hazards; and
7. Providing special facilities for runway trucks on steep downgrades.

Road safety improvements are sometimes considered as a relatively low-cost alternative to major realignment of a problem section of road.

SUMMARY

In recent years, road design practice in Australia has placed greater emphasis on selecting cost-effective solutions to particular road problems. This has required a move away from the rigid application of design standards and a critical review of road improvement needs. A particular feature of evolving road design practice is a reduced emphasis on a high alignment design speed. Improved alignment is generally quite expensive, and if funds are limited, it may not be the most cost-effective approach in many cases.

As an alternative, primary emphasis is placed on consistency of the road environment as seen by the driver. Greater consistency may be achieved by localized alignment improvements, and large changes in speed environment should where possible be introduced in stages. As a last resort, traffic management measures such as signs and road markings should be used to alert drivers to unexpected road features.

A second element of evolving design practice is a much greater emphasis on provision for overtaking. Auxiliary lanes are now being provided at both grade and level locations on Australian highways, giving substantial improvements in traffic performance. Cost is an important factor in decisions about auxiliary-lane locations, and other guidelines are included in this paper.

The use of partial (1.0 to 1.5 m) sealed shoulders on Australian rural highways is also growing. These provide for substantial accident reductions, reduced maintenance costs, and improved driver level of service. The need for gravel shoulder adjacent to the narrow seal is uncertain, and a recent study of stopped vehicles on road shoulders gives some information on the requirements of these vehicles and the expected frequencies of meeting them on a given road.

The alignment and cross section of a road can be designed to minimize the possibilities and severities of road accidents. However, an ideal design may be extremely expensive and often not practical in particular circumstances. The provision of guard fence and runaway-vehicle facilities can be a cost-effective alternative approach.

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Publication of this paper sponsored by Committee on Geometric Design.

Economic Analysis of Broad-Based Dips Versus Aluminum Pipe Culverts on Low-Volume Roads

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ABSTRACT

There is currently some controversy regarding the relative benefits and costs of culverts and broad-based dips as drainage devices on low-volume roads where no intermittent or permanent streams exist. A research project was undertaken in the Monongahela National Forest to attempt to determine, by using an economic analysis, under what conditions broad-based dips were more appropriate than 18-in. aluminum pipe culverts. Construction cost data for each of the drainage devices were acquired from Forest Service records and a survey of contractors. Eighty percent of the contractors preferred culverts to dips. The type of drainage structure specified in project plans affected contractors' bids. Typical culvert costs were around $500 per structure, whereas dips averaged around $350 per device. Maintenance costs had to be estimated on the basis of discussions with private foresters, because no actual maintenance cost data were available. Annual maintenance cost per culvert was estimated at $8.33 whereas that per dip was $10. To gain insight relative to road user attitudes toward broad-based dips, a truck driver questionnaire was utilized. Almost 90 percent of the respondents reported feeling physical discomfort when passing through a dip; one-half of the drivers suggested eliminating the use of dips entirely. It was found that the additional travel time through a dip can be neglected; excess vehicle operating costs were estimated at $0.077 per vehicle per dip. Broad-based dips were less expensive than pipe culverts for roads carrying traffic volumes in the range of 5 to 10 vehicles per day. At higher volumes, the increased road user costs associated with dips made culverts the more economical alternative.

In order to meet the continuing demand for timber and mineral resources, there has been an increase in the number of logging and mining roads being constructed. As a result of budget constraints, the financial resources available to build these low-standard roads (which may serve only 0 to 50 vehicles per day) are severely limited. Although it is in the best interests of the operators to construct roads that are cost-effective, these roads must also protect the natural environment. The problem is not limited to logging and mining roads, however; similar goals apply to the low-volume roads being built to stimulate economic and social benefits in developing nations.

One of the primary concerns in locating and designing low-volume roads is drainage. There must always be adequate drainage if a road is to remain usable. Roadway drainage begins with the removal of surface runoff from the roadway itself. In addition, drainage design must consider (a) the removal of excess water from under the roadway; (b) provision of roadside ditches of proper size, shape, and slope; (c) the prevention of side-slope and ditch erosion; and (d) the passage of water flowing in all natural and man-made drainage channels. These considerations imply the need for a variety of drainage structures or devices.

Several types of drainage devices are used for controlling water flow on low-volume roads; probably the most common type is the culvert. Culverts, as shown in Figure 1, are closed conduits that carry surface water across or from the road right-of-way. A second device is the broad-based dip, a depressed outsloped section of roadway that acts as a water catchment and drainage channel. Dips can be used instead of culverts for cross drainage where no intermittent or permanent streams are present. Figure 2 shows the plan and profile of a typical broad-based dip.

Currently, there is some controversy among foresters and engineers regarding the relative benefits and costs of each of these devices. One school of thought suggests that metal culverts are superior for most drainage needs. The initial cost of culverts is high compared with simple drainage devices but they have relatively long lifetimes, require relatively little maintenance, and are essentially unnoticed by road users.

Others promote broad-based dips because of their several advantages. Dips have a relatively low initial cost and, unlike culverts, they can be used without the expense of a ditch line. When high flows exceed the design capacity of a culvert, there is the potential for increased ditch scour, extensive erosion of the road surface, and mass failure of roadway fills. In such cases, dips located just downgrade of the culvert can serve as a safety overflow device. Properly constructed dips have low maintenance costs and, like culverts, do not increase wear on vehicles or reduce hauling speeds. However, a disadvantage of broad-based dips is that equipment operators need special training in order to be able to construct them properly. Thus, dips are often not built according to the intended specifications.

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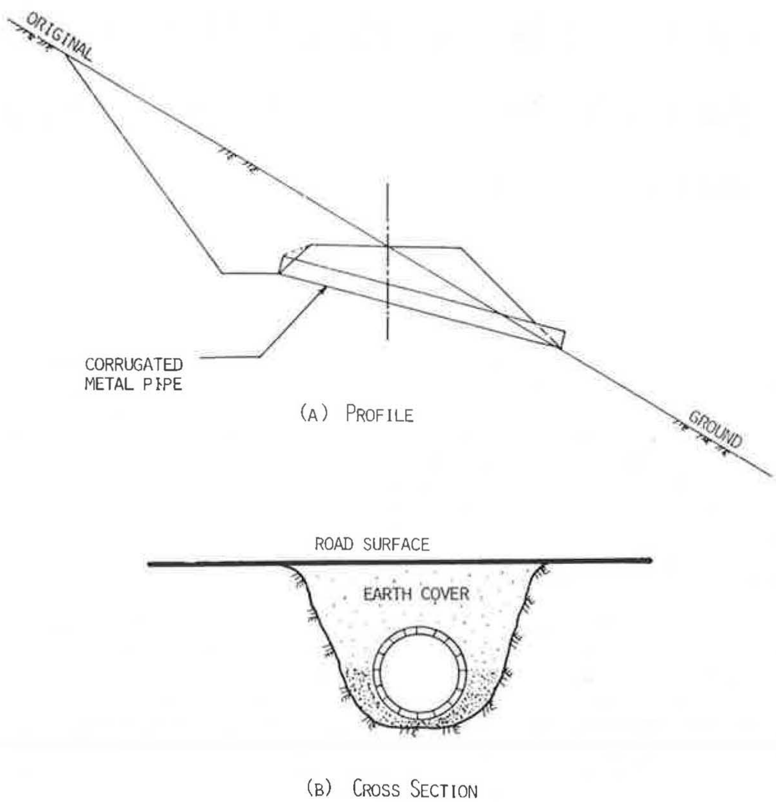

FIGURE 1 Profile and cross-section views of typical ditch relief culvert used on forest roads.

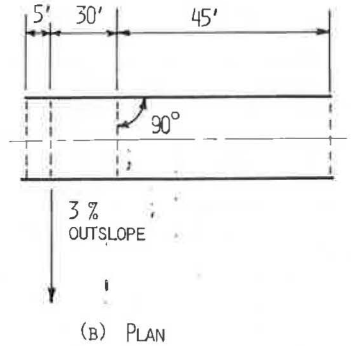

FIGURE 2 Plan and profile of broad-based dip currently used by national forests in North Carolina.

Design criteria have been established for both broad-based dips and culverts, although actual device dimensions and other details may vary from one geographic region to another. Most drainage devices, if constructed according to specifications and if placed at an appropriate location, will perform satisfactorily for many years. Although dips and culverts each have their place as a drainage device on low-standard roads, there are certain conditions under which one is more appropriate than the other. Apparently, however, no formal engineering economic analysis has ever been made of this issue.

COSTS ASSOCIATED WITH DRAINAGE STRUCTURES

There are three major cost categories pertinent to the evaluation of the two types of roadway drainage structures under discussion here: construction, maintenance, and road user costs. In this section a summary of published qualitative and quantitative data relative to broad-based dips is presented. Data on culverts will not be presented here because their design criteria are relatively well established and their costs can be determined by using standard construction estimating procedures.

Construction Costs

Although engineers and foresters appear to agree that the initial cost of dips is less than the cost of metal culverts, there is limited documentation in the published literature comparing the costs of the two devices. Koger (1) reported on a study in which the construction of three dips was observed. He found that a dip could be constructed by an operator using a 105-hp tractor in approximately 3 min. Total construction cost was estimated at $1.17, which does not include the time required to move the bulldozer to the next broad-based dip or any materials, such as gravel surfacing. The results of this study are open to question for several reasons. The sample size of three is too small to determine a reliable average cost. The reported construction time of 3 min also appears unrealistic, because the construc-

tion of a dip involves creating a structure that is relatively detailed.

Kochenderfer and Wendel (2) presented perhaps the most detailed information available up to that point on the construction and operation of broad-based dips. On the study roadway, 6 hr was required to construct 19 broad-based dips; installation of two culverts required 5 hr. The total cost of a graveled dip averaged $128.

Maintenance Costs

Both dips and culverts require periodic maintenance to ensure that the devices remain in working order. Because culverts are the traditional drainage structures on many roads, maintenance crews are generally familiar with the procedures for maintaining ditches and culverts. The training and experience of equipment operators must be considered in the maintenance of dips. Hewlett et al. (3) noted that properly constructed dips should require little maintenance except under heavy traffic or off-season use. It was reported that some dips held up under weekly automobile trips for 15 years without reshaping or maintenance. However, traffic must be limited during unfavorable weather to reduce soil erosion and maintenance costs.

In the U.S. Forest Service Transportation Engineering Handbook (4), it is noted that dips are intended for low-volume, low-speed roads where there may be extended periods of nonuse. When properly constructed, dips provide a relatively maintenance-free structure. Although the initial cost of a dip may be cheaper than purchasing and installing a culvert pipe, unless the dip is properly designed and constructed, the total cost (including maintenance) may be more than if a pipe culvert had been installed. Several disadvantages of dips were noted, including low travel speeds, poor riding comfort, difficult blading of the traveled way, and possible adverse effects on water quality.

Maintenance costs are obviously an important factor in decision making. To date, however, no published maintenance costs comparing culverts with broad-based dips have appeared in the literature.

Road User Costs

The volume and types of vehicles traversing a road can influence the drainage device selected. Culverts should be used exclusively on roads open to automobile traffic. This is because vehicles with low ground clearance may have trouble negotiating broad-based dips. Because of the ground clearance problem, dips of the design being discussed here should not be used on paved roads.

Culverts could be said to be invisible to the road user in that they can go unnoticed by vehicle operators. However, the associated ditch and headwall can pose hazards to vehicles that stray from the roadway. In contrast, dips have a definite impact on the road user. Most vehicles will have to slow down to negotiate the rather abrupt change in grade. Cook and Hewlett (5) noted that an improperly designed short dip will cause a jolting of the driver and the vehicle regardless of the reduction in speed. Furthermore, the vehicle will undergo some twisting action, which, in extreme cases, could cause damage to the vehicle, the load, or both. The cost of vehicle damage and wear, time loss, and additional fuel consumption attributable to dips must be considered in comparing the two drainage devices. In an attempt to obtain specific data on this issue, Hafterson (6) sought a dip design that would be hydraulically efficient as well as comfortable to drivers. Relationships were developed between vehicle speed and vertical acceleration with a view toward driver comfort. The dip was correlated to vertical curvature and limits for the maximum sharpness of curvature were determined by using vehicle underclearance as a criterion.

As is apparent from the preceding discussion, the information found in the literature was of a general and sometimes subjective nature. Little, if any, quantitative benefit/cost data or economic analyses were available to permit an objective comparison to be made between dips and culverts. This points to the need for additional research concerning the economics of broad-based dips. It must be emphasized that for valid comparisons to be made between culverts and dips, it is necessary to examine maintenance and road user costs in addition to construction costs.

STUDY OBJECTIVES

A research project was undertaken in the Monongahela National Forest (West Virginia) to determine, by using an engineering economic analysis, under what conditions conventional metal culverts are more appropriate than broad-based dips on low-volume roads. To address this overall goal, several specific objectives were established:

- To conduct a literature review to acquire qualitative and quantitative information relative to the use and performance of broad-based dips and conventional metal culverts on low-volume roads (a brief synopsis of this review was presented in the preceding section);
- To conduct an economic analysis of 18-in. aluminum culverts (the typical device used in the study area) and broad-based dips, considering construction, maintenance, and road user costs; and
- To recommend, on the basis of the foregoing, specific conditions under which culverts, broad-based dips, or both should be installed.

DATA COLLECTION

The primary component of the data collection effort was input from practitioners. The objective here was to contact persons with experience or knowledge in design, construction, maintenance, operations, or all of these aspects of logging roads to acquire information about broad-based dips and 18-in. aluminum pipe culverts. Persons providing input for this part of the study can be categorized as Forest Service personnel, private foresters, logging-road contractors, and log haulers. Persons contacted included those both within and outside West Virginia but the study was generally confined to the Appalachian Region of the eastern United States.

Contact with Forest Service personnel was through telephone conversations, personal interviews, and field trips. These interviews supplied valuable insight about technical details and cost information; field trips provided an opportunity to observe at first hand the performance of dips and culverts.

Based on conversations with Forest Service personnel, two lists of persons or firms were developed: those involved with road construction and maintenance, for example, private foresters and contractors, and those involved in log-hauling operations. It was decided to telephone those on the first list to request information about drainage, construction, and maintenance procedures and costs. In order to ensure that the same type of information

was received from each conversation, a questionnaire was designed to be administered during the telephone conversations. Questions asked pertained to the labor, materials, and equipment required to build and maintain dips and culverts. Relatively detailed cost information for each of these three items was also sought from contractors. From the telephone survey, bid prices for the construction of both broad-based dips and 18-in. aluminum culverts were obtained.

Because the data were confidential, respondents were encouraged to give estimations for construction and maintenance costs; however, several contractors were willing to provide actual bid prices.

As noted earlier, although the literature mentions a number of road user consequences of broad-based dips, there was little, if any, hard data available, especially in the area of drivers' perceptions of broad-based dips. However, because broad-based dips must be traversed by log-hauling vehicles, the investigators believed that any criteria for determining need and location should also include input from vehicle drivers. A one-page questionnaire, which could be administered by telephone or in person, was developed to obtain information on truck drivers' experiences with and perceptions of broad-based dips. Specifically, the survey form sought information on the type of driving done by the vehicle operator and on the impacts of dips on road users (e.g., vehicle damage and time loss). More details about the truck driver questionnaire are contained in the full research report (7).

One of the findings of the literature review was the implication that broad-based dips result in an extra cost to road users. Lost time, increased fuel consumption, and wear and tear on the vehicle can occur when a vehicle reduces speed to traverse a dip. In an attempt to determine whether broad-based dips result in a loss of time or other resources to the log hauler, a pilot study was formulated to analyze travel times of logging trucks over both dipped and undipped sections of a logging road in the Monongahela National Forest (West Virginia).

The approach selected for the study was to determine how long it took to travel a measured distance. Two adjacent 100-ft sections of an active logging road of 4 percent grade with similar surfacing and other characteristics, one containing a broad-based dip and the other not, were marked in the field. One stopwatch was used to record travel time over the dipped section and another for the undipped section.

For the vehicles checked during a 1-day period, the time loss due to the dip was negligible (on the order of 1 sec per dip). Given that a logging truck typically makes only two or three round trips per day over the haul road (in the Appalachian Region), an extremely large number of dips would be required to have any significant effect on truck travel time. Because any time loss that results from a broad-based dip would be too insignificant to justify the expense of a large-scale study of the type described here, it was decided not to pursue a full-scale travel-time study as originally planned. Note that the study just described did not attempt to evaluate the increased fuel consumption and wear and tear on the vehicle brought about by the existence of the dip; for this evaluation, the published literature was consulted.

RESULTS

Construction Costs

Thirteen contractors from the central Appalachian Region responded to the survey on drainage device construction and maintenance procedures and costs. Eleven firms provided actual cost figures; two others furnished only qualitative information. Although cooperation from the contractors was generally excellent, there were a few firms that could not be reached by telephone and several others that elected not to participate in the study. However, it is believed that the firms providing data formed a representative sample of logging-road contractors.

In response to the question of whether the firm had a preference for dips or culverts, 80 percent of the contractors said that they preferred culverts. Those favoring dips cited economics as the primary reason. One specifically remarked that there was no ditch line to worry about. In contrast, a wide variety of reasons was given by those preferring culverts. Most frequently cited was the opinion that dips require more maintenance than culverts. Several others mentioned hidden costs incurred by trucks hauling construction material as they traverse the dips (additional wear and tear on the truck) and those associated with maintaining dips during the construction period. Three firms mentioned that although dips look simple, there are so many variables involved that it is difficult for equipment operators to install them properly.

This same theme was found in responses to the question about whether the type of drainage structure specified in project plans has an effect on the contractor's bid. Eight firms noted that their bids were affected when dips were specified. Although two contractors stated that dips would be less expensive than culverts, the majority of firms indicated that they would increase their bid where dips were specified. The main reason given was that dips were harder to bid on because they involve more guesswork than pipe culverts. The difficulty of constructing dips was mentioned several times. One contractor noted that although he currently did not like to install dips, once he had learned how to install them properly, he would probably prefer them to culverts. These results suggest that contractors' attitudes toward broad-based dips might improve if they could acquire hands-on experience or training in proper field construction of them.

Although contractors were in general agreement about the types of equipment used to construct dips and culverts, there were different practices in terms of the labor requirements. In general, construction of a culvert requires a backhoe, a tamper, and hand tools. Construction of a dip typically requires only a small bulldozer or grader. All but one contractor reported that three persons (a foreman, a laborer, and an equipment operator) were required to construct a culvert. Labor requirements for broad-based dips ranged from one to five persons, with the typical value being two (a foreman and an equipment operator). This wide variation in personnel requirements may be due to the just-mentioned practice by some contractors of intentionally bidding high on broad-based dips. That a foreman, an equipment operator, and three laborers would be required to build a dip appears somewhat extravagant when the literature reports the construction of many successful dips by a single bulldozer operator.

The reported time required to construct each drainage device was surprisingly similar. Construction time for culverts ranged from 2 to 5 hr with a mean of 3.2 hr. Dip construction time ranged from 1 to 7.5 hr with a mean of 3.2 hr. In both cases, the median construction time was 3 hr. Once again, it is believed that the time to construct a dip has been intentionally inflated in certain cases.

Contractors were asked to furnish estimated cost figures for constructing dips and culverts. The cost data are summarized as follows [the culvert used is an 18-in. aluminum corrugated metal pipe (CMP)]:

Cost Category	Cost ($) Culvert	Broad-Based Dip
Mean	546	373
Median	530	300
Range	230-805	68-1,000
Standard deviation	153	227

It is apparent that the average reported cost of a broad-based dip is significantly less than that of a pipe culvert. Also apparent is the wide variability in cost of broad-based dips. Reasons for this variation have been discussed previously.

To validate the cost data compiled from the contractor survey and to supplement the survey's admittedly small sample size, additional sources of cost data were sought. A principal source was data compiled for the researchers by Monongahela National Forest engineers for a sample of 16 road projects in the forest. However, these data were limited to culverts (average length, 25.3 ft) and did not include broad-based dips. The cost per pipe culvert varied from $391.50 to $765 with a mean cost of $527.10 per culvert ($23.0 per linear foot of culvert). Note that this is virtually identical to the median culvert cost ($530) determined from the contractor survey.

A third source of drainage structure cost data was a bid tabulation for the Falls Road project, also provided by Monongahela National Forest engineers. Falls Road was (through summer 1984) the only project in the Monongahela National Forest for which dip construction was identified as a bid item. It should be noted that the item to be bid was for excavation only; stone surfacing was not included. The bid tabulation also included cost data for 18-in. CMP culverts. A summary of the dip and culvert construction cost information derived from the Falls Road bid tabulation is as follows:

Cost Category	Cost ($) Culvert	Broad-Based Dip
Engineer's estimate	547	70
Mean	632	500
Range	426-760	200-1,000
Standard deviation	161	356

Note that the average costs of both culverts and dips are significantly higher than those determined from the contractor survey data. As before, the dips demonstrate a greater variability in cost.

For the economic analysis, it was necessary to have a single typical cost for dips and culverts. On the basis of the preceding discussion, it was decided to use the following drainage device construction costs in the economic analysis: 18-in. CMP culvert, $530; broad-based dip, $350.

Maintenance Costs

Data on drainage device maintenance costs were not readily available. None of the contractors responding to the telephone survey indicated that they had experience with maintenance of drainage structures. Maintenance cost data could not be obtained from Monongahela National Forest engineers because records were not kept of the type of data requested by the researchers. The only maintenance cost data that could be obtained were those acquired through telephone conversations with private foresters or those estimated by the researchers on the basis of the literature review. The costs that were obtained or estimated were on a per-mile rather than a per-structure basis.

According to estimates provided by private foresters, approximately $100 per mile per year would be required to maintain ditches and culverts. This cost includes maintaining the ditch line as well as checking and cleaning the heads of culverts. The cost was based on an average of 12 culverts per mile; thus the average annual maintenance cost per culvert was $8.33.

Because no maintenance cost data could be found for broad-based dips, the costs were estimated by the researchers. According to private foresters, a bulldozer or motor grader should be able to dig 4 to 5 mi of ditch line per day (average of 4.5 mi per day). Under the assumption that two to three passes of the grader or bulldozer might have to be made to maintain the dip, it appears reasonable to expect that 1.5 mi of dipped road could be maintained per day. Assuming a dip spacing of 200 ft, this would mean that 40 dips could be maintained daily or 5 dips per hour during an 8-hr work day. In Monongahela National Forest in summer 1984, costs for a bulldozer and operator were approximately $50 per hour. Because such maintenance should be done once a year, the annual dip maintenance cost was estimated at $10 per dip.

Truck Driver Survey

The last of the three cost elements to be considered in the economic analysis was the road user costs. Before the road user cost estimates are actually presented, however, a brief review of the results of the truck driver questionnaire will be given to provide a driver's perspective of the impacts of broad-based dips.

All respondents drove single-unit, or straight, trucks; no tractor-semitrailer operators responded. Although the predominant truck in Appalachian forests is the single-unit variety, there are at least a few tractor-semitrailers in operation. Just over three-fourths of the respondents (78 percent) drove tandem rigs (two rear axles), whereas 22 percent drove triaxle trucks (three rear axles). The average wheelbase for the tandems was 213 in. compared with 268 in. for the triaxles.

A high percentage of respondents (89 percent) made either two or three round trips (between loading point and mill) per day. On logging roads having no broad-based dips, drivers estimated that they traveled at an average speed of 13.8 mph. It is interesting to note that the average reported speed for roads with dips was exactly one-half this value, or 6.9 mph. All respondents said that dips caused them to be delayed in traveling over logging roads.

Almost 90 percent of the respondents reported feeling physical discomfort when passing through a broad-based dip. Reasons given ranged from "throws driver around cab" to "bouncing, jarring and twisting of truck causes driver to feel off-balance." Slightly more than one-third of the drivers indicated that a dip had caused damage to a truck or load. A cracked or broken frame was the most frequent complaint. One driver reported that trucks get "hung up" in deep and narrow dips.

Three-fourths of the respondents thought that some dips scare drivers because they look like they might damage the truck or the load. A variety of suggestions was given on how to build dips that would be less intimidating to drivers. Half of the drivers suggested eliminating the use of dips entirely.

It was clear from the survey results that truck drivers in general have a negative attitude toward broad-based dips. One limitation that must be kept in mind when evaluating the results is that it was

not known whether the dips with which respondents were familiar met appropriate standards. One timber purchaser contacted by telephone noted that properly constructed dips created no problems for trucks or drivers. However, the firm had sometimes experienced vehicle damage and other problems because of improperly constructed dips. It is the researchers' belief that more than a few dips are improperly constructed. These dips, because of the problems they create, might tend to be remembered by drivers and cause them to dislike all dips even though the majority of drainage structures can be traversed with no problems. It is interesting to note that the possibility of trucks overturning in dips located on horizontal curves, which had been cited by some engineers as a possible problem, was not even mentioned by survey respondents.

The data collection section presented results from a pilot study conducted to assess the effects of dips on vehicle travel time. It was found that, for dips built according to standards, the vehicle delay attributable to a dip is on the order of a few seconds. On this basis, a vehicle would have to traverse 10 or more dips to decrease its travel time by 1 min. Although drivers have reason to believe that they are being delayed at each dip, the overall impact is minimal. The investigators believe that the travel-time effects of dips are not an issue, because the travel time saved on an undipped road is not sufficient (unless the truck makes a large number of round trips per day) to cause an increase in the number of round trips that can be made per day.

Road User Costs

The road user consequences, either beneficial or adverse, of drainage structures on forest roads occur primarily through the operating cost of motor vehicles, the change in highway accidents, and the change in travel time. Because of the low traffic volume on forest roads, motor vehicle accidents are rare events, and in most cases accident data are not generally available. Therefore, accident costs have not been included as a road user consequence in this study. No evidence could be found either in the literature or during the data collection task to indicate that the accident experience with broad-based dips was any different from that with culverts.

Culverts can be said to be invisible to road users in that vehicles can traverse them at the design speed of the roadway and neither vehicles nor drivers are subjected to any extraordinary forces or sensations. An exception to this statement would be the situation in which the roadway is constricted by severe erosion of the culvert inlet or outlet, thus causing the vehicle to reduce speed while traversing the culvert. However, this is a correctable situation, so it is not appropriate to include such excess travel-time costs in the economic analysis.

Dips, on the other hand, because of their shape, require that all vehicles reduce speed while traversing the drainage device. It was shown earlier that the extra time involved to negotiate a broad-based dip was negligible. Because this conclusion was based on limited field data, it was decided to confirm these results by using data from Winfrey's (8) comprehensive text on highway economics. Winfrey (8) presents tables showing the excess hours consumed (excess above continuing at initial speed) per speed-change cycle, in which speed-change cycle consists of the reduction in speed and the return to the initial speed. Although separate tables were not developed for the three-axle single-unit truck typically found in log-hauling operations, Winfrey noted that for highway economic analysis purposes, the three-axle single-unit truck could be put in the same class as the 40-kip tractor-semitrailer. To provide a conservative estimate of excess time, an initial speed of 15 mph and a reduced speed of 5 mph were assumed based on the 13.8 mph and 6.9 mph speeds reported in the truck driver survey. Under these conditions, there would be 0.00134 hr consumed per cycle, or 0.08 excess min per dip. On a per-dip basis, this is a negligible quantity. However, it was believed worthwhile to examine the effect of a large number of dips. Assumptions were made that (a) the typical log-haul road contains 17 dips (this was the average value for 19 sites examined in a related study) and (b) the typical logging truck makes five passes (2.5 round trips) per day over these dips. The time lost by drivers under these conditions is

(17 dips/trip) x (5 trips/day) x (0.08 min/dip) = 6.8 min/day.

Because the driver cannot put this small increment of time to any productive use in terms of increasing the number of round trips made per day, it was concluded that under typical conditions in Appalachian forests, the additional travel time through a broad-based dip can be neglected for purposes of economic analysis.

Although broad-based dips do not produce a significant increase in travel time, there is an increase in fuel consumption and wear and tear on vehicles brought about by the presence of a dip. Because project resources did not permit the monitoring and collection of actual logging truck operating costs, vehicle running cost data available in the literature had to be used to estimate the road user impact of dips. Relatively little is known about the factors that affect log-hauling costs. Cost data available presented general information based on hauls between landings and the mill. Such data were not detailed enough to permit computation of the additional cost generated by traversing a broad-based dip. Fortunately, Winfrey (8) included data that could be adapted to the situation. He presented a table containing the excess cost in dollars of speed-change cycles (excess cost above continuing at initial speed) for a 40-kip tractor-semitrailer operating on a high-type pavement in good condition. If an initial speed of 15 mph and a reduced speed of 5 mph are assumed, the table yields an excess cost of $19.42 per 1,000 speed-change cycles. The cost includes fuel, tires, engine oil, maintenance, and depreciation. However, the cost was based on an 18-cent/gal price of gasoline and other unit prices typical of the mid- to late 1960s. Recalculation of this figure using current costs was complicated because Winfrey (8) did not provide a detailed breakdown of the cost components for the speed-change situation. Thus, current costs had to be estimated in the following manner.

An updated (1977) version of Winfrey's data was found in the literature (9). The 1977 cost per 1,000 speed-change cycles for trucks with an initial speed of 15 mph and a reduced speed of 5 mph was $45. Based on U.S. Department of Labor consumer price index information, the 1977 cost figures were converted to current (spring 1984) dollars by multiplying them by 1.7. This yielded an excess cost of approximately $77 per 1,000 speed-change cycles, or $0.077 per dip. If the typical conditions used earlier (17 dips traversed, five passes per day) are assumed, the excess cost per vehicle may be determined:

(17 dips/trip) x (5 trips/day) x ($0.077/dip) = $6.54/day.

This value probably represents an underestimate of the actual excess costs incurred because it does not account for the additional vehicle wear and tear created by the twisting motion induced by the broad-based dip.

It must be emphasized that the road user costs for broad-based dips just described represent costs over and above those incurred on a road without dips. In this analysis, the actual road user costs for a section of forest road will not be calculated. The road user costs presented here represent the extra cost attributable to broad-based dips.

Overall Evaluation

The total cost associated with a drainage device represents the sum of construction, maintenance, and road user costs. For the purposes of this analysis, the equivalent-annual-cost approach was used to compare culverts with broad-based dips. Assuming, as noted earlier, that only the additional road user costs incurred by dips are of interest, the equivalent annual cost (EAC) of a CMP culvert would be

$$\text{EAC} = \text{construction cost} \times (\text{crf-i-n}) + \text{annual maintenance cost} \quad (1)$$

The cost for a broad-based dip would be

$$\text{EAC} = \text{construction cost} \times (\text{crf-i-n}) + \text{annual maintenance cost} + \text{additional annual road user cost} \quad (2)$$

where

crf = capital recovery factor applied to convert the initial cost (a present value) to an annual cost,
i = discount rate, and
n = expected life of drainage structure.

Given the difficulties associated with selecting an appropriate discount rate and given that high rates tend to favor projects with low costs initially but high costs later, whereas low rates tend to favor projects with high costs initially and low costs later, it was decided to perform the economic analysis with three different discount rates: 5, 10, and 15 percent.

Because of the large number of variables affecting drainage device durability, it is difficult to specify a single value for n. However, on the basis of discussions with practitioners, it appeared that 20 years was a reasonable lifetime for aluminum pipe culverts. Because of the lack of any information to the contrary, it was assumed that, with proper maintenance, broad-based dips would also have an expected life of 20 years.

The following costs were derived earlier for use in Equations 1 and 2:

Cost	Culvert ($)	Broad-Based Dip ($)
Construction	530.00	350.00
Annual maintenance	8.33	10.00
Road user	--	0.077/vehicle

It should be noted that road user cost is a function of traffic volume. For the purpose of this study, it was assumed that hauling takes place only during good weather, that is, May 1 through September 30 of each year. This represents a period of 100 working days. It was recognized that depending on the manner in which the timber or mining operation is managed, there could be a variety of different traffic conditions. Two scenarios were considered in the analysis presented here:

1. Traffic volume remains constant for each year of the 20-year life of the structure, and
2. Traffic uses the road for the first 3 years of its existence to harvest timber, but then no additional traffic, other than perhaps a negligible amount of administrative traffic, uses the road for the next 17 years.

Traffic volumes in five-vehicle-per-day increments from 0 to 15 vehicles per day (the range of traffic volumes experienced by the study sites) were considered in this analysis. Other scenarios could be developed by applying the procedures outlined here. Using appropriate compound interest factors, equivalent annual drainage device costs were calculated for these traffic volume increments for the two scenarios just described.

Plots were made of equivalent annual cost versus average daily traffic (ADT) (daily traffic volume during the hauling period as opposed to annual ADT) to determine the traffic volumes at which one device is less expensive than another. Results for the first scenario for discount rates of 5, 10, and 15 percent are shown in Figure 3. Because of the high

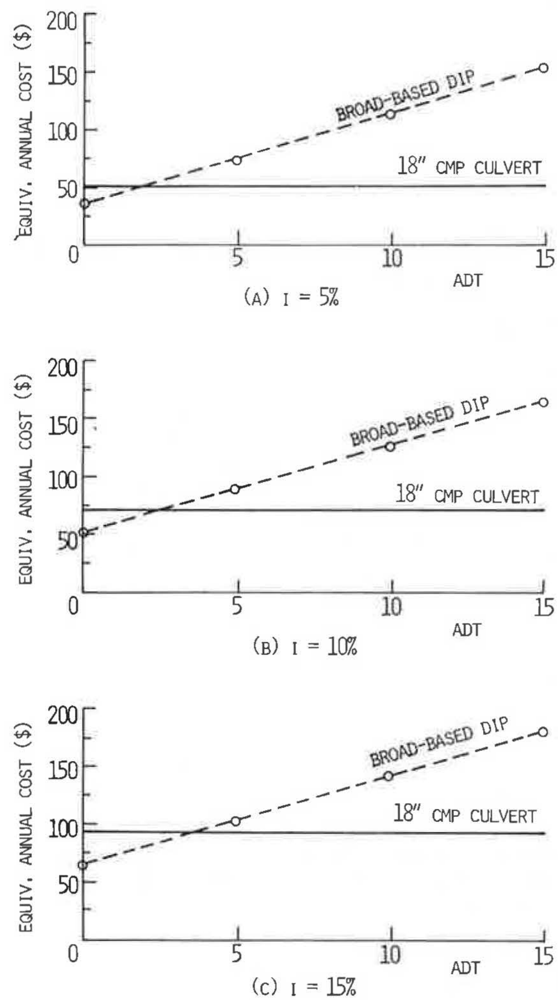

FIGURE 3 Relationship between average daily traffic and equivalent annual cost for dip and culvert, Scenario 1, for various interest rates.

road user costs incurred by assuming constant traffic volume over the entire 20-year life of the structure, it can be seen that dips are less expensive than culverts only when traffic volumes are in the range of less than five vehicles per day. This represents a worst-case condition for traffic, because most logging roads with dips would not carry the same volume year after year; there would be gaps of several years or more when no harvesting (and therefore, no hauling) occurred.

The second scenario was believed to represent expected traffic conditions more realistically. Once again, plots were made of equivalent annual cost versus ADT. These results are shown in Figure 4 for discount rates of 5, 10, and 15 percent. The plots indicate that dips are the lower-cost drainage device in an ADT range of about 8 to 12 vehicles per day and less. When traffic volumes exceed approximately 15 vehicles per day, culverts become more economical than broad-based dips because of the user costs incurred with dips.

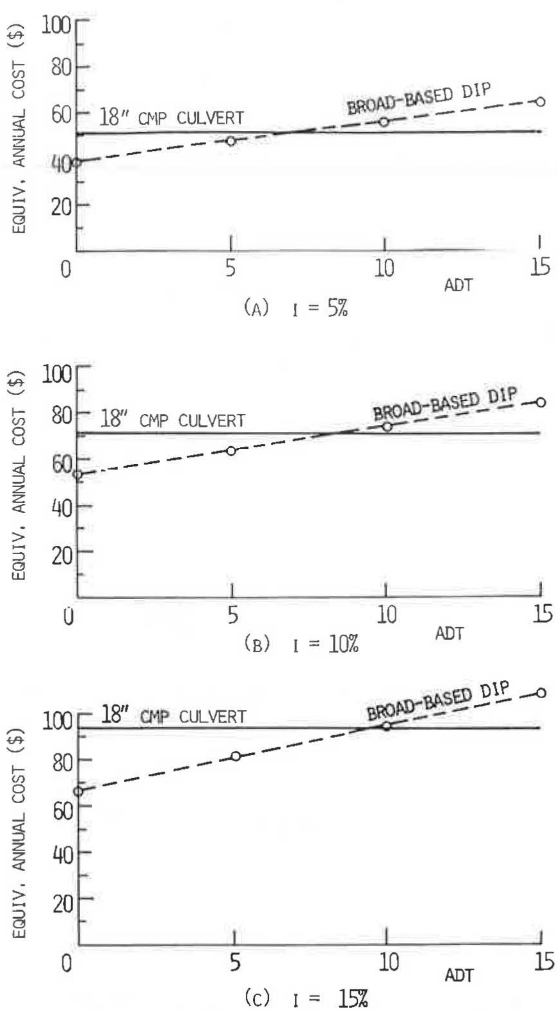

FIGURE 4 Relationship between average daily traffic and equivalent annual cost for dip and culvert, Scenario 2, for various interest rates.

CONCLUSIONS AND RECOMMENDATIONS

The results of the economic analysis confirmed what had been learned from the literature review and practitioner input tasks of the study. That is, broad-based dips are less expensive than pipe culverts for roads carrying traffic volumes in the range of 5 to 10 vehicles per day. At higher volumes, the increased road user costs associated with broad-based dips make culverts the more economical alternative. It is recommended that any road handling in excess of 15 vehicles per day, whether subjected to this traffic level for 2 years or for 20 years, use strictly culverts because of the high user cost associated with dips. Dips are appropriate on roads with traffic volumes less than 5 vehicles per day, assuming that their use is not ruled out by design, soils, or hydrologic factors. For traffic volumes between 5 and 15 vehicles per day, the decision to use a dip or culvert is influenced by the nature of vehicle use of the road. If the road is used each year for the life of the road (assumed to be 20 years), the high road user costs associated with broad-based dips make culverts the preferred drainage device. If the road is to be used only during the first few years of its life and then "put to sleep" for some period of time, broad-based dips are the more economical drainage structure.

It must be emphasized that there are certain conditions under which one drainage device is more appropriate than the other and a decision based solely on an economic analysis could lead to serious problems in terms of greatly increased future maintenance or road user costs. The decision whether to use a culvert or a dip in a particular situation should be based on both economics and physical factors such as design elements, soils and geology, hydrology, and traffic factors. The authors have developed a framework, incorporating economics and other factors, that can be used in deciding whether to use metal pipe culverts or broad-based dips to handle cross drainage on low-volume roads. A paper describing the decision-making framework is in preparation.

This study indicated an important aspect that could influence the use of broad-based dips: the negative attitude toward them on the part of contractors and truck drivers. There was evidence that contractor attitudes would improve if they could gain experience in dip construction. Hands-on workshops in proper field construction of broad-based dips are recommended as one way of providing this experience. Such workshops should also improve the quality of dips constructed by those contractors already having experience.

It was noted in the economic analysis that the additional road user costs incurred by traversing a broad-based dip were underestimated because of the difficulty in estimating vehicle damage associated with the dip. Additional research in this area appears warranted. A truck could be instrumented with strain gauges or other devices to monitor the motion of the truck frame and other components as the truck negotiates a number of different broad-based dips. By correlating this movement with loading condition, material properties, and other factors of this nature, one should be able to specify better the dimensions of a "tolerable" dip.

In the economic analysis, the cost data with the greatest uncertainty were those for dip maintenance. It is recommended that dip maintenance cost data be acquired for a large number of dips. Not only would this provide more accurate cost data so that the economic analysis could be refined, but it would also yield a better understanding of dip maintenance procedures and frequency.

ACKNOWLEDGMENTS

This paper is based on research supported by funds provided by the Northeastern Forest Experiment Sta-

tion, Forest Service, U.S. Department of Agriculture. Special thanks are extended to Randall Burks, whose efforts contributed a great deal to laying the groundwork for the successful completion of this study. Appreciation is also expressed to the staff of the Monongahela National Forest for their assistance and cooperation throughout the study.

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Publication of this paper sponsored by Committee on Low Volume Roads.

Effects of County Highway Management Practices on Maintenance Costs for Unpaved Roads in Indiana

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ABSTRACT

Costing highway maintenance for both budgeting and performance reporting is a critical problem, particularly for counties and other local highway agencies. In Indiana, guidelines are available for preparing county highway accounts. However, the success with implementation has varied among counties. Road maintenance costs calculated for four Indiana counties are presented. The effects of management practices and policies such as grading frequency, regraveling, and equipment use on maintenance costs of unpaved roads are discussed. In the analysis presented, traffic volume is found to be an important consideration, especially in determining grading frequencies. However, another important factor, road condition or performance, is seldom routinely measured by Indiana counties. Although the condition of unpaved roads is not discussed in detail in this paper, it is considered important to monitor road condition periodically and relate it to the maintenance strategies adopted. The problems of data collection and accurate maintenance costing are traced to deficiencies in the accounting system, which does not enable costing by activity, road type, or location. An improvement in the maintenance cost accounting system is considered necessary for better cost control as well as accurate life-cycle costing.

The ability to estimate the costs of various construction and maintenance activities is an important element in effective highway planning and management. Cost information is useful for both budgeting and accomplishment reporting. However, costing highway maintenance is a critical problem, particularly for counties and other local highway agencies. Generally, because construction and major capital improvement projects are bid by private contractors, the costs are usually well known. The variations from the original contract costs can usually be monitored and the final costs can be estimated. Because most of the maintenance is performed by an agency's own personnel, the ability to estimate highway maintenance cost depends on the maintenance cost-accounting or management system in operation. In Indiana, county highway departments differ in their approach to management, and some of them are unable to monitor and keep adequate records of their operations for accurate budgeting or for reporting accomplishment.

In this paper the procedure used to determine county highway maintenance costs in Indiana is discussed; this procedure is part of research undertaken by the Highway Extension and Research Project for Indiana Counties and Cities (HERPICC) at Purdue University (1). The costs of maintenance activities for unpaved roads undertaken in 1983 and also some maintenance activities for paved roads are determined from data available from county records or collected specially as part of the study. In addition, the impact of various county highway management practices on maintenance costs, particularly for unpaved road surfaces, is also examined.

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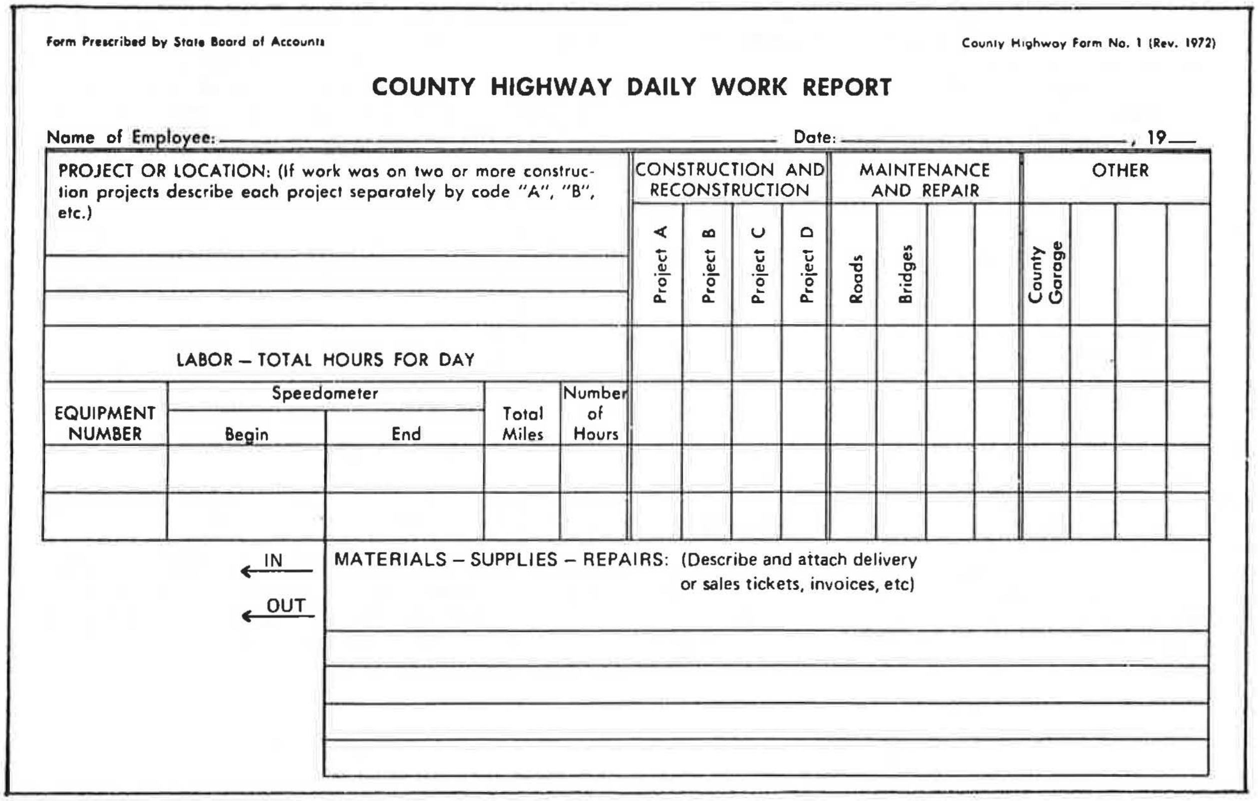
METHODS OF DATA COLLECTION

To obtain the required cost information, arrangements were made with engineers and supervisors from five counties--Bartholomew, Huntington, Jasper, Tippecanoe, and Warrick--to monitor and maintain records for maintenance activities on selected unpaved road sections for 1983. The maintenance activities included blading or grading, regraveling or spot regraveling, brush cutting, mowing, side ditching, snow plowing, and sign maintenance. As much as possible, the existing data reporting system used as part of county maintenance cost accounting was used. This data collection approach was aimed at reducing problems likely to arise if new data forms were introduced. The only special requirement was that equipment operators and other personnel working in any activity on the selected roads were to provide detailed information for certain items on the daily work reports. Information reported included labor time, equipment time, and distance traveled as well as material types and quantity and costs for each activity. An example of the daily work report form currently used by Indiana counties is shown in Figure 1. Because maintenance activities are not adequately identified on the form, they were described in the space provided for project or location. These details were discussed with the county highway officials before the special study.

Another approach adopted was to analyze past annual operating reports submitted by the counties, which provide the only reported source of information on county operations. Costs from Mason County, in Washington State (2), which has implemented a maintenance management system, are also presented for comparison with costs estimated for Indiana.

The data collection procedure was implemented fairly successfully by three counties, especially when the highway engineers, supervisors, or clerks became personally involved in monitoring the information. In another case, the information was ex-



Form Prescribed by State Board of Accounts

County Highway Form No. 1 (Rev. 1972)

COUNTY HIGHWAY DAILY WORK REPORT

Name of Employee: Date: , 19

PROJECT OR LOCATION: (If work was on two or more construction projects describe each project separately by code "A", "B", etc.)

CONSTRUCTION AND RECONSTRUCTION: Project A, Project B, Project C, Project D

MAINTENANCE AND REPAIR: Roads, Bridges

OTHER: County Garage

LABOR – TOTAL HOURS FOR DAY

EQUIPMENT NUMBER | Speedometer: Begin, End | Total Miles | Number of Hours

IN

OUT

MATERIALS – SUPPLIES – REPAIRS: (Describe and attach delivery or sales tickets, invoices, etc)

FIGURE 1 Indiana county highway daily work report form.

TABLE 1 Breakdown of Annual Maintenance Expenditure by County, 1980-1983

Item	Percent Expenditure by County: Bartholomew[a]	Huntington[b]	Jasper[c]	Tippecanoe[d]	Warrick[e]
Administrative and operational overhead	22.1	26.6	25.3	23.0	21.7
Garage and mechanical overhead	9.8	18.6	15.7	9.9	14.6
Equipment	3.4	2.3	6.3	7.5	8.1
Stone and aggregate	9.5	7.0	12.3	8.0	4.7
Bituminous and mixes	13.9	3.6	5.4	13.5	5.3
Culverts and tiles	1.5	1.9	2.1	1.0	–
Road signs	0.4	1.1	1.3	0.4	0.6
Other materials	1.5	3.2	0.2	9.6	15.5
Labor	19.9	23.4	24.4	17.7	25.0
Contractual services	17.9	12.2	7.0	9.6	4.6

Note: The expenditures given are 4-year averages.

[a]Total spent, $1,137,000.
[b]Total spent, $1,118,000.
[c]Total spent, $919,000.
[d]Total spent, $2,024,000.
[e]Total spent, $1,399,000.

tracted from the daily work reports and accounting records of 1983 supplemented by interviews with the bookkeepers, the highway supervisor, and the grader operators assigned to the roads in the study. The main problem observed during data collection was that the daily work report form, in spite of the provision for other entries, served in most cases as mainly a labor time card. The extent to which other details were provided depended on the individuals completing the forms within the same county. The success at implementation also varied from county to county. Reporting without the necessary detail on equipment and material use by location was sufficient for annual operating reports. The annual reports showed mainly gross summaries of labor, equipment, and material costs for major budget classifications. These summaries are usually provided from the various accounting ledgers and forms kept by the counties. However, suggestions were made for modifying the current reporting system to enable maintenance activity costing; these suggestions are described in greater detail by Riverson (1).

UNIT MAINTENANCE COSTS ESTIMATED FROM ANNUAL REPORTS

In general, reasonable estimates can be made of the mix of labor, materials, equipment, and overhead from annual expenditure reports. A breakdown of major cost items for maintenance and repair calculated from annual reports is given in Table 1. Following procedures set out in the county accounting guide (3), the administrative and operational overhead and the garage and mechanical overhead were estimated from annual reports. Items included in overhead are as follows:

1. Administrative and operational overhead
   a. Personal services and administrative staff
   b. Office supplies
   c. Other supplies and charges
   d. Employee benefits
   e. Communication and transportation
   f. Insurance (excluding garage)
   g. Professional services and other charges
   h. Capital outlays (properties)
2. Garage and mechanical overhead
   a. Salaries of garage mechanics and other staff
   b. Garage and motor supplies
   c. Insurance premiums (garage only)
   d. Utilities
   e. Repairs to garage and service building
   f. Rents (garage only)

From the foregoing analysis, unit costs of maintenance and repair for all roads independent of surface type ranged from $990/mi to $2,310/mi. The variation in costs appears to be more a function of revenue received than mileage maintained. The county with the highest mileage had the least unit cost per mile of road maintained. Comparatively, it also had the second highest mileage of gravel roads. However, this appears to be in line with the findings in another aspect of this research: counties in Indiana with the highest unpaved road mileage also spent less per mile of road (1). Figures 2 and 3 present the relative expenditure by cost category and the percentages of total maintenance and repair expenditure for all counties. The average annual costs for the 4 years, 1980-1983, were used in the plots. The specific cost items plotted are as follows:

1. Administrative and supervision overhead
2. Garage and mechanical overhead
3. Equipment maintenance and rental costs
4. Stone and gravel materials
5. Bituminous materials and mixes
6. Other materials (culverts or tiles, bridge metal, road signs, etc.)
7. Labor (truck drivers, equipment operators, etc.)
8. Contractual services

On the average, over the period 1980-1983, the three highest expenditure categories for all five counties were, in decreasing order, administrative and supervision overhead (item 1) followed by labor (item 7) and garage and mechanical overhead (item 2). Together they represent between 50 and 60 per-

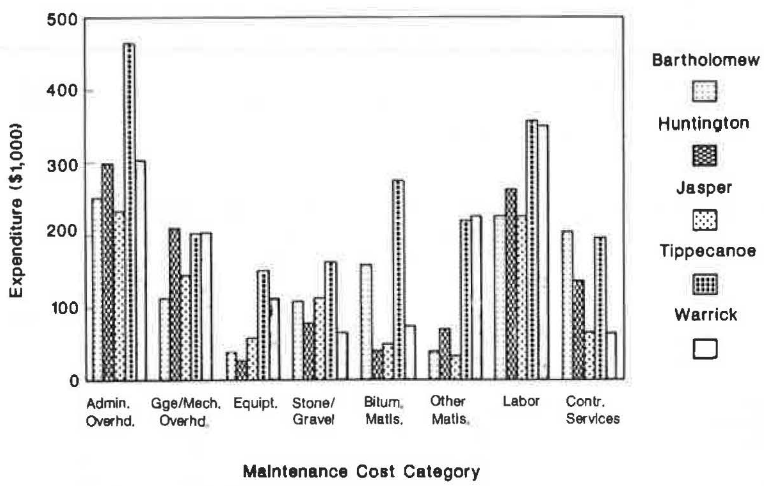


FIGURE 2 County highway maintenance expenditure (1980-1983 average by category).

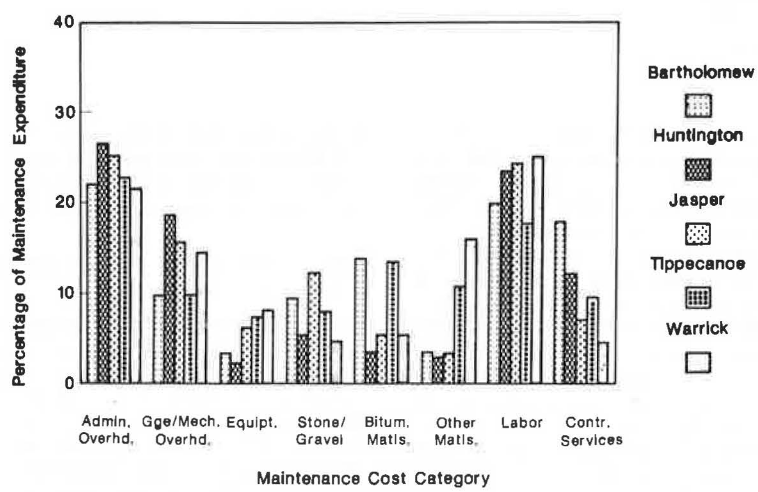


FIGURE 3 Percentage of county highway maintenance expenditure based on average 1980-1983 costs by category.

cent of the annual expenditure on maintenance and repair. Operationally, the control of these categories of expenditure will greatly determine the final costs of individual activities. Because they make up the larger portion of costs, they also affect the relative expenditure on the other items, mainly of materials used, which represents essentially the more variable component of maintenance costs. Average costs of all materials and supplies, if considered together, represents the highest individual cost category, about 26 percent for the five counties, ranging from 17 to 32 percent.

DETERMINING COST OF COUNTY ROAD MAINTENANCE

Data returns from four of the five counties for the selected unpaved road sections in the study provided some indication of relative costs of gravel road maintenance. Information on activity costs for all county roads, paved or unpaved, from Huntington County in Indiana and Mason County in Washington State, both of which implemented some maintenance management procedures, are also presented.

Gravel Road Maintenance Costs

The major activities reported on gravel roads are grading and spot regraveling. Other minor activities included brush cutting or vegetation control, side ditching, and snow plowing. Some of these costs were reported by Huntington and Jasper counties. In the former county, data were obtained from its management information system. Costs for activities common to both gravel and paved roads, such as snow plowing, were usually not separated by surface type.

Bartholomew County

Annual unit costs of gravel road maintenance comprising mainly grading and gravel addition for the three study roads in Bartholomew County averaged $259/mi and ranged from $155/mi to $480/mi. However, the unit grader cost of $11.14/hr provided by the county represented labor and grader running costs for fuel and oil and not the total cost for operating the grader including depreciation and other repair costs. The maintenance cost details are presented in Table 2.

Huntington County

The 1983 data for Huntington County gave a unit gravel road grading cost of $320/mi. With additional data provided by the county engineer, a 3-year (1981-1983) average unit cost for grading of $365/mi was obtained. There was more blading of unpaved roads in 1981 and 1982; hence the 3-year average unit cost was higher than the unit cost in 1983. The aggregate patching cost for 1983, however, was $93/mi. On the average in 1983, just for the two major activities of grading and aggregate patching, the unit maintenance cost was estimated at $409/mi of aggregate road. In Table 3, additional cost calculations were made in consultation with the county engineer for those activities such as vegetation control, sanding and snow plowing, drainage structures, and sign repair that were undertaken irrespective of road type. The percentages of the total expenditure on each activity spent on gravel roads are also shown in Table 3. Including the additional items, annual unit gravel road maintenance cost was $750/mi.

Jasper County

The annual unit costs in 1983 for the roads studied in Jasper County ranged from $540/mi to $6,567/mi, giving an average gravel road maintenance cost of about $1,360/mi. The rather high estimated unit cost of $6,567/mi was for a road that serves primarily a private large-scale mint farming project, which gen-

TABLE 2 Gravel Road Maintenance Costs on Study Roads: Bartholomew County

			Grading Activities			Stone Added			
Road	Length (mi)	ADT	Frequency[a] (no. x days)	Time (hr)	Cost ($)	Tons	Cost ($)	Annual Total ($)	Annual Cost ($/mi)
1	6.5	38	6 x 55	29	323.06[b]	169	1,049.49	1,372.55	211.
2	1.75	55	5 x 69	7	77.98	31	192.5	270.49	155.
3	3.25	52	6 x 58	10	111.40	156	968.76	1,080.16	480.

[a]Frequency is shown as number of gradings recorded during the year and average number of days between gradings.
[b]County unit grader costs at $11.14/hr (labor + grader).

TABLE 3 Gravel Road Maintenance Costs on Study Roads: Huntington County

	Expenditure ($)						
Activity	Labor	Materials	Equipment	Contractual	Overhead	Total	Percent of Total
Grading	18,680	5,765	42,432	-	15,605	82,482	42.3
Aggregate patching	2,391	14,798	5,151	-	1,890	24,230	12.4
Sanding and snow plowing[a] (15, 25)	3,459	10,010	6,434	-	2,872	22,835	11.7
Vegetation control[a] (33)	7,363	-	7,863	-	8,756	23,982	12.3
Drainage structures[a] (33)	4,851	10,481	7,645	-	4,080	27,057	13.9
Side ditching[a] (25)	420	66	922	-	17	1,425	0.1
Sign repair[a] (25)	5,384	2,633	1,422	-	3,703	13,142	6.7
Total						195,154	

[a]These activities were costed irrespective of surface type. Amounts shown in parentheses are proportions assigned to unpaved roads. Sanding, including addition of salt, was estimated as 15 percent compared with 25 percent for snow plowing on gravel roads.

TABLE 4 Gravel Road Maintenance Costs on Study Roads: Jasper County

Road	Length (mi)	ADT	Grading Activities Frequency[a] (no. x days)	Time (hr)	Cost ($)	Stone Added Tons	Cost ($)	Brush Cutting ($)	Side Ditching ($)	Snow Plowing ($)	Annual Total ($)	Annual Cost ($/mi)
1	2.5	118	38 x 7.5	330	14,286	380.75	1,092.1	840	-	200	16,418.1	6,567
2	4.5	44	24 x 11.5	72	3,389.2	533.05	1,471.6	840	-	200	5,900.8	1,311
3	2	60	19 x 13.4	21.5	890.7	133.9	435.22	280	-	70	1,675.9	838
4	2.5	149	37 x 7.4	53.5	2,024.6	457.2	1,459.2	560	-	105	4,148.8	1,659
5	4.0	14	19 x 13.4	31.5	1,259.8	181.2	555.93	280	-	70	2,165.5	541
6	2.0	28	20 x 8.0	15	588.5	122.7	398.8	560	-	105	1,652.3	826
7	4.0	28	11 x 12.6	11	655.04	31.95	168.5	70	11.36	140	1,044.93	261
8	4.25	34	17 x 12.6	33.5	1,069.7	-	-	886.5	-	70	2,026.2	477

[a]Frequency is shown as number of gradings recorded during the year and average number of days between gradings.

erates high average daily traffic volume mainly of farm vehicles. Maintenance expenditure on that road is borne largely by the private company engaged in the project. Average daily traffic on that road was 118. Excluding that road, average gravel road maintenance cost for 1983 was about $800/mi. The details of costs for Jasper County are presented in Table 4.

Grading Costs for Tippecanoe County

Consistent monitoring of costs on the study sections was not undertaken by Tippecanoe County. However, information was collected from the county to calculate the cost of grading on the sections under study. The daily work report completed by grader operators and the information on equipment maintenance and fuel use were the main source of data. Grading costs are presented in Table 5. Grading costs in 1983 varied from $203/mi to $346/mi for gravel roads in the seven districts of the county. The costs, were, as expected, a function of the frequency of maintenance and the equipment used in the different districts. Equipment operating costs include annual depreciation costs based on a straight-line depreciation over 10 years of estimated life of the grading equipment.

Effect of Different Management Practices

Three management practices that were examined included grading policies, equipment used, and spot regraveling. The implementation of such policies in practice may stem from budget limitations; however, detailed examination of their effects is seldom undertaken by the average county.

Grading Policies

Differences in cost between counties are a direct result of policy differences, which also affected the grading frequencies applied. The lower unit costs for Bartholomew County are due to less frequent grading. Grading frequencies ranged from five times at an average interval of every 69 days (10 weeks, approximately) to six times at an average interval of every 55 days (less than 8 weeks). The range of average daily traffic (ADT) on the roads, however, does not provide a basis for determining the relationship to traffic volumes using the roads.

For Huntington County, the grading frequency ranged from every 14 days to every 54 days on the study sections. Figure 4 is a plot of grading frequency versus traffic volume. A logical pattern is seen in which roads with higher traffic volumes are graded more frequently than roads with lower traffic volumes. A simple linear regression of grading frequency versus traffic volume is shown in Equation 1 (r^2 is 90.4 percent for six data points with an adjusted r^2 of 88 percent):

$$Y = 72.989 - 0.656T \quad (1)$$

where Y is the number of days between gradings and T is the ADT. On the basis of Equation 1, if a maximum frequency of grading of every 7 days is adopted, traffic volume should be at least 100 vehicles per day. A minimum frequency of about once every 73 days is expected for roads carrying little or no traffic.

Figure 5 is a similar plot of grading frequency and ADT for Jasper County. A linear trend is again evident, showing a decrease in days between grading with increasing ADT. The range of grading frequency for this county is, however, smaller. Grading frequencies ranged from approximately every 7 days or weekly to about every 14 days (13.4 days). The linear regression equation obtained for the plot is as follows:

$$Y = 13.119 - 0.039T \quad (2)$$

where the variables are the same as those in Equation 1.

TABLE 5 Unit Costs of Grading Operations: Tippecanoe County

District	Gravel Road (mi)	Year of Purchase	Grader Cost ($)	Annual Depreciation Cost ($)	Total Repair Cost[a] ($)	Number of Bladings a Year[b]	Grader Operating Cost ($/mi)	Unit Grader Cost ($/mi)	Annual Grading Cost ($/mi)
1	56	1973	24,989	2,499	7,223	14	9.6	14.5	203
2	49	1974	28,263	2,826	7,731	18	9.3	14.2	256
3	58	1972	27,227	2,723	6,850	16	7.8	12.7	203
4	42	1973	24,989	2,499	10,053	19	11.9	16.8	319
5	43	1982	111,176	11,118	4,035	14	18.8	23.7	332
6	48	1980	80,615	8,062	8,534	18	14.3	19.2	346
7	47	1983	55,533	5,553	5,275	15	12.1	17.	255

[a]Total grader repair cost for all uses (assumed grading is 75 percent).
[b]Equivalent number of complete bladings of all gravel roads in district. Actual blading may differ from road to road.
[c]Includes operator wages estimated at $6.59/hr. Assumes 1-mi grading takes 0.75 hr of operator's time.

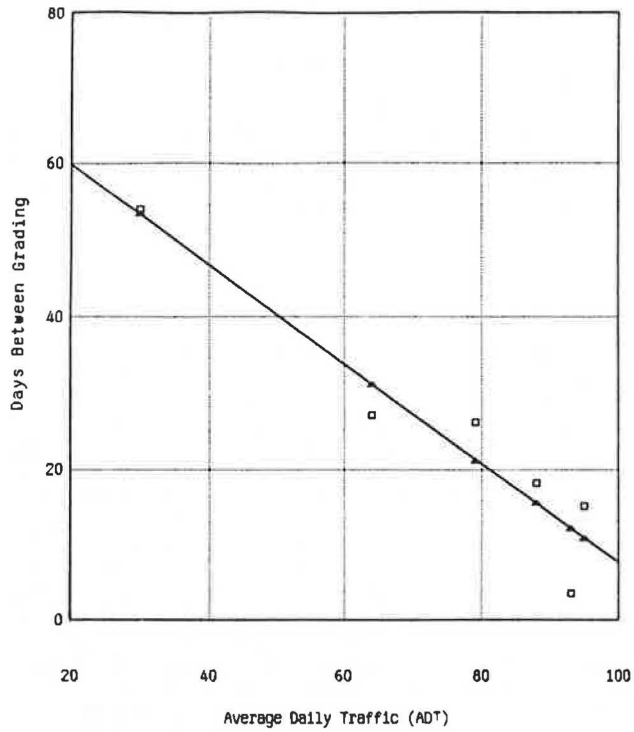


FIGURE 4 Frequency of grading versus traffic volume: Huntington County.

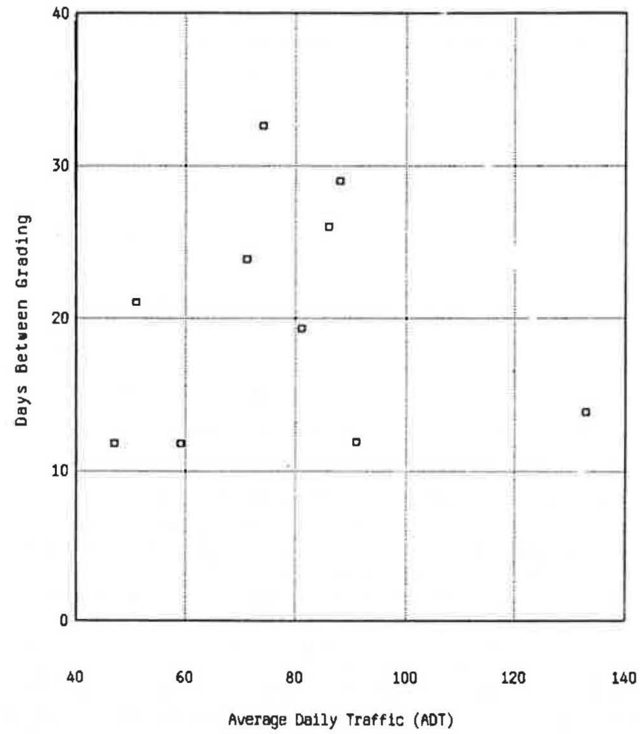


FIGURE 6 Frequency of grading versus traffic volume: Tippecanoe County.

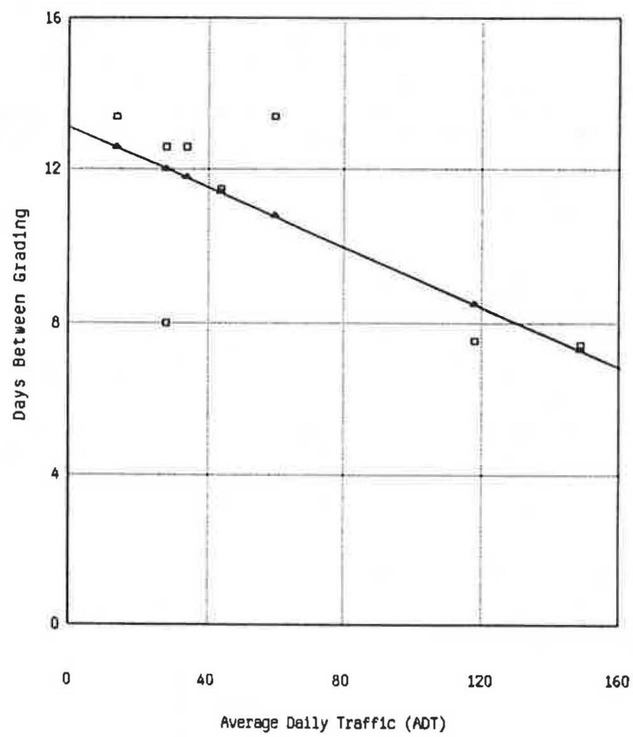


FIGURE 5 Frequency of grading versus traffic volume: Jasper County.

The r^2 is 0.49 and the adjusted r^2 is 0.407. One or two outliers are evident in the plot. From Equation 2, a road should carry a traffic volume of at least 157 vehicles a day for it to receive the maximum frequency of grading every 7 days. The higher frequencies in Jasper County appear to be the result of generally poor subgrades in several parts of the county. High water tables and prevalence of "muck" soils create a need for frequent addition of gravel on some roads. The county highway department also grades the roads frequently in order to re-spread surface gravel.

Grading frequency plots for the gravel road sections in Tippecanoe County are presented in Figure 6. Regression analysis resulted in a very low r^2-value of less than 1 percent for the relationship between grading frequency and ADT. The grading policy in the county is that every road be graded at least once a week. Grader operators in the seven grading districts attempt to meet the schedule. However, graders use personal judgment and knowledge of the area, and some differentiation is made in the frequency of maintaining individual unpaved roads in their care. No definite equation was determined for this plot because it appears that traffic volume had little influence on the frequency of grading in that county.

The various grading policies identified have had an influence on the local unit costs of unpaved road maintenance in each county. In general, traffic volume appears to be an important consideration in determining grading policies for counties. Applying appropriate differences in traffic volumes could ensure that money is spent where the need is greatest. Unpaved road distresses such as roughness, rut depth, potholes, and corrugations have been found to reduce with grading or blading (1,4-7). However, as a practice, few Indiana counties measure road distresses or monitor road condition routinely as part of maintenance management (8).

The equations for Huntington and Jasper counties provide first estimates of possible relationships. Because grading frequency in Jasper County is influenced greatly by the poor existing subgrade material, the curve for Huntington County was used to develop possible grading frequency for various ranges of ADT. Riverson (1) also found in a separate

TABLE 6 Suggested Traffic Volume Groups, Grading Frequency, and Other Considerations for Unpaved-Road Maintenance

Traffic Volume (vehicles/day)	Grading Frequency: Days Between	Grading Frequency: Times/Yr	Annual Cost ($/mi)	Remarks
<50	40-60	7-5	150-108	Roads with steep grades; frequent corrugation may require minimum days
50-100	21-40	13-7	280-150	Same as above, including locations with frequent driveways
100-200	7-20	40-13	860-280	Same as above; some dust control may be required
>200	7 or less	>40	>860	Same as above; stabilization or paving

study of unpaved road roughness that Huntington County roads had on average the lowest roughness number values compared with those of the other counties. Although the values were also influenced by the material gradation characteristics in the various counties, the curve for Huntington County was used as a reasonable basis, initially, for establishing grading frequencies for Indiana counties. Table 6 shows the grading frequencies for various ADT ranges and the variation in grading as a result of terrain and other condition factors such as the rate of corrugation development.

Equipment Used

Another aspect of importance for maintenance management is the type of equipment frequently used for grading unpaved roads. Huntington County, for example, uses mainly trucks with mounted underbody blades for grading gravel roads. Routine blading usually required to maintain a gravel road in good condition is adequately provided by the trucks. In addition, gravel or stone required for spot regraveling can be transported to the road site on the truck, which enables material to be dumped and spread by the same unit of equipment. Annual unit cost of equipment maintenance using 1983 equipment cost data is about $163 per year per mile of road maintained (261 mi of unpaved roads). Unit cost in 1983 of using the truck-mounted blade maintainer is $13.42/hr compared with $28.69/hr for using the motor grader.

Thus, the choice of equipment affects unit costs of grading and hence gravel road maintenance. This choice is usually the prerogative of the particular county engineer or supervisor. Actual road performance such as roughness after blading or relative deterioration of roads that have been maintained by any piece of equipment is also an important consideration. Nevertheless, it is expected that continued use of any particular piece of equipment by any county would be an indication of the satisfactory road performance experienced previously in any particular county as well as better production rates.

Spot Regraveling

The amount of spot or total regraveling undertaken by any county could increase or lower the unit maintenance costs for gravel roads. A minimal amount of spot regraveling was undertaken by Huntington County, for example, in 1983. It was estimated by the county that to bring existing roads to standard, the average gravel road in the county requires a coating of 1,000 yd^3 of gravel (1,500 tons), about 3 in. of gravel over an average width of 20 ft. The unit cost is estimated as $4,255/mi of gravel road. Such a program can, however, only be sustained on a periodic basis. The 260 mi of unpaved roads in Huntington County will require an annual budget of over $1.1 million for regraveling alone. As expected, because of the poor subgrade conditions prevailing in Jasper County, spot graveling costs there, shown in Table 4, are much higher than those estimated for Huntington County.

Total regraveling should thus be considered like paving of the gravel surface, as a periodic maintenance activity. In the absence of adequate funds, spot regraveling could continue to be undertaken as a routine maintenance activity. With a properly developed unpaved surface crust, as was evident on several roads in Huntington County, a minimal amount of spot regraveling would usually be necessary. The unit cost of gravel road maintenance for Mason County is about $1,000/mi for grading and regraveling. However, this results from undertaking regraveling every year on selected roads, which in turn has led to a reduction in grading. Gravel roads are graded at a frequency of six times per year (2).

Costing Other Activities

Huntington County in Indiana and Mason County in Washington State both provided information to determine the relative costs of other maintenance activities at the county level. On the basis of their different reporting procedures, the top 10 activities in 1983 on which each county spent the most money are indicated in Table 7. Although the actual amounts spent on different activities differ between

TABLE 7 Comparison of the Top 10 Expense Items in Huntington and Mason Counties

	Huntington County, Indiana			Mason County, Washington		
Rank	Activity	Amount ($000s)	Percent of Total	Activity	Amount ($000s)	Percent of Total
1	Ice control and snow removal	142.4	17.1	Seal coating	418.5	26.2
2	Bituminous patching	133.8	16.0	Bituminous patching	256.7	16.0
3	Administrative[a]	114.1	13.7	Ditching and minor maintenance	208.8	13.1
4	Sealing	98.2	11.7	Traffic control and striping	156.	9.8
5	Grading	82.5	9.8	Administrative	114.4	7.2
6	Storm drainage structures	82.4	9.8	Vegetation control	104.4	6.5
7	Vegetation control	69.2	8.2	Storm drainage structures	92.08	5.8
8	Traffic sign repair	52.6	6.3	Aggregate patching	80.4	5.0
9	Aggregate patching	24.2	2.9	Miscellaneous (including road cleaning)	66.4	4.2
10	Shoulder maintenance	21.3	2.5	Grading	63.2	4.0
Total		820.7	98.0		1,560.9	97.8

[a]Includes office and field engineering, training, county survey, garage, grounds, parking area, and vehicle maintenance.

TABLE 8 Comparison of Maintenance Expenditure in Two Counties by Roadway and Environmentally Related Activities

	Percent of Total Expenditure	
Item	Huntington County	Mason County
Pavement and shoulder		
Paved	30.2	42.3
Unpaved	12.7	9.0
Winter maintenance	17.1	2.3
Roadside drainage	0.8	13.1
Vegetation control	8.2	6.5
Sewer and storm drainage	9.8	5.8
Traffic control and services	6.3	9.8
Total	84.3	88.8

the two counties, it is clear that ice control and snow removal constituted a major cost item in Huntington County and not in Mason County. The differences in weather effects on expenditure in the two counties are highlighted. Bituminous patching and seal coating were among the top four activities with the highest expenditure levels in both counties. Administrative costs, however, may not have been defined in the same way in the two counties. Huntington County, for example, accounts for administrative overhead in each activity.

An alternative breakdown of maintenance and repair expenditure is presented in Table 8. The cost items covered include pavement and shoulder maintenance for paved and unpaved roads. Winter maintenance, vegetation control, roadside drainage, sewer and storm drainage, structure maintenance, and traffic control accounted for over 80 percent of the 1983 expenditure on maintenance and repair in both counties. Pavement and shoulder maintenance activities on paved roads were the highest expenditure items in both counties compared with similar expenditure on unpaved roads.

EFFECT OF ROAD CONDITION

Ultimately, the management effects mentioned earlier would affect the condition of gravel roads. However, in most counties in Indiana, no formal procedures based on quantitative measurements are used to determine the condition of roads in the network (8). Road condition is determined subjectively from inspections conducted by grader operators, foremen, supervisors, or engineers. Considerable weight is also given to complaints by local residents in determining the need for maintenance or major repairs. However, unless care is taken to ensure some rationality in planning maintenance activities, unnecessarily high costs of maintenance may result. For unpaved roads, typical distresses include rutting, corrugation, potholes, dust generation, and loss of surface gravel or stone. The results of other parts of the research by HERPICC concerning unpaved road condition parameters such as roughness, rut depth, material characteristics, and so on are presented by Riverson (1). For management purposes, it is important to note that the condition of unpaved roads changes more rapidly than that of paved roads and it is influenced by factors like material characteristics, traffic volume, weather, and drainage, including surface crown or cross slope, as well as the frequency and type of maintenance. Hence, a "one-time" condition measurement may not be sufficient, and any periodic monitoring of unpaved road condition should be related to the strategies adopted for maintenance as well as the desired level of service and the budget limitations.

CONCLUSIONS

The attempt to isolate gravel road maintenance costs from county highway accounts has thrown some light on specific maintenance management needs of most counties in Indiana. Continuing the present system of accounting will only provide annual summaries of labor, equipment, and materials costs for various budget classifications. Proposals have been made for a maintenance management system to fulfill these needs (1). The system adopts current accounting procedures with some modifications to improve county maintenance management.

Maintenance management of unpaved roads, even if based simply on annual accounts, will be enhanced if adequate consideration is given to traffic volume, road condition, and performance in its planning and implementation. Proper allocation and control of county maintenance funds for specific activities will ensure the best use of funds for the revenue received. Each county will need to take its peculiarities into consideration. However, proper reporting by activity will greatly aid cost accounting and improve maintenance management, which depends on good estimates of maintenance costs of roads of all surface types. As maintenance activity costing is improved in the counties, adequate life-cycle costing procedures can be implemented as the ability to monitor road performance is also enhanced. For some counties, however, the procedure may have to be implemented on a step-by-step basis.

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Publication of this paper sponsored by Committee on Low Volume Roads.

Commentary Driving Applied to Safety Evaluation of Low-Volume Rural Roads: Training and Use

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ABSTRACT

The procedure of commentary driving is a simple field technique used in the safety evaluation of roadways. The procedure and its uses are described. A set of checksheets based on the concepts of decision sight distance are introduced. These checksheets should be a valuable tool in the safety evaluation of sites that are found to be information deficient by commentary driving. Two teaching methods for the technique of commentary driving were studied. The study was conducted to determine the amount of time required to teach someone the technique of commentary driving and then to determine the effectiveness of the two alternative teaching methods. The two teaching methods were to have the students make commentaries (a) while viewing a videotape of a predetermined route (VIDEO) or (b) while driving a predetermined route (DRIVE). It was concluded that the commentary driving technique and the use of the information-deficient location checksheets can be taught to county personnel in a 1- to 2-day workshop. It was also concluded that the VIDEO or DRIVE methods work about equally well in teaching the use of the commentary driving technique.

Every day, county personnel from states across the nation are faced with the problem of signing and maintaining the low-volume roads (roads with less than 400 vehicles per day) within their county. Many of the counties have their own methods for the inventory and inspection of their signs and markings. However, few counties have a simple method for the evaluation of information-deficient locations on their road systems.

Whereas an inventory is simply a matter of the number of signs and their respective locations, the inspection is concerned with the physical condition and appearance of the sign. An evaluation determines whether the current signs are correct, needed at all, or missing (i.e., an information-deficient location).

One can readily see that there is a definite need for some type of simple procedure by which the counties can evaluate the road systems for locations that are information deficient or potentially hazardous. Commentary driving is one such procedure. Commentary driving is a technique in which, at the beginning of a section of road to be evaluated, the driver states his expectancies of the road and as he proceeds along the road he comments on locations or conditions that violate his expectancy.

This study was conducted to determine the amount of time required to teach someone the technique of commentary driving and then to determine the effectiveness of two alternative teaching methods. The two teaching methods were to have the students make commentaries (a) while viewing a videotape of a predetermined route (VIDEO) or (b) while driving a predetermined route (DRIVE).

This study dealt only with the aspects of teaching the technique to county personnel in Kansas and to Kansas State University students who have the same background as the county personnel. The term "background" refers to the students' knowledge of the proper rules, regulations, general signing, and geometrical layout of county and township road systems. Even though, as described later in the Summary, commentary driving can be used for various other situations, this study was concerned only with its application on low-volume rural (LVR) roads. The reason for applying this limitation to the study was to gather information on teaching commentary driving to county personnel and to later add a section on commentary driving to the Handbook of Traffic Control Practices for Low Volume Rural Roads (LVR Handbook) (1).

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BACKGROUND

Commentary Driving Procedure

The information that a driver receives from the roadway must be correct, pertinent, concise, and presented in such a way that it is readily usable by the driver. In many cases, however, this information is not consistent with what he expects to, or should, receive. If the driver's expectancy of the roadway environment is violated, a potentially hazardous situation exists. The procedure of commentary driving was developed by R.S. Hostetter et al. (2). Generally stated, commentary driving is a simple field technique that requires no special equipment and from which information is gathered concerning the roadway environment to help eliminate all information-deficient locations. Information-deficient locations are specific locations on the roadway where the information, received by the driver from the roadway, is not sufficient to give the driver the needed information to safely traverse the roadway.

In the planning for the evaluation of a county's road system it is recommended that the roads be divided into several routes. Each route is from 3 to 15 mi long. Every road that the county is responsible for is placed on only one of the routes. The routes are listed on a priority basis so that the roads deemed to be most hazardous are evaluated first (2).

After the routes have been established and listed on a priority basis, either a team of two or an in-

dividual with a tape recorder (see section on personnel requirements) drives the roads, making sure to drive each route in both directions and, if necessary, driving some routes at night. As the team or individual drives the route, the driver will comment verbally on what information is needed versus what information is received from the various situations on the route. The driver's commentaries will usually be stored on a cassette tape so that later reference can be made to them if necessary.

The driver's commentary is divided into two parts. Within the first half-mile, the driver makes statements concerning the general nature of the roadway environment. Included in this group of general comments are the classification of the road, the surface quality, existing positive guidance (3), predicted safe driving speeds, availability of warning signs, and other general expectancies of the road. The driver's comments then focus more specifically on the events that he encounters as he moves farther along the roadway, commenting on the situations as they arise (2). The comments regard

1. The driver's expectancy concerning direction (i.e., straight or curves to the left or right), vertical curves, sharpness and safe speed of curves, oncoming traffic, culvert and bridge width and alignment, right-of-way controls at intersections, etc.;
2. What actions may be necessary regarding speed changes, lateral movement, turns, etc.; and
3. Any uncertainty related to any of the two foregoing items.

During the running commentary, the driver may believe it necessary to restate his initial comments. This is especially true on long straight tangents where there is little need for specific comments. It is believed that during the initial statement and restatement of expectancies, obvious information-deficient locations will be identified as a result of the commentary.

Verbal comments are suggested because this forces the driver to state what he expects from the road environment ahead and thus makes him more sensitive to any inconsistencies that may confront him. It is also suggested that the driver maintain a speed as close to the posted speed limit as is comfortably possible. If no speed limit is posted, the driver should drive the road as he believes a reasonably prudent driver would.

As stated earlier, the driver's comments should be stored on an audiotape cassette in a cassette tape recorder so the driver can replay the tape in the event that he must further investigate a site. For this reason, in addition to identifying the route, it is necessary that the driver record the mileage at the beginning of and also at the specific points of interest along the route. Although some drivers may be uneasy with the tape recorder at first, with a few hours of practice they will become relaxed and proficient in its use (2). This point is discussed in more detail in the Conclusions of this paper.

The last step in the procedure is to conduct more detailed surveys of the sites that have been identified as information deficient during the commentary driving portion of the task. This job is made easier by using the checksheets developed by Hostetter et al. (2,Vol.2). Figures 1 and 2 are 2 of the 10 checksheets. The other eight checksheets are for horizontal curves, tangential intersections, intersections that require a turn, railroad-highway grade crossings, uncontrolled Y-intersections, low water stream crossings, height and weight restrictions, and other situations. Table 1 is mentioned in Figures 1 and 2 and in the other eight checksheets. All of the checksheets were developed to aid the crew when they revisit the information-deficient locations to conduct further study of the site. The checksheets are self-explanatory for experienced highway personnel. The locations in question are those in which there was no obvious solution on the initial drive-through of the route. These locations can then be listed in priority order and later improved as the county acquires the funds for this purpose.

Survey Frequency

It is important to note that this type of survey probably need not be done at any set interval of time. In fact, once the initial survey has been finished, the only reason for redoing it would be for substantial changes in the nature of the roadway environment. This in no way means that once the survey is completed, the responsible county engineer is no longer concerned with providing the needed information to the motoring public. He must continue the routine inspection of all his roadways (2). Note that surveys during seasons of high vegetation growth can be very helpful in determining problems of obstruction of signs by weeds or trees.

Personnel Requirements

When a team of two is used, the driver does the commentary and the passenger acts as a guide or navigator. The passenger can also be a recorder if the tape recorder is not used. The main objective in using a team of two is to free the driver from concerns about staying on the route so he may concentrate on evaluating it.

Although there are no rigid requirements for selecting a driver, it is recommended that he be knowledgeable in the application of traffic control devices, particularly signs. He should also be familiar with the Manual of Uniform Traffic Control Devices (MUTCD) (4) and in particular the LVR Handbook (1). The preferred driver would be unfamiliar with the road system to be driven (i.e., an engineer borrowed from the neighboring county). The driver should be neither too cautious (overstates deficiencies) nor too aggressive (has high tolerance for deficiencies) (2).

Hostetter (2) suggests that the driver be a traffic engineer and the recorder be a technician. From their experience in Kansas, the authors believe that the driver (commentator) should be a county engineer or road supervisor or some other of the technical personnel experienced in the use of the LVR Handbook. Although it would be helpful if the passenger (navigator or recorder) were a technician, the authors do not believe this to be necessary. On the other hand, if the driver is a county engineer or road supervisor from, say, an adjacent county, the passenger should be a technically qualified person from the county in which the roads are located.

EXPERIMENTS

In this section the two experiments are described that were designed to answer the question, Can a student show that he has learned the technique of commentary driving by watching a videotape of a route in a classroom and commenting on what he sees, or does the student need to do the commentary from an automobile out on the road?

The commentaries were about 40 to 50 min long and

the routes were 20 to 25 mi long for both experiments. All routes include examples of the various types (A, B, and C from LVR Handbook definitions) of LVR roads. Included in this section is a brief explanation of each experiment followed by a section on the statistical results.

Experiment 1

Procedure and Experimental Design

The 21 subjects for this experiment were all members of the fall semester 1984 Route Location and Design

ROUTE ID ____________________ INTERSECTING ROUTE ____________________

APPROACH DIRECTION N S E W (circle)

DATE __________ TIME __________ AM PM INSPECTOR______________

SPEED LIMIT ______MPH ESTIMATED TYPICAL APPROACH SPEED ________MPH

DECISION SIGHT DISTANCE (circle one set)

SPEED (max of above)	30	35	40	45	50	55	60
DSD (feet)	220	275	345	420	500	585	680

(1) Is the intersection clearly visible from decision sight distance?
_____Yes _____No

(2) Is the stop sign clearly visible from decision sight distance?
_____Yes _____No

If no, go to (4)

(3) From decision sight distance, can you determine that the stop sign applies to you? _____Yes _____No

If yes, go to (6)

(4) Is there a STOP AHEAD warning sign present? _____Yes _____No

If no, go to (6)

(5a) Is the STOP AHEAD warning sign clearly visible on the approach?
_____Yes _____No

(5b) Is the STOP AHEAD warning sign designed according to the specifications in the MUTCD? _____Yes _____No

(5c) Is the STOP AHEAD warning sign properly located? (i.e., neither too far upstream such that you would "forget" it or too close to the intersection such that you still would not have sufficient time to stop) (Check Table of Placement Distances for Advance Warning Signs)* ____Yes ____No

(6) Do other informational sources (i.e., roadway surface edges, terrain cuts, brush/tree line, shoulder edges, centerlines, etc.) provide information suggesting either 1) that the situation ahead is <u>not</u> a stop-controlled intersection, 2) that stop sign does not apply to your approach, or 3) that the stop controlled intersection is located further down stream than it actually is?
_____Yes _____No

If yes, then identify those sources and describe how they provide confusing, conflicting or misleading information:____________________

(7) Is the presently available information sufficient for you to recognize the stop-controlled intersection at a distance such that you can stop safely? _____Yes _____No

(8) Would the presently available information be sufficient for you to recognize that a stop-controlled intersection is located downstream:

- o during nighttime conditions? _____Yes _____No
- o when the roadside vegetation is at its densest growth? _____Yes _____No

* See Table 1

FIGURE 1 Information-deficiency evaluation checksheet for a stop-controlled intersection.

SUGGESTED TREATMENTS

____ Install STOP AHEAD warning sign
____ Improve visibility of STOP AHEAD warning sign
____ Relocate STOP AHEAD warning sign
-Move closer to intersection by ____feet
-Move back from intersection by ____feet
____ Replace non-standard warning sign with standard STOP AHEAD warning sign
____ Improve sight distance to intersection
____ Improve visibility of stop sign
____ Install stop lines
____ Improve markings at intersection
____ Improve signing at intersection
____ Correct for confusing, conflicting or misleading information:

____
____
____

____ Implement other treatment:

____
____
____

FIGURE 1 continued.

ROUTE ID____________________ LOCATION: ______________MILES FROM REFERENCE POINT____________________

APPROACH DIRECTION N S E W (circle)

DATE____________ TIME____________ AM PM INSPECTOR____________________

SPEED LIMIT__________MPH ESTIMATED TYPICAL APPROACH SPEED______________MPH

DECISION SIGHT DISTANCE (circle one set)

SPEED (max of above)	30	35	40	45	50	55	60
DSD (feet)	230	290	355	430	510	590	680

(1) Is the bridge clearly visible from decision sight distance?
_____Yes _____No

If no, go to (3)

(2) From decision sight distance, can you perceive the reduced roadway width at the bridge? _____Yes _____No

If yes, go to (5)

(3) Is there a NARROW BRIDGE or ONE-LANE BRIDGE warning sign present?
_____Yes _____No

If no, go to (5)

(4a) Is the warning sign accurate? (i.e., the ONE-LANE BRIDGE is applicable to bridges with usable roadway widths less than 16 feet or 18 feet if a significant number of wide vehicles cross the bridge or if the approach alignment is winding)
_____Yes _____No

(4b) Is the warning sign clearly visible on the approach?
_____Yes _____No

(4c) Is the warning sign properly designed according to the specifications in the MUTCD? _____Yes _____No

(4d) Is the warning sign properly located: (i.e., neither too far upstream such that you would "forget" it or too close to the bridge such that you still would not have sufficient time to select a safe speed and decelerate to it) (Check Table of Placement Distance for Advance Warning Signs)* _____Yes _____No

(4e) Is there a supplemental speed advisory plate attached to the warning sign? _____Yes _____No

* See Table 1

FIGURE 2 Information-deficiency evaluation checksheet for a narrow or one-lane bridge.

(5) Do other informational sources (i.e., hazard panels, guardrails, edgelines, roadway edges, bridge abutments, etc.) provide information suggesting 1) that the situation ahead is not a narrow/one-lane bridge, 2) that usable roadway width across the bridge is wider than it actually is, or 3) that a narrow/one-lane bridge is located further downstream?
_____Yes _____No

If yes, then identify those sources and describe how they provide confusing, conflicting or misleading information:____________________

____

____

____

____

(6) Is the sight distance to opposing vehicles sufficient for you to make a safe decision on whether you can safely cross the bridge and to safely execute the selected maneuver? _____Yes _____No

(7) Is the presently available information sufficient for you to recognize the narrow/one-lane bridge at a distance such that you can decelerate safely to a safe and comfortable crossing speed?
_____Yes _____No

(8) Would the presently available information be sufficient for you to recognize that a narrow/one-lane bridge is downstream:

o during nighttime condition? _____Yes _____No
o When the roadside vegetation is at its densest growth? _____Yes _____No

SUGGESTED TREATMENTS

_____ Install NARROW BRIDGE warning sign
_____ Install ONE-LANE BRIDGE warning sign
_____ Improve visibility of advance warning sign
_____ Relocate advance warning sign
-Move closer to bridge by _____feet
-Move back from bridge by _____feet
_____ Replace non-standard warning sign with standard warning sign
_____ Install supplemental speed advisory plate;
suggested speed is _____MPH
_____ Install other advance warning signs, i.e.,
_____ Curve warning
_____ Intersection warning
_____ Low overhead clearance
_____ Other (specify) ____________________
____
_____ Improve pavement markings at bridge (i.e., tapered approach treatment)
_____ Install hazard panels at bridge
_____ Improve visibility of bridge
_____ Correct for confusing, conflicting or misleading information:
____
____
____
_____ Implement other treatment:
____
____
____

FIGURE 2 continued.

class in the Civil Engineering Department at Kansas State University (KSU).

Before the subjects began the experiment, they attended several lectures and slide presentations in which they were given information on how to identify various types of problem locations. In addition, they were required to read the information and concepts presented in the LVR Handbook (1). Furthermore, they were exposed to the technique of commentary driving by way of prepared commentary driving tapes (videotaped segments of road with someone correctly doing commentary driving), and they were given handouts showing hypothetical examples of commentaries (2) (Figures 3 and 4).

The first group (pairs, driver and navigator) was assigned to go into the field and actually drive a designated route. While driving the route, the driver did commentary and identified the problem locations on an audiotape. The navigator simply made sure that the driver stayed on the designated route. Drivers were told that they would be graded on their ability to identify all of the problem locations on the route and to follow the recommended commentary driving procedure. They also were told that they would be penalized for reporting a location that was not actually a problem location. This was done to keep them from commenting that every little spot in the road was a problem location. The second group (individuals) was given the same assignment with the exception that they demonstrated their ability at

TABLE 1 A Guide for Advance Warning Sign Placement Distance[1] *(1,4)*

Posted or 85 percentile speed MPH	Condition A high judgment needed [3] (10 secs. PIEV)	General warning signs[3]					
		Condition B—Stop condition	Condition C—Deceleration condition to listed advisory speed—MPH (or desired speed at condition)				
		0	10	20	30	40	50
20	[5] 175	([4])	([4])	....	....	....	....
25	250	([4])	[2] 100	....	....	....	....
30	325	[5] 100	150	[5] 100	....	....	....
35	400	150	200	175	....	....	....
40	475	225	275	250	[5] 175	....	....
45	550	300	350	300	250	....	....
50	625	375	425	400	325	[5] 225	....
55	700	450	500	475	400	300	....
60	775	550	575	550	500	400	[5] 300

Typical Signs for the Listed Conditions in Table II-1; Condition A—Merge. Right Lane Ends, etc; Condition B—Cross Road, Stop Ahead, Signal Ahead, Ped-Xing, etc.; Condition C—Turn, Curve, Divided Road, Hill, Dip, etc.

1 Distances shown are for level roadways. Corrections should be made for grades. If 48-inch signs are used, the legibility distance may be increased to 200 feet. This would allow reducing the above distance by 75 feet.

2 In urban areas, a supplementary plate underneath the warning sign should be used specifying the distance to the condition if there is an in-between intersection which might confuse the motorist.

3 Distance provides for 3-second PIEV, 125 feet Sign Legibility Distance, Braking Distance for Condition B and Comfortable Braking Distance for condition C as indicated in *A Policy on Geometric Design of Rural Highways,* 1965, AASHTO, Figure VII-15B.

4 No suggested minimum distance provided. At these speeds, sign location depends on physical conditions at site.

5 Feet

identifying problem locations by looking at a prerecorded videotape of the same designated route. Both groups were given a tape and tape recorder for recording their comments. At the end of the experiment, both groups returned their tapes.

Measurement

The experimenter evaluated the subjects' tapes by comparing them with a key tape (the experimenter's evaluation of the routes). The subjects were graded according to (a) the number of actual problem locations that they were able to identify and (b) the number of locations that they identified as problem locations when in fact they were not. A score was calculated for each subject by totaling the number of correct observations made and subtracting the number of incorrect observations made. The scores then were averaged for the subjects within the groups and the variances were found. The averages and variances then were compared for the two groups. The explanation of how the experimenter compared the subjects' tapes with his is given elsewhere (5).

Results

The tapes produced by the students were evaluated and a score was determined for each. The score was determined as described in the previous section. Table 2 shows the scores arranged in descending order and separated into the two conditions, VIDEO (commentaries made while viewing a videotape) and DRIVE (commentaries made while driving a selected route). The subject numbers have been arbitrarily defined and do not suggest the order in which the route was driven. The highest possible score for this route was 366 according to the experimenter's evaluation of the route.

Averages and standard deviations were calculated for both conditions. The average score for the viewers of the videotape (VIDEO) was 221 (range 189 to 257), whereas the average score for the students driving (DRIVE) was 175 (range 150 to 206). The

"Now travelling on Rt. 101, Northbound. The road has a smooth surface with a 2-4 foot paved shoulder and open terrain. The road is generally straight with a few gentle curves and short crests with generally good sight distance. The road is marked with centerline and edgeline. I expect to be able to travel at 55 mph even though a speed limit is not posted. I am not concerned about on-comming traffic. If there are curves or other situations requiring a speed reduction, I expect to be warned through appropriate signing."

or

"Now travelling on Jones Bridge Road, Southbound. The road is paved but there are occasional breaks in the pavement. There is no shoulder or centerline and I am not certain as to my lane limits. The road is curvilinear with several crests and dips which limit the sight distance. Except for some locations my safe speed is about 50 mph. There will be several occasions where I will have to reduce my speed but I expect to receive curve warning signs with speed advisory only at those locations."

FIGURE 3 Two hypothetical examples to show how one might comment on initial expectancies *(2).*

Item	Possible Commentary
	Example A
Approach to Crest	"Crest curve ahead, view of road limited . . . tree Vertical Curve line indicates that road goes straight ahead . . . not concerned about on-coming traffic . . . wide enough pavement . . . can maintain cruising speed . . ."
On Vertical Curve Crest	"Confirmed" [continue with next section]
	or
	"Expectation violated . . . tree line went straight but road curved left . . . not sharp enough to cause any problem . . . no need for warning sign."
	[continue with next section]
	or
	"Expectation violated . . . tree line went straight but road turned left sharply . . . needed to reduce speed . . . should have had curve warning sign at least . . . possibly speed advisory . . . mark site for study"
	Example B
Approach to Horizontal Curve	"Curve left ahead . . . see curve warning sign, no speed advisory . . . should be able to take curve at cruising speed . . . looking out for opposing vehicles because of narrow width"
Point of Curvature	"Curve sharper than anticipated . . . speed reduction neccosary expecially if on-coming vehicles . . . mark site for speed advisory check"
	Example C
Approach to Narrow Bridge on Curve	"Curve right ahead . . . see curve warning sign . . . assume I can maintain speed . . ."
Closer to Curve/Bridge	"See bridge headwalls . . . narrower pavement . . . not certain if wide enough for two vehicles . . . need to slow down . . . can't see across bridge for opposing vehicles . . ."

FIGURE 4 Sample commentaries for specific situations *(2)*.

TABLE 2 Subject Scores by Subject

Condition and Subject	Score
VIDEO	
1	257
2	239
3	237
4	226
5	206
6	191
7	189
DRIVE	
8	206
9	203
10	175
11	168
12	168
13	159
14	150

standard deviation for the VIDEO condition was 26 as compared with 21 for the DRIVE condition.

The objective of the analysis of data by the F-test (6,pp.364-365) was to find out whether there was a significant difference between the two conditions of the experiment. Because the F-test assumes that the two samples are normally distributed, the two groups were checked for normality by using the Kolmogorov-Smirnov one-sample test (7,pp.47-52,251). The calculated D, the statistic for the Kolmogorov-Smirnov test, for the VIDEO condition was 0.16, whereas that for the DRIVE condition was 0.23. The critical D for both conditions ($N = 7$, $\alpha = 0.05$) was 0.49. Therefore, because both of the calculated values were less than the critical value, it was concluded that the sample could be assumed to be normally distributed.

Next, the F-test was run on the data set. The null hypothesis for this test was that the mean scores of the two conditions were equal ($H_0 : \mu_v = \mu_d$), where μ_v is the mean score of the VIDEO condition, and μ_d is the mean score of the DRIVE condition. The calculated F, the test statistic for the F-test, for this set of data was 12.64. The critical F for degrees of freedom $v_1 = 1$ and $v_2 = 12$ with an α-level, or probability of rejecting the null hypothesis when it is true, of 0.05 was 4.75. Because 12.64 is larger than 4.75, there is a significant difference between the two sample mean scores. In other words, the two sample sets probably do not come from the same distribution. Because the mean for the VIDEO condition was larger than that of the DRIVE condition, the subjects watching the videotape scored higher and performed better than those subjects driving the road.

Experiment 2

Procedure and Experimental Design

The second experiment was divided into two sections. The only difference between the two sections was the type of subjects used. The first section used 23 students from the spring semester 1985 Route Location and Design class at KSU. The second section enlisted the aid of 23 county-level highway employees (county personnel). Included in this group were county engineers, engineering technicians, road supervisors, bridge supervisors, signing foremen, and a Kansas Department of Transportation (KDOT) safety engineer. This section of the experiment was conducted as an experiment-workshop type of exercise. Two consecutive 6-hr days of instruction and experiment were used.

The subjects in each section were separated into two groups. The first group consisted of several pairs (driver and navigator), who were assigned to the DRIVE condition of the experiment. The second group consisted of the remaining individual subjects, who were assigned to the VIDEO condition.

Before the subjects began the experiment, they attended several lectures and slide presentations on how to identify various types of information-deficient locations. In addition, they were required to read the information and concepts presented in the LVR Handbook (1). Twenty-two of the 23 members of the county personnel had attended a 3-day workshop on the use of the LVR Handbook within the last 2 years. Furthermore, they were given instruction on the technique of commentary driving by way of lectures and prepared commentary driving tapes (videotaped segments of road with someone correctly doing commentary driving) along with handouts illustrating hypothetical examples of commentary for particular situations on a road (2) (see Figures 3 and 4).

Each section of the experiment consisted of two trials. In Trial 1, the first group (pairs, driver and navigator) was assigned to go into the field and actually drive a designated route. While driving the route, the driver did commentary and identified the problem locations and the navigator made sure the driver stayed on the designated route. Drivers were told that they would be graded on their ability to identify all the problem locations on the route and to make the correct and appropriate comments that described the route. The second group (individuals) was given the same assignment with the exception that they demonstrated their ability to identify problem locations by looking at a prerecorded videotape of the same designated route. Both groups were told that they would be penalized for reporting a location that was not actually a problem location, for the same reason as that in Experiment 1. At the end of Trial 1, the subjects in both groups returned their tapes to the experimenter.

During Trial 1 of the second section, the experimenter decided to see whether more than one person could participate in the VIDEO condition at one time. He found that by using full audio protection earmuffs, he could keep the subjects from hearing one another's comments. He also found that by using external microphones, held close to the subject's mouth, the comments from one subject did not record on the tapes of the other subjects. The subjects were spaced about 5 ft apart. In this part of the experiment only four subjects were trained at a time, but it is believed that more can be trained if room space and the field of view to the video monitor are available.

Before the start of Trial 2, the experimenter listened to portions of each subject's tape. From these tapes he was able to get a fairly good idea of how well the subjects were doing. Then the experimenter talked to the subjects about the types of comments that they had made and gave several suggestions that might improve their performance.

After the conference between subjects and experimenter, the subjects were sent out to the route for the second trial. In Trial 2, the assignment was similar to that given during the first trial. Once again the subjects were to use commentary driving to pick out the information-deficient locations on a route. The route was the reverse direction of travel of the route driven in Trial 1. As with the first trial, the driver-navigator pairs drove the route. For this trial, however, the individuals responsible for doing commentary while viewing the videotape of the route in the first trial became drivers (commentators) and went out with a navigator to drive the route. Navigators were either the experimenter or someone who had previously finished this part of the experiment.

Measurement

The experimenter evaluated each subject's tape for each route and compared it with a key tape (the experimenter's evaluation of the routes). The subjects were graded according to the same criteria listed in the Measurement section of Experiment 1. The score was calculated for each subject by totaling the number of correct observations made, subtracting the number of errant observations, and then dividing this by the total possible for each of the routes. This score reflects a subject's percentage of correct observations for a route and allows for the comparison of the performance of the participants in both directions around the route. The total possible score for the first route was 733 and for the second (reverse) route it was 798. An explanation of how the experimenter compared the subjects' tapes with his own is presented elsewhere (5).

Results

As described in the earlier paragraphs, the second experiment consisted of two sections. The first involved the use of students as subjects, whereas the second used county-level transportation personnel. In each section there were 23 subjects split into two groups, which reduced the amount of data collected even further. In an effort to make the tests more sensitive, the experimenter believed that the data should be combined in such a way that only two conditions were left; either the subject (student or county personnel) drove the route or else he watched a video of the route. In other words, both types of subjects were combined into one large sample within each condition.

The hypothesis for this test was that the scores for the two types of subjects, within conditions, were from the same distribution. It was assumed that the data sets were all normally distributed. The F-test was used to determine the statistic (6).

The first set of data that was analyzed was the Trial 1 scores for the VIDEO condition. The mean score for the students was 51.7 (range, 37.8 to 60.4), whereas that of the county personnel was 51.3 (range, 41.6 to 58.1). The standard deviations were 7.2 and 5.4, respectively. The calculated statistic, F, was 0.02. The critical F-value with degrees of freedom of $\nu_1 = 1$ and $\nu_2 = 14$ and at level $\alpha = 0.05$ was 4.60. Therefore, because 0.02 is less than 4.60, there is no significant difference between the two samples, and the samples could be from the same distribution.

TABLE 3 Subject Scores by Subject for the Combined Groups of Subjects

	Trial 1		Trial 2	
Subject	Condition	Score (%)	Condition	Score (%)
1	VIDEO	37.79	DRIVE	41.73
2	VIDEO	45.70	DRIVE	50.38
3	VIDEO	46.38	DRIVE	40.60
4	VIDEO	51.71	DRIVE	56.39
5	VIDEO	53.21	DRIVE	68.92
6	VIDEO	55.80	DRIVE	51.00
7	VIDEO	56.48	DRIVE	41.60
8	VIDEO	57.71	DRIVE	70.68
9	VIDEO	60.44	DRIVE	60.28
10	VIDEO	41.61	DRIVE	67.04
11	VIDEO	49.25	DRIVE	56.02
12	VIDEO	49.80	DRIVE	70.80
13	VIDEO	51.71	DRIVE	55.76
14	VIDEO	51.71	DRIVE	46.37
15	VIDEO	56.75	DRIVE	56.52
16	VIDEO	58.12	DRIVE	74.19
17	DRIVE	30.97	DRIVE	28.57
18	DRIVE	34.79	DRIVE	52.76
19	DRIVE	38.74	DRIVE	61.40
20	DRIVE	43.66	DRIVE	51.00
21	DRIVE	44.20	DRIVE	50.38
22	DRIVE	55.66	DRIVE	60.15
23	DRIVE	57.30	DRIVE	70.55
24	DRIVE	38.47	DRIVE	44.49
25	DRIVE	40.93	DRIVE	63.78
26	DRIVE	44.75	DRIVE	63.41
27	DRIVE	45.43	DRIVE	64.04
28	DRIVE	49.25	DRIVE	62.66
29	DRIVE	50.89	DRIVE	71.68
30	DRIVE	50.89	DRIVE	81.33
31	DRIVE	51.98	DRIVE	68.17

Next the sample sets from Trial 2 for the VIDEO condition were analyzed. The mean score for the students was 53.5 (range, 40.6 to 70.7), whereas the mean score of the county personnel was 61.0 (range, 46.4 to 74.2). The standard deviations were 11.5 and 9.9, respectively. The calculated F was 1.68, and the critical F was 4.60 with the same parameters just listed. Again there was no significant difference between the two samples, and the two data sets were combined.

The third data set to be analyzed was from the subjects in the DRIVE condition of Trial 1. The mean for the students was 43.6 (range, 31.0 to 57.3), and the mean for county personnel was 46.6 (range, 38.5 to 52.0). The standard deviations were 9.9 and 5.0, respectively. The calculated F-value was 0.55. The critical F with $\nu_1 = 1$ and $\nu_2 = 13$ at $\alpha = 0.05$ was 4.67. Once again the samples could be combined.

The last sample that was checked for the possibility of combining the two types of subjects was the Trial 2 scores for the DRIVE condition. The students' mean score was 53.7 (range, 28.6 to 70.5) as compared with that of the county personnel, which was 64.9 (range, 44.5 to 81.3). The respective standard deviations were 13.1 and 10.4. The critical F was 4.67. The calculated F was 3.44. Therefore there was no significant difference between the two samples.

Because there was no significant difference between the two groups of subjects as noted in the four cases just discussed, the experimenter combined the two groups. The remaining analysis of data is based on the two combined groups of subjects. Table 3 shows the reduction of the data due to the combination of subjects.

Before running the F-test (6) on the combined data sets, the experimenter checked the assumption of normality by using the Kolmogorov-Smirnov one-sample test (7). The four cases tested were Case 1: VIDEO condition, Trial 1; Case 2: VIDEO condition, Trial 2; Case 3: DRIVE condition, Trial 1; and Case 4: DRIVE condition, Trial 2. The calculated statistics were 0.0844, 0.1330, 0.0880, and 0.1160, respectively. The critical values were 0.328 for the VIDEO condition ($N = 16$, $\alpha = 0.05$), and 0.338 for the DRIVE condition ($N = 15$, $\alpha = 0.05$). Therefore, because all of the calculated values were less than the respective critical values, the results of this test show that the samples can be assumed to be normally distributed.

The first F-test, using the combined subjects, was run on the data taken from the tapes of Trial 1. The mean score of the VIDEO condition was 51.5 (range, 37.8 to 60.4) with a standard deviation of 6.3. In contrast, the mean score of the DRIVE condition was 45.2 (range, 31.0 to 57.3) with a standard deviation of 7.6. The calculated value of F was 6.43, which was greater than the critical F with degrees of freedom $\nu_1 = 1$ and $\nu_2 = 29$ and at α-level 0.05 of 4.18. Therefore there is a significant difference between the two conditions at $\alpha = 0.05$. This means that, on the average, the VIDEO subjects did a better job than did the DRIVE condition subjects.

The final F-test was run on the Trial 2 scores for the combined subjects. The objective for taking this set of data was to draw conclusions about which of the two methods better prepares the subject for the real-world environment.

The results of the F-test are as follows: The mean score for the VIDEO condition was 56.8 (range, 40.6 to 74.2) with a standard deviation of 11.1. In comparison, the mean score for the DRIVE condition was 59.7 (range, 28.6 to 81.3) with a standard deviation of 12.7. The critical F was 4.18 with the same parameters as were listed in the previous test. The calculated statistic F was 0.47. Because 0.47 is smaller than 4.18, there is not a significant difference between the two conditions. In other words, both methods equally prepare the student for the real world, that is, prepare him to identify problem locations on the actual roadway.

In both experiments the subjects' scores were low compared with the experimenter's evaluation of the route. The reason for this is that the experimenter wanted the tests to be as sensitive as possible. Therefore, as he listened to the tapes, he was looking for very specific comments that were not necessarily mandatory, but that could have been made if the commentator had thought about it at the time, for example, the location of every crest vertical curve, where power poles [positive guidance (3)] switch from one side of the road to another, whether the adjacent land is wooded or farm ground, and so on. These comments do not really impose a constant threat to the driver but they are a part of the roadway environment.

Although the scores were low, the experimenter believes that subjects did a satisfactory job of finding the really critical problem areas on the roads. The experimenter could go back and reanalyze the tapes without looking for the specific comments, but he believes that the time consumed would be wasted on a trivial matter. The experimenter is convinced that the subjects will be able to do an evaluation on LVR county roads that is complete and correct.

DISCUSSION OF RESULTS

The VIDEO condition can be looked at as a simulation of the real world while driving in the real world. The VIDEO condition also provides the opportunity to create real-life situations and combinations of situations that may not be readily found on the local roads but that may confront the student somewhere

later. These situations can be set up and filmed and then removed so as not to pose a hazard to the drivers of the road. This allows for a multitude of "what-if" situations. The major drawback to this advantage is that it requires the road to be closed for the taping if the temporary situation is not a permanent feature of the road environment.

The instructor has no control over what the student in the field may miss when driving the roads. The instructor can, however, control what the student sees on the videotape. For example, assume that there is a sign, vital to the driver, with lettering too small to be read at the traveling speed or that is obscured by vegetation; the instructor can capture this sign on tape so that the student realizes that there is a problem at that location. Thus the student will be made aware that such situations do exist in the real world and will be able to find a corrective measure.

One major problem encountered in the DRIVE condition was the student driver's getting lost. This will always be a problem with the students learning by the DRIVE condition. Even with the navigator in the vehicle, the possibility of this problem exists. With videotapes of the route there is no possibility of the driver's getting lost. The VIDEO condition allows the driver to concentrate on the task of learning to do commentary driving and picking out the problem locations without losing his way.

The VIDEO condition allows for the training of people in remote counties that cannot afford to send someone to a central location for the needed training. The equipment is relatively lightweight and compact. The instructor, with considerable time, can locate various routes that have the same or familiar terrain as that found in the county that he will be visiting. He can then get these routes on videotape and take them to the county with him. Then as he trains personnel from other counties with similar terrain, he can use these same tapes. With the DRIVE condition the instructor would still have to go out several days in advance, locate routes to drive, and then put on the workshop, and if he needed to visit another county, he would to go that county and find even more routes instead of using the routes he had already found. The VIDEO condition is also independent of weather conditions present during the training period. If necessary, the videotapes can be used to train students at night who are normally too busy during the daytime hours.

The VIDEO condition can be used to train several people at the same time; therefore less valuable time is wasted than is necessary with the DRIVE condition. The multiple-person training session requires the use of full audio protection earmuffs and would be aided by the presence of more than one video monitor. The DRIVE condition requires a separate vehicle for each driver-commentator; therefore one must take into account the added expenses incurred.

CONCLUSIONS

Results of the Study

It can be concluded that students learn to do commentary driving equally well, if not better, by watching videotapes of routes than if they were sent out in an automobile to do the commentary while driving the same routes. It has been proven that a student will be able to do commentary driving in a real-world situation, driving the roads, even though he was trained to do the technique by watching a videotape of the route.

On the basis of the experience with Kansas county personnel in early 1986, the commentary driving technique and the use of information-deficient location checksheets can be taught in a 1-day workshop; a realistic schedule of activities for this workshop is as follows:

1. Introduction, purpose of workshop, and so on (0.5 hr);
2. Review of use of LVR Handbook (1.0 hr);
3. Introduction to commentary driving, examples, instructions for doing commentary driving from videotapes (1.0 hr);
4. Participants do commentaries and audiotapes from 30-min videotape (two video monitors, five participants per monitor, participants wear earplugs or muffs); 40 min per group of 10 participants should be allowed (2.0 hr);
5. Evaluation of commentary audiotapes by participants (students check the students) (1.0 hr);
6. Presentation, discussion, and instruction in the use of checksheets (0.5 hr); and
7. Feedback on participant commentaries (general observations on commentaries; meet with any individuals having particular problems with the technique) (1.0 to 2.0 hr).

This schedule assumes that the participants are experienced in the use of the LVR Handbook and that the number of participants is 30 or fewer. It has been found that the length of the videotape for commentaries could be reduced to about 30 min if various roadway sections or situations were carefully selected.

The most time-consuming part of the workshop was the evaluation of individual participant commentary tapes by the instructional staff and feedback to the participants. The evaluation could take about 10 to 15 hr of instructional staff time. In view of this problem, the students checked the other students, that is, participants (students) exchanged commentary audiotapes. Each participant then listened to the exchange audiotape and checked the accuracy of the commentary against a worksheet. The worksheet was prepared by the workshop instructional staff and contained, in sequential order, brief statements of the most important comments. It was found necessary to include the tape-counter number at regular intervals on the worksheet so the checker would not lose his place. This was necessary because the amount of commentary differed considerably among participants. The tape-counter number is a surrogate for the vehicle odometer reading used in specifically locating problem spots during commentary driving on the roads.

The scores from each worksheet for each participant were checked by the instructional staff. Each checker was asked whether he believed that the person whose audiotape he evaluated could do commentary driving. Each participant was also asked whether he believed that he could do commentary driving. For those persons with problems in doing commentary driving, the instructional staff gave additional instruction and answered individual questions.

Checksheet Evaluation

The checksheets (Figures 1 and 2) are based on the concept of decision sight distance (8,p.70;9). These checksheets were introduced to the group of county personnel in a workshop situation. They were asked to look over the checksheets and then give the instructors their opinion of how useful the sheets might be. The consensus was that the checksheets were ideally suited for suggesting treatments of sites found to be information deficient. The county personnel also agreed that the checksheets were easily followed and self-explanatory.

Use of Tape Recorders

It was found that only a short period of time was required by the subjects in both experiments to become relaxed while talking into the tape recorder. While listening to the tapes, the experimenter noticed that most of the subjects sounded awkward in their initial comments. After about 2 or 3 min, the subjects adjusted and there was a noticeable improvement in both the types of comments made and in the confidence and voice qualities with which these comments were made.

Summary

Commentary driving is a useful technique for highway personnel in the everyday safety evaluation of their projects. Although this paper has dealt only with its use on county low-volume roads in Kansas, it should be helpful in many other situations on higher-volume roads and highways. In particular, the technique could well be used at work-zone sites, school zones, and in the evaluation of signing and warnings at narrow or one-lane bridge sites.

ACKNOWLEDGMENT

The authors wish to thank the Kansas Department of Transportation for the financial support provided for this study.

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Publication of this paper sponsored by Committee on Low Volume Roads.

Waterbound Macadam as a Base and as a Drainage Layer

EMILE HORAK and RALPH H. H. TRIEBEL

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

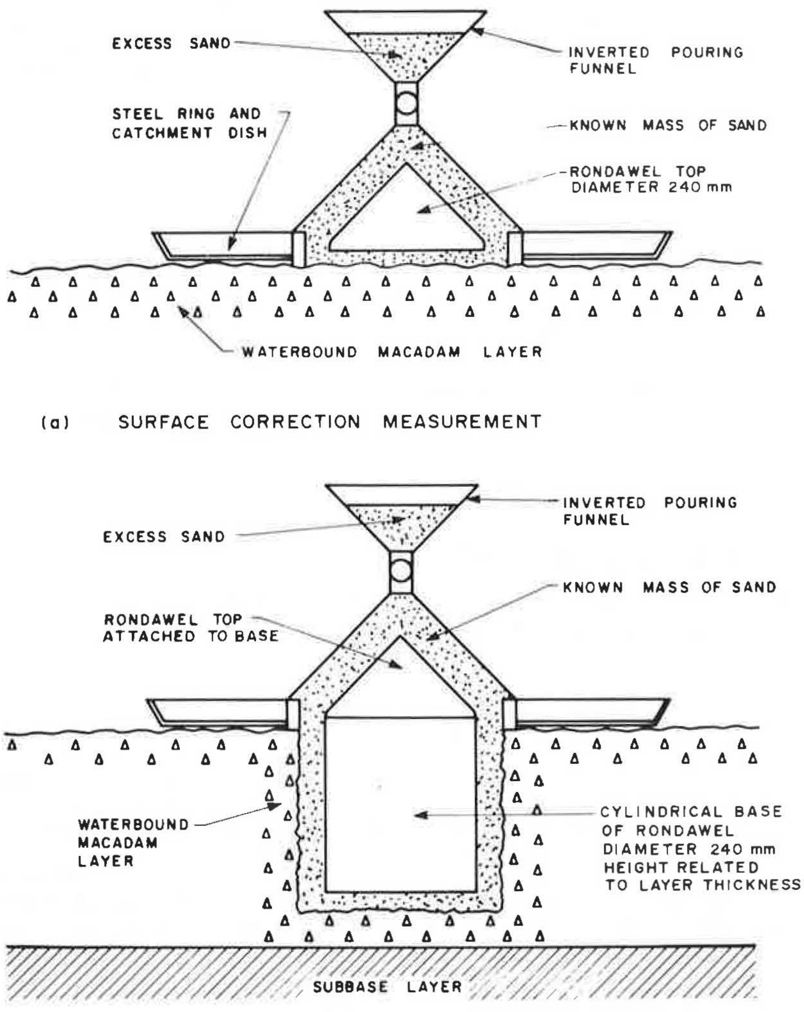
a base layer should not be underestimated, as discussed later.

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 $\nless$ 4
	G2	Graded crushed stone	Dense-graded unweathered crushed stone; Max. size 37,5 mm 100-102 % mod. AASHTO. Fines PI $\nless$ 6
	G3	Graded crushed stone	Dense-graded stone + soil binder; Max. size 37,5 mm; Minimum 98% mod. AASHTO. Fines PI $\nless$ 6
	G4	Natural gravel	CBR $\nless$ 80; PI $\ngtr$ 6
	G5	Natural gravel	CBR $\nless$ 45; PI $\ngtr$ 10 to 15 depending on grading; Max. size 63 mm.
	G6	Natural gravel	CBR $\nless$ 25; Max. size $\ngtr$ $\frac{2}{3}$ layer thickness
	G7	Gravel-soil	CBR $\nless$ 15; Max. size $\ngtr$ $\frac{2}{3}$ layer thickness
	G8	Gravel-soil	CBR $\nless$ 10; at in situ density
	G9	Gravel-soil	CBR $\nless$ 7; at in situ density
	G10	Gravel-soil	CBR $\nless$ 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 $\nless$ 3,8 MPa; Max. size $\ngtr$ 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 $\ngtr$6 88-90% of apparent density
	WM2	Waterbound macadam	Max. size 75 mm, PI of fines $\ngtr$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 $\nless$ 30 MPa; interlocking shapes



(a) SURFACE CORRECTION MEASUREMENT

(b) DETERMINATION OF VOLUME OF HOLE

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).

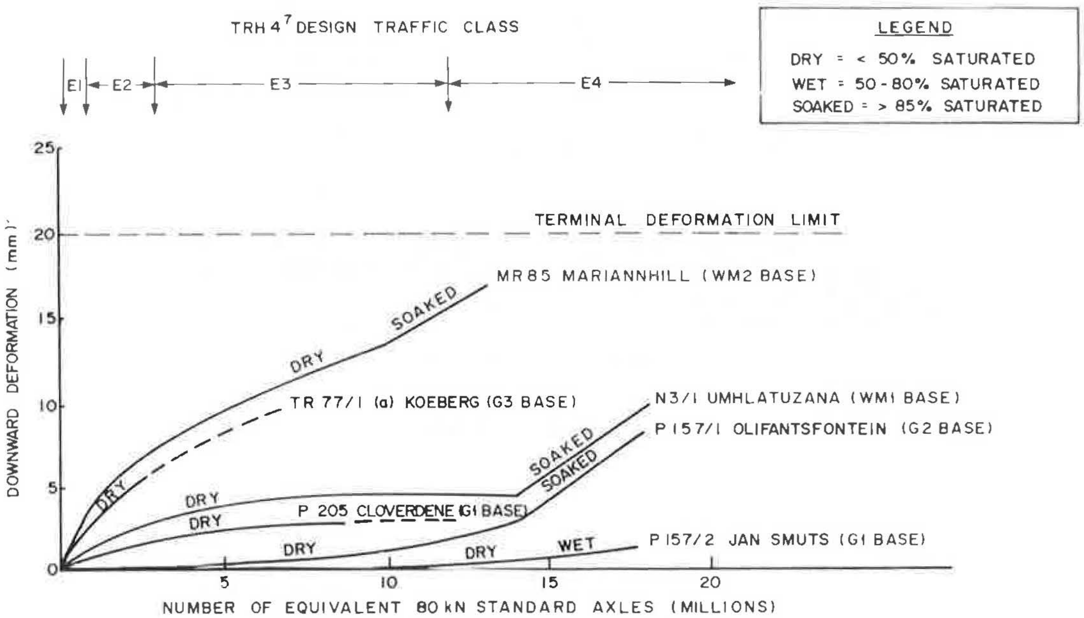


FIGURE 2 Comparison of the pavement deformation induced in various crushed-stone base pavements by HVS trafficking.

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:

$$D = (P/80)^d \tag{1}$$

where

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

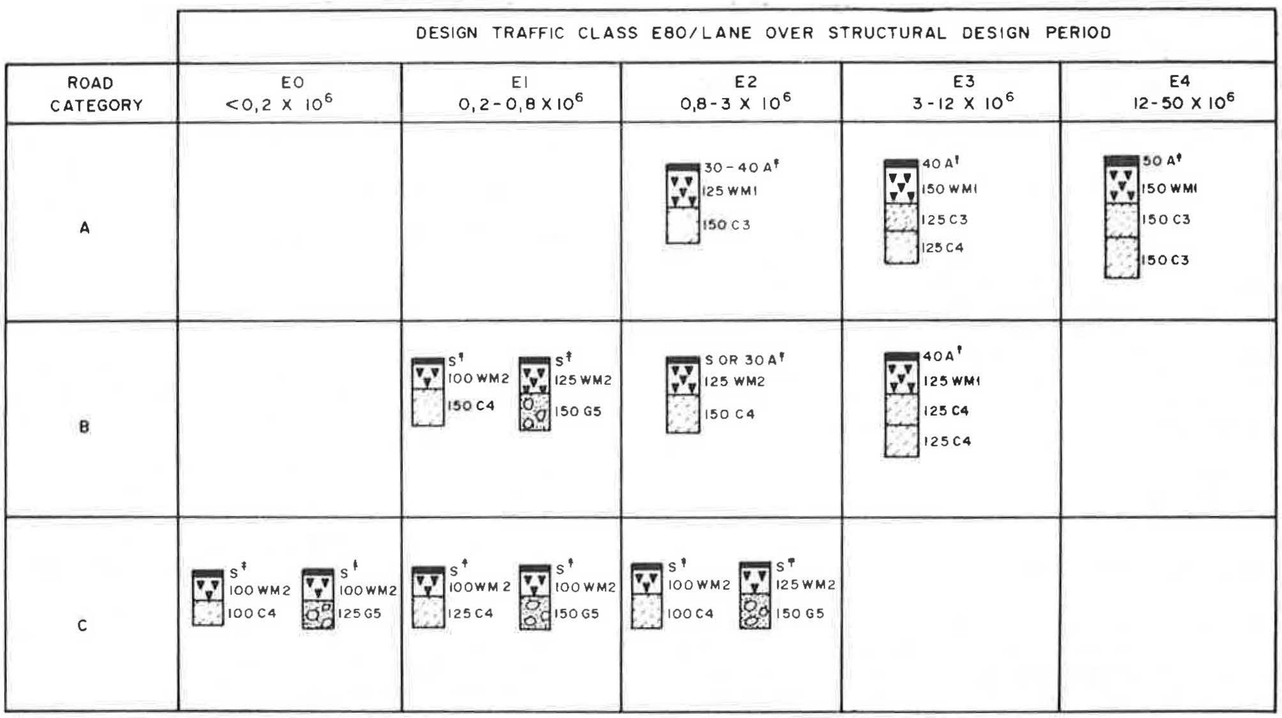
A value of d equal to 3 rather than 4.2 was found to be applicable to deep pavement structures with granular bases (9). In Figure 2 a comparison is made of various granular-based pavements tested with the HVS. The behavior of typical WM1 and WM2 quality bases is compared with the findings of Maree et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

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

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 50×10^6 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.

ACKNOWLEDGMENT

The authors wish to extend their thanks to the Chief Director of the National Institute for Transport and Road Research for permission to publish this paper and to the consultants and contractors of the Department of Transport, Natal Roads Department, for their involvement in this project.

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Publication of this paper sponsored by Committee on Mineral Aggregates.

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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of the application of regression analysis to cost estimation.

DATA SOURCE

A sample of 26 projects was collected from the western United States. The sample included road projects constructed in the study area during 1980-1982. The information on quantities, bid cost, and average bid unit-cost estimates was provided by an automated bid tabulation system. Other road characteristics such as side slope, clearing, and so on, were collected by questionnaire.

The six components investigated were engineering, earthwork, bases, pavement, bridges, and other incidental items (4). A review of past projects in the study area indicated that although the relative proportion of total cost attributable to each of the components varies from job to job, earthwork cost generally constitutes the largest portion of the total (nearly 50 percent). The average percentage composition of the total construction cost for the selected 26 roads was roughly as follows:

Item	Portion of Total Construction Cost (%)
Engineering	2.7
Earthwork	46.9
Bases	26.4
Pavement	4.1
Bridges	0.7
Incidental	19.2

These six components are used as the explanatory variables that make up the total project cost. Fourteen major cost items that contributed the bulk of the costs attributable to the six components were chosen for estimating unit prices.

METHODOLOGY

The system assumes that the bid price is a function of the project characteristics that directly influence the required effort to complete the project and the scale of economy. These characteristics may affect the bidding behavior and bid cost. For this study they were identified as follows:

1. Side slope (percent),
2. Soil condition:
   a. Common (percent),
   b. Solid rock (percent),
   c. Riprap (percent),
3. Clearing:
   a. Light (percent),
   b. Medium (percent),
   c. Heavy (percent),
4. Remoteness (travel distance from center of local community to project by miles),
5. Length of road (miles),
6. Net gradient of the project (percent),
7. Complexity of project (in terms of number of items), and
8. Quantity of project (by item or overall).

In addition to the aforementioned factors, socioeconomic conditions including per-capita personal income, unemployment rate, the number of construction workers, and the number of bidders were also used to examine the bidding behavior. However, it has been found that these factors have no significant relationships to the bid cost.

Two types of models will be developed. One is for estimating the unit prices of major items and the other is to estimate the total cost of a project. They are discussed in the following paragraphs.

Unit Price

Unit price (U) as a function of road characteristics (y_r) and quantities (q_k) is

$$U = a + \sum_{k=1}^{d} b_k q_k + \sum_{r=1}^{e} b_r y_r \qquad (1)$$

where a and b are constants. Fourteen major items were selected for the unit-price estimate. They are classified as follows:

201 (01) Clearing and grubbing
201 (03) Clearing and grubbing
203 (01) Excavation, Method 1
203 (02) Excavation, Method 2
203 (03) Excavation, Method 3
304 (01) Crushed aggregate
306 (01) Reconditioning of roadbed
408 (01) Liquid asphalt
601 (01) Mobilization
603 (01) Corrugated metal pipe
611 (01) Pit development
619 (01) Hand-placed riprap
621 (01) Spillway inlet assemblies
625 (03) Seeding, hydraulic method with mulch

Total Cost

Total cost (T) as a function of the sum of the estimated low-bid prices by items (c_i) is

$$T = a + b \sum_{i=1}^{n} c_i \quad 1 = 1, 2, 3, \ldots, n \qquad (2)$$

where n is the number of items and a and b are constants to be estimated. Note that the low bidder differs in his prices from item to item and is not necessarily the bidder who was awarded the contract.

Total cost (T) as a function of the costs of the major components (c_j) is

$$T = a + \sum_{j=1}^{m} b_j c_j \quad j = 1, 2, 3, \ldots, m \qquad (3)$$

where m is the number of major components and other terms are as defined previously.

Total cost (T) as a function of the sum of the relative importance scores in terms of cost (s_i) is

$$T = a + b \sum_{i=1}^{n} s_i \quad i = 1, 2, 3, \ldots, n \qquad (4)$$

where the relative importance scores may be developed as follows:

1. Compute the average cost by item,
2. Select the item with the least deviation as the basic item,
3. Compute the weight of each item by dividing the cost of each item with the cost of the basic item, and
4. Multiply the quantity of each item by its weight and obtain the relative importance scores.

Total cost (T) as a function of relative importance scores of the major components (s_j) is

$$T = a + \sum_{j=1}^{m} b_j s_j \qquad (5)$$

Total cost (T) as a function of preliminary quantity estimate (q_k) and road characteristics (y_r) is

$$T = a + \sum_{k=1}^{d} b_k q_k + \sum_{r=1}^{e} b_r y_r \quad \begin{matrix} k = 1, 2, 3, \ldots, d \\ r = 1, 2, 3, \ldots, e \end{matrix} \qquad (6)$$

where d and e are the number of items and other terms are as defined previously.

The regression analysis will be used for the model calibration. The stepwise precedure, that is, entering a variable at a time, of the Statistical Package for the Social Sciences was used for computation.

MODEL CALIBRATION

Unit-Price Models

As shown in Table 1, unit-price models were developed for 14 major cost items. The data indicate that the unit price of clearing and grubbing, in terms of dollars per acre, is related to the percentage of clearing in the light category, side slope, road length, and time of the year when the bid took place. The first two variables represent the effort required and the third variable represents the project size. The signs for these three variables are as expected. The variable for the time of the year is a dummy variable that equals 1 for the time period of April to September and 0 otherwise. This means that when the bid takes place during the construction season, the unit price for clearing and grubbing is \$213/acre lower than that of the off-construction season bid. The coefficient of determination (R^2) for the equation is 0.2194.

When clearing and grubbing is measured in dollars per mile, its unit price can be explained by its quantity and side slope. As expected, the unit price tends to be reduced when the size of the project is larger. On the other hand, the increase in side slope tends to increase the unit price. The coefficient of determination (R^2) of the model is 0.3538.

The size of the project in terms of road length has been found to be a significant variable for explaining the unit price of excavation ranging from Methods 1 to 3. However, the unit price of excavation, Method 1, is also related to the total excavation of a project, whereas the unit price of excavation, Method 2, is also affected by the percentage of solid rock and time of year when the bid took place. The three models for the unit price of excavation are significant, with R^2 ranging from 0.6964 to 0.8731.

Two models have been developed for the component of bases. One is for crushed aggregate and the other is for reconditioning of the roadbed. The unit price of crushed aggregate has been found highly related to the quantity of crushed aggregate and road length. The R^2 is equal to 0.9309. The unit price for reconditioning of the roadbed can be explained by side slope and time of the year when the bid took place. However, R^2 for the model is only 0.2898. The low value of R^2 is due to the stability of the unit price for this item.

The unit price of liquid asphalt is highly related to the quantities of liquid asphalt and the total excavation. The model indicates that the liquid as-

TABLE 1 Unit-Price Models

Specification No.	Description of Item	Unit-Price Model	R^2	Mean (\$/unit)	Standard Error of Estimate	Durbin-Watson Test
201 (01)	Clearing and grubbing (\$/acre)	$U = 2{,}987.49 - 14.6864X_1 - 1.1805X_2 - 213.5127X_3 + 11.220X_4$	0.2194	2,742.45	1,359.57	1.7461
201 (03)	Clearing and grubbing (\$/mi)	$U = 1{,}667.82 - 174.6936X_5 + 48.0763X_4$	0.3538	1,196.23	1,820.34	1.7184
203 (01)	Excavation, Method 1 (\$/yd^3)	$U = 2.63 - 0.0140X_6 - 0.00036X_2$	0.7628	1.96	0.38	2.1573
203 (02)	Excavation, Method 2 (\$/yd^3)	$U = 2.78 + 0.0715X_7 - 1.2506X_3 - 0.00053X_2$	0.8731	2.09	0.42	1.7052
203 (03)	Excavation, Method 3 (\$/yd^3)	$U = 8.02 - 0.7174X_2$	0.6964	3.94	1.56	2.1752
304 (10)	Crushed aggregate (\$/yd^3)	$U = 19.54 - 0.00108X_8 - 0.00198X_2$	0.9309	1.34	13.66	3.1312
306 (01)	Reconditioning of roadbed (\$/mi)	$U = 286.19 + 14.3063X_4 - 68.1271X_3$	0.2898	847.67	267.03	1.7702
408 (09)	Liquid asphalt (\$/ton)	$U = 266.19 - 0.1568X_{10} - 0.7525X_6$	0.8865	195.00	130.51	1.9974
601 (01)	Mobilization (\$/job)	$U = 1{,}555.82 + 125.0380X_9 - 8.6642X_2 + 213.4994X_6$	0.7539	13,324.95	9,422.94	2.2350
603 (01)	Corrugated metal pipe (\$/ft)	$U = 26.61 + 3.2709X_7 - 0.2364X_4 - 0.0013X_2$	0.7955	25.73	10.22	1.1010
611 (10)	Pit development (\$/pit)	$U = 3{,}472.56 + 65.2432X_{11} - 2{,}359.7274X_3 - 0.2272X_{12}$	0.9271	1,714.29	760.02	1.8307
619 (01)	Hand-placed riprap (\$/yd^3)	$U = 70.39 - 5.5560X_2$	0.3549	47.47	22.19	2.3570
621 (01)	Spillway inlet assemblies (\$/each)	$U = 4.64 + 2.515X_7 + 1.484X_9 + 0.5282X_4 + 0.7984X_{11}$	0.7933	88.85	18.08	1.8184
625 (03)	Seeding, hydraulic method with mulch (\$/acre)	$U = 502.11 - 74.9600X_{13} + 20.7764X_4$	0.4320	902.71	501.77	1.1744

Note: Variables are defined as follows:
- U = unit price (\$1,000 per unit),
- X_1 = percent of clearing in light category (%),
- X_2 = length of road (mi),
- X_3 = time of year (1 for April to September period and 0 otherwise),
- X_4 = side slope (%),
- X_5 = clearing and grubbing (acres) – 201 (03),
- X_6 = total excavation (1,000 yd^3),
- X_7 = percent of solid rock (%),
- X_8 = crushed aggregate (yd^3) – 304 (01),
- X_9 = remoteness (mi from local community to project),
- X_{10} = liquid asphalt (ton),
- X_{11} = percent of riprap (%),
- X_{12} = pit-run aggregate (yd^3) – 304 (01),
- X_{13} = seeding, hydraulic method with mulch (acre) – 625 (03), and

R^2 = coefficient of determination (ranges from 0 to 1 for the quality of model from poor to perfect).

Standard error of estimate = $[(u - \bar{u}^2)/(n - 2)]^{1/2}$, where $\bar{u}$ is the mean, u is the estimated value, and n is the number of observations.

phalt unit price tends to be low when the amounts of asphalt and excavation are large. For paved roads, excavation accounts for more than half of the new construction cost and may represent the size of the project. The trade-off between the liquid asphalt unit price and excavation quantity is expected. However, this trade-off is ignored by the conventional cost-estimate approach. The model has an R^2 as high as 0.8865.

The last six models of Table 1 were developed for the six major items of incidental construction. As expected, the cost of mobilization is highly related to the remoteness or distance from the local community to the site of the project and the length of the road. Because the remoteness reflects the transportation cost, and the road length represents the size of the project, the bidder considers distance as the major factor for determining mobilization cost and is willing to trade off this cost with the cost of other items. The model has a significant coefficient of determination, $R^2 = 0.7539$.

The unit price of corrugated metal pipe can be explained by the percentage of solid rock, side slope, and road length. Solid rock requires extra effort for excavation and thus tends to increase unit price. On the other hand, steep ground requires less effort for pipe installation and tends to reduce unit price. The equation also indicates that the bidder is willing to trade off the unit price of corrugated metal pipe with the size of the project in terms of road length in miles. The unit price of pit development is a function of the percentage of riprap, the quantity of aggregate, and time of the year when the bid took place. As expected, riprap increases difficulty in pit development and enhances unit price. However, the cost would be reduced if the quantity of aggregate to be produced is large or the bid takes place in the construction season. These two models are highly significant, with R^2 equal to 0.7955 and 0.9271, respectively.

The unit price of hand-placed riprap has been found to be related to road length. In this case, the road length represents the size of the project or the quantity of hand-placed riprap, or both. Therefore, the longer the road segment is, the lower the unit price of hand-placed riprap. Four important explanatory variables included in the model for the unit price of spillway inlet assemblies are percentage of solid rock, remoteness, side slope, and percentage of riprap. All of these four factors tend to increase the unit price of spillway inlet assemblies because of the difficulty in installation or high transport cost. The unit price of seeding by the hydraulic method with mulch can be explained by the quantity of such seeding and the side slope. The coefficients of determination for these three models are 0.3549, 0.7933, and 0.4320, respectively.

The foregoing discussion indicates that the unit price is determined by the level of effort required for accomplishing a job such as a side slope, category of clearing, type of soil, and the size of the project in terms of road length or quantities of specific items. The more the required effort is, the higher the unit price. On the other hand, the larger the project is, the lower the unit price.

Total-Cost Models

Five models for estimating total cost of a project are given in Table 2. The first two models require estimating unit price and quantities for all items, and the third and fourth models require estimating unit price and quantities for major items. The last equation requires only an estimate for asphalt and gravel in terms of thickness (inches) as well as the work on excavation (cubic yards).

The first model assumes that the total cost of a project is a function of the sum of low-bid costs for all items. The second model indicates that the gross construction cost is highly related to the sums of low-bid costs by items for components of earthwork, bases, and incidental construction. The model also reveals that a project requiring construction staking tends to lower the cost. The coefficients of determination for both models are 0.9643 and 0.9807, respectively.

A set of the relative importance scores by item were derived from the average unit price. The product of these scores and the engineering estimated quantities forms the data base for developing the second and third equations of Table 2. The assumptions of these two models are similar to that of the first two equations. The coefficients of determination for both models are 0.9504 and 0.9755, respectively.

The last model of Table 2 is composed of three independent variables including pavement index, aggregate index, and total excavation. Both indices

TABLE 2 Total Cost Models

Approach of Modeling	Total Project Cost Model	R^2	Mean ($000s)	Standard Error of Estimate	Durbin-Watson Test
Aggregated low-bid costs	$T = 12.860 + 0.00156X_{23}$	0.9643	381.39	69.01	1.9448
Itemized low-bid costs	$T = 12.736 + 0.00182X_{22} + 0.00484X_{24} + 0.00065X_{25} + 0.0027X_{26}$	0.9807	381.39	54.42	1.6294
Aggregated scores	$T = 28.667 + 0.00321X_{27}$	0.9391	381.39	88.76	1.9406
Itemized scores	$T = 31.602 + 5.0211X_{28} + 0.00157X_{29} + 0.0137X_{30} + 0.0562X_{31} + 0.0542X_{32}$	0.9685	381.39	71.20	1.7316
Quantities and road characteristics	$T = 66.79225 + 1.82267X_{33} + 5.61322X_{34} + 1.00170X_{35}$	0.8484	381.39	148.50	1.7106

Note: Variables are defined as follows:

T = total cost of a project ($000s),
X_{22} = 1 if the component of bridge construction is included in the project,
X_{23} = sum of low-bid costs by items ($),
X_{24} = sum of low-bid costs by items of earthwork,
X_{25} = sum of low-bid costs by items of bases,
X_{26} = sum of low-bid costs by items of incidental construction,
X_{27} = sum of relative importance scores for overall project,
X_{28} = sum of scores for clearing and grubbing (201, 202, 207, 209, 210, 211, 212),
X_{29} = sum of scores for excavation (203, 205, 206),
X_{30} = sum of scores for bases,
X_{31} = sum of scores for bituminous pavements,
X_{32} = sum of scores for incidental construction,
X_{33} = pavement index, which is equal to road length (mi) times thickness of pavement (in.) times asphalt haul distance (mi),
X_{34} = total excavation (yd^3 000s), and
X_{35} = aggregate index, which is equal to road length (mi) times thickness of aggregate (in.) times aggregate haul distance (mi).

TABLE 3 Estimated and Actual Costs of Six Road Projects

	Project 1	2	3	4	5	6	Average[a]
Type of construction	New	New	Reconstruction	New and reconstruction	New and reconstruction	Reconstruction	–
Date of advertisement	11/24/81	5/1/80	8/3/82	6/15/82	11/16/81	9/2/82	–
Low bid ($)	184,234	418,865	1,618,584	165,060	39,030	561,915	–
Average-bid unit-price estimate ($)	225,811	302,460	2,052,814	205,618	44,315	737,459	–
Percentage difference	+22.6	–27.8	+26.8	+24.6	+13.5	+31.2	24.4
Aggregated low-bid cost model ($)	173,550	324,600	2,023,550	181,700	41,260	415,900	
Percentage difference	–5.8	–27.5	+25.0	–10.1	+5.7	–26.0	15.8
Itemized low-bid cost model ($)	181,630	409,600	1,631,520	157,700	40,730	515,270	
Percentage difference	1.5	–2.2	+0.8	–2.6	+4.3	–9.0	3.0
Aggregated score model ($)	173,410	404,160	1,830,000	158,100	44,730	593,830	
Percentage difference	–5.9	–3.5	+13.1	–4.2	+14.6	+5.7	9.9
Itemized score model ($)	186,260	373,590	2,149,000	176,300	63,100	531,450	
Percentage difference	+1.1	–10.8	+32.8	+6.8	+61.7	–5.4	19.8

Note: + = overestimate, – = underestimate.
[a]Absolute value.

are the product of quantities and haul distance, that is, thickness (inches) times road length (miles) times haul distance (miles). This model does not require an engineering estimate and can be used to predict the costs for projects in the planning stage. The coefficient of determination for the last model is 0.8484.

The foregoing discussion indicates that by using regression analysis, a cost estimating procedure can be developed. All the explanatory variables selected in the modeling analysis are used in actual road construction.

MODEL VALIDATION

Six projects were selected from the study area for model verification. These projects were not included in the model development and had costs ranging from $44,000 to over $2 million per project.

Because the validation of unit-price models required additional data collection, the equations contained in Table 1 were not verified. However, by assuming a 40 percent clearing in the light category, a 25 percent side slope, a 2-mi road project, and a July bid date, the unit price for 201(01) clearing and grubbing was computed by using the first equation in Table 1, as follows:

$$
\begin{aligned}
\$2,956.14 &= \$2,956.14 - (14.686 \times 40.0 = 587.44) \\
&\quad - (1.1805 \times 2.0 = 2.36) - (213.52 \times 1.0 \\
&= 213.52) + (11.220 \times 25.0 = 280.50) \\
&= \$2,433.32.
\end{aligned}
$$

The first four models of Table 2 were applied to the six selected projects. The result is shown in Table 3, in which it is shown that all of the models developed gave better preliminary estimates than simply applying average-bid unit prices without considering the project characteristics. The average difference from the actual low bid was 24.4 percent utilizing the average-bid unit-price estimating procedure and the range was from 3.0 percent for the itemized minimum-cost model estimate to 19.8 percent for the itemized-score model estimate.

Note that in the foregoing applications, the actual unit prices as bid were used in the estimation. Assume that the use of unit-price models will result in a 10 percent of error of the estimate. The ranges of deviations for the four models will be 16.7 to 17.3 percent, 8.3 to 11.7 percent, 9.3 to 13.6 percent, and 20.5 to 24.6 percent, respectively. Therefore, with consideration of unit-price estimate errors, these models can still yield better preliminary estimates than that made by the average-bid unit-price estimate.

SUMMARY AND CONCLUSIONS

Multiple regression analysis was applied to historical-bid data to develop estimating models to determine preliminary construction costs of low-volume roads. A sample collected from western United States was used to develop 14 unit-price models for major items and five total project cost models for total construction costs. In the modeling, 13 project characteristics were identified and analyzed as the independent variables of unit-price equations. Fifteen component quantities were utilized for developing total project cost equations. The study clearly indicated that extensive data-gathering effort is required to develop models. However, once the model has been developed, it requires no more data than the existing cost-estimate practice. On the basis of a verification check, it was found that by using regression analysis, it is possible to estimate preliminary construction cost for low-volume roads in the western United States with a higher degree of reliability than the average-bid unit-price estimate.

It was found that the bid price is a function of the effort required to complete a job item and the size of the project. The effort is defined by the level of clearing and grubbing, side slope, soil conditions, and remoteness, whereas the project size is described by the quantity of a particular job item. Less effort and large projects tend to lower the unit price, and vice versa. Two total project cost models require both engineering quantities and unit prices; the other three models require only engineering quantities. These models were developed to evaluate the feasibility of using regression analysis for preliminary cost estimating and have not been implemented. Therefore, information on the required time and expense involved in doing a cost estimate by using these models is not available at this time.

ACKNOWLEDGMENT

The authors would like to extend their appreciation to Ken Tompkins, Chief Engineer, Transportation Preconstruction and Design, Forest Service, U.S. Department of Agriculture, for his helpful comments.

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Publication of this paper sponsored by Committee on Low Volume Roads.