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Design and Upgrading of Surfacing and Other Aspects of Low-Volume Roads

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Evaluation of Existing Aggregate Roads to Determine Suitability for Resurfacing

ELDON J. YODER, D.G. SHURIG, AND BENJAMIN COLUCCI-RIOS

A major problem of counties in the United States is maintenance costs for local roads in an energy shrinking environment and under constraints that result from reduced revenues for maintenance. Many local roads have been built in the United States over the past century. In some cases these are largely nonsurfaced aggregate roads. On the county and state level, major construction of local roads is largely complete in the United States, but increased traffic has resulted in a need to upgrade many of these roads, particularly those near metropolitan areas. Increased truck traffic on many of these roads has caused considerable distress with resulting rising maintenance costs. There is a need to evaluate these roads to determine whether they are suitable for upgrading. Often a gravel surface that is satisfactory without a bituminous-wearing course becomes water logged and unsuitable after paving. This paper discusses concepts for evaluating existing nonsurfaced roads with the end point of determining suitability for placing a surface. Emphasis is placed on field tests that can be made to evaluate the in situ strengths of existing roads. Rapid tests that might be used for evaluation are discussed. These tests include the use of penetrometers and impact tests. Use of deflection measurements in evaluating nonsurfaced roads is presented. The advantages of performing a large number of tests by using quick and simplified methods are discussed. Location and types of tests that should be made are discussed. Methods for analyzing these data, including quantity and quality requirements, and recommendations that might be made for upgrading the surface are also presented.

A major problem of counties in the United States is maintenance costs for local roads in an energy shrinking environment and under constraints that result from reduced revenues for maintenance. Fifty to sixty years ago a concentrated effort was made by county, state, and federal governments to provide at least an aggregate wearing course for local low-volume roads. Gravel from local deposits and crushed stone from local quarries were hauled and spread on the road to provide a base course and support light traffic.

Major construction of local roads on the county and state level is largely complete in the United States, but increased traffic has resulted in a need to update many of these roads, particularly those near metropolitan areas. As traffic loads and volume increased, so also has the need to perform surface maintenance on these unpaved roads. In general, maintenance consisted of the periodic adding of additional gravel or crushed stones to locally failed areas. After the aggregate was spread, this was usually followed by drag maintenance to smooth the surface.

As a result, many of the existing gravel roads in the United States were not designed as such but came about through the evolution of road construction and road maintenance.

Maintenance needs for unpaved roads are still determined largely by visual inspection of what is needed to accommodate local traffic. In many cases, however, increased truck traffic on many of these roads has caused considerable distress with resulting rising maintenance costs.

As the need to upgrade county roads mounts and revenues decline, there is a need to make the most efficient use of available funds. Unfortunately, instances arise where placement of a wearing surface on an existing road can in fact cause detrimental effects. The base may become softened due to water accumulations since the road can no longer breathe due to the impervious surface. There is an obvious need to determine the quality of the existing base material before proceeding with application of a surface.

In a different context, in developing areas, roads are often planned for stage construction. In these cases the road may be placed under fairly close materials and design specifications and a planned resurface placed sometime during the life of the pavement. In this case also, there is a need to evaluate the existing road to determine what needs to be done to bring the base up to design standards before proceeding with the next stage of construction.

This paper focuses on quick field tests that can be used to evaluate existing aggregate roads. These methods measure equivalent California bearing ratio (CBR) values by using rapid field tests.

REASONS FOR EVALUATION

The type of evaluation process to use and the subsequent alternatives that might be considered depend on the needs for making the evaluation, as illustrated in Table 1. Data in this table present only a few of the items; it is difficult at best to generalize problems of this type but it is essential that the scope of the problem be kept in perspective.

Development of an Area

In the development of new areas, the justification may be both sociological and economical. This requires that a network analysis be made to determine potential growth of the area to determine the economic gain derived from the planning process. Alternatives for solution are listed in the last column of Table 1.

Environmental Factors

Dusting can be a problem not in terms of safety but also from environment of the surrounding terrain. Here again safety considerations, public opinion, and policy may dictate that something be done but this would not require a complete evaluation since application of a dust palliative may be all that is required.

Safety

Particularly at the county road level, increased traffic near metropolitan areas can cause safety problems from dusting, skidding, and other problems associated with nonsurfaced roads. Here public opinion and public policy coupled with safety aspects require that something be done to improve the surface. Along with this, increased truck traffic may cause an increase in maintenance costs. Before proceeding, the engineer would need to check the grade and alignment and all other physical aspects of the road and then make a general evaluation.

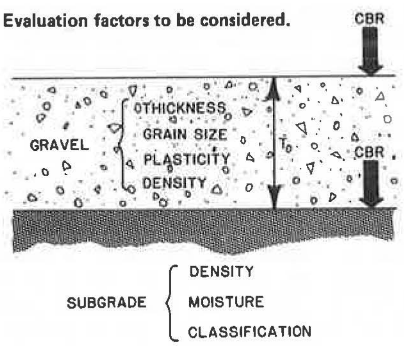
Planned Staging Other Than for Developing Areas

In many cases it can be shown that the most economical approach to pavement design is that of staging the construction of a pavement. Justification for improving the pavement can simply be the staging process itself although it should be based on sociological and economic factors.

Table 1. Reasons and alternatives for evaluation of existing aggregate roads.

Reason for Evaluation	Type of Evaluation	Alternative
Development of an area–sociological and economical	Network analysis	
Environmental factors–public opinion, public policy	Evaluate road materials	Stabilize base, apply surface
Safety–increased traffic, dangerous intersections, narrow bridges	Check grade, alignment, and other geometric factors	Widen, reconstruct
Planned staging other than for a new area	Economic analysis at the project level, evaluate structurally, evaluate materials	Continue to maintain, improve base, apply surface
Increased costs–maintenance, road user	Evaluate structurally, evaluate materials	Continue to maintain, apply surface

Figure 1. Evaluation factors to be considered.



Increased Maintenance and Road User Costs

As traffic increases on a road, costs of maintaining the gravel surface also increase appreciably. Whenever these maintenance costs begin to tax the budget of the governmental agency, it becomes necessary to make an evaluation before proceeding with surfacing of the road. The alternative strategies might include continued maintenance or application of a surface.

Road user costs might have the same effect on the decisionmaking process as maintenance costs. However, often there is a tendency at the county level, at least, to neglect the costs to the road user in the decisionmaking process.

FIELD INFORMATION REQUIRED

In the interest of economy, the reconstruction of a county road to a higher-type surface should, wherever practical, use the existing in-place materials. The in-place materials, however, are generally subject to wide variations in both quantity and quality. Therefore, to make the most-effective use of the existing materials, both of these properties need to be evaluated.

A list of items to consider in the evaluation is given below:

1. Geometrics--Relocate curves and raise grade;
2. Slopes--Correct landslides, clean benches, and correct fill movement;
3. Drainage--Clean ditches, improve cross fall, improve shoulders and reduce build up, and correct flooding conditions with culverts;
4. Pavement geometrics--Thickness of all components, uniformity of thickness, width, uniformity of width, and shoulder widths; and
5. Pavement properties--Quality of existing surface (grain size and plasticity), in situ strength of all components (field tests, laboratory tests, and density tests).

If the geometrics of the existing road require upgrading, this will have its effect on the decisions that will be made relative to the pavement itself. All existing deficiencies of the road should be corrected wherever possible within the economic constraints of the area. It may be necessary to relocate curves and correct grade deficiencies.

Corrective measures should be made regarding drainage of the road. This would include cleaning of the ditches, improving cross fall, and correcting flooding situations. To ensure reasonable pavement life and return on the investment of upgrading the road, all surface deficiencies that might affect the performance of the pavement should be corrected before proceeding.

Need for bridge replacement, improvement of culvert slopes, increasing drainage openings, cleaning drain pipes, and other factors many times present the major problem that confronts the policymaker in allocating funds for road improvement. In developing areas, correction of landslides, stability of fills and foundations, and other factors related to the roadway itself may be the governing consideration.

Insofar as the pavement structure itself is concerned, factors that should be evaluated include thickness and uniformity of thickness, width and uniformity of width, and shoulder width and condition. The evaluation process must consider the quality of the existing surface and in situ strength of all components along with an estimate of in-place strength values at the critical time of the year.

SCOPE OF THIS PAPER

This paper focuses on the field and laboratory evaluation of an existing gravel road that is scheduled for surfacing. We assume that the decision has been made to consider applying a surface, whether for one or for all the reasons listed in Table 1. The ultimate end point of the evaluation process is to determine whether the existing road is satisfactory for surfacing as it exists or whether some sort of improvement should be applied first. Many times the road may need to be reconstructed. The decision to surface a road is dependent on many factors, as listed in Table 1.

FIELD INVESTIGATION OF AGGREGATE SURFACES

Whenever an existing aggregate road is under consideration for surfacing, it is in the interest of economy to use the existing base to its fullest extent. Many times, the existing material can be used with a minimum amount of work. In other cases, however, stabilization or complete reconstruction may be required.

Field investigation and in-place testing of existing base materials and subgrades should be a standard procedure. To gamble against the unknown is a risky operation.

Factors of quantity and quality of the wearing surface and subgrade that should be determined are shown in Figure 1. For complete evaluation, a large number of tests need to be run; however, tests are expensive.

In field testing two different philosophies can be used. The first requires that tests be run with great accuracy so that exact values can be determined at one or two select locations. As suggested in Figure 1 and for the complete evaluation, field CBR tests should be made on both the surface and subgrade and then laboratory tests should be made on each of the materials for classification, grain size distribution, and many other properties. This approach is costly and requires a great amount of time. Often the size of the job and the required number of tests is overwhelming and the engineer in charge of the work will not make the required tests because he or she has more than enough to do to keep up with the routine duties of his or her office.

The second philosophy is based on a larger number of tests, by using quick portable devices. Research into quality control and use of statistics and design have indicated that the latter approach is the most feasible one for most situations. Admittedly some degree of accuracy at a specific location may be sacrificed over the detailed testing procedure, but the fact that a large number of tests can be made more than offsets the shortcomings of the detailed procedures.

IN-PLACE CBR TESTS

Based on the premise that quick, simplified tests are best for the evaluation of gravel roads, the Highway Extension and Research Project for Indiana Counties (HERPIC) at Purdue University has presented a series of manuals that outline techniques that can be used. In the first of these Shurig and Hittle (1) outlined simplified tests that can be used for field identification of soils and aggregates. These tests are based on standard American Society for Testing and Materials (ASTM) techniques and reliance is placed on the Unified Classification System and correlations of properties with this system.

In a second publication Shurig and others (2) outlined techniques for field investigation of county road bases and subgrades. In this latter report the tests presented in the identification manual are supplemented with field tests for determining in situ properties of aggregates and subgrades.

CBR Determinations

Three methods of rapid determination of field CBR have been evaluated at Purdue University for the counties. These include a high-load penetrometer developed by the Boeing Corporation for the evaluation of granular bases of airport pavements (3), the impact cone first developed in Australia and further evaluated in South Africa (4), and the Clegg impact device developed at the University of Western Australia (5). The Boeing cone requires a truck or other heavy equipment as a reaction; the instruments developed in Australia and South Africa are portable and can be carried by a single person. The impact cone is suitable for evaluating fine-grained materials. The Clegg meter is suitable for evaluating all materials and is particularly adaptable to evaluation of aggregate surfaces.

Dynamic Cone Penetrometer

Figure 2 shows the low-load penetrometer. It is a light portable tool used for the rapid determination of the equivalent CBR of in-place fine-grained subgrades. When the cone is driven into the soil by the drop hammer, an equivalent CBR of all soil layers may be determined. For sands the cone point is removed and a slightly rounded tip is attached. This penetrometer can measure equivalent CBR values up to 50 percent. A modification of this instrument is used for measuring bearing capacity of sands in Western Australia.

For evalution of subgrades, a hole is dug through the gravel surface. In general, two people operate the instrument, the first drops the hammer and the second reads the penetration of the cone. The penetration measured in units or millimeters per blow yields the equivalent CBR by using the calibration given in Table 2.

Normally, because one test can be run in about 4 or 5 min, a minimum of three tests are performed per hole. The rod permits determining in-place CBRs continuously from the top of the subgrade to a depth of about 4 ft below the top of subgrade. Figure 3 illustrates a typical test made at a location. Note that various layers can be detected by the slope of the depth versus CBR curve. For the example in Figure 3, a relatively hard layer is found several inches below the top of the subgrade.

Clegg Impact Device

Clegg at the University of Western Australia (5) developed a simple hand-operated device for measuring in situ CBRs of granular materials. Figure 4 shows this instrument in use. It consists of an accelerometer mounted in the head of a modified Proctor compaction hammer (10-lb weight) that is dropped from the standard 18-in height. The deceleration of

Figure 2. Diagram of dynamic cone penetrometer.

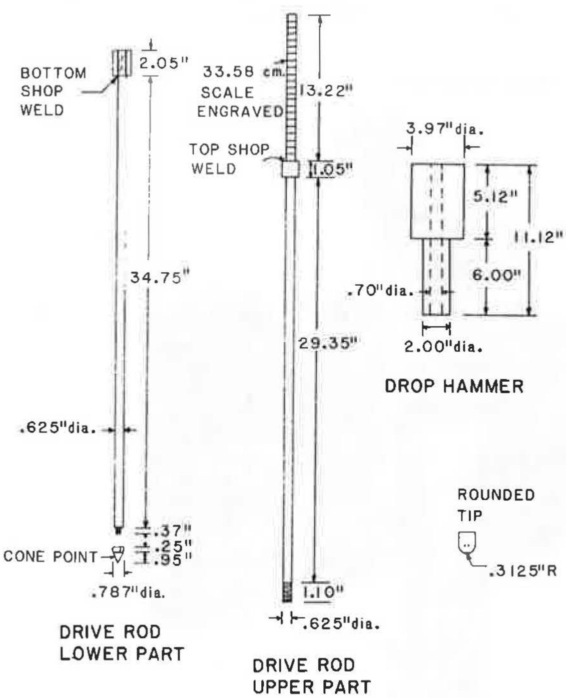


Table 2. Penetration versus equivalent CBR by using the dynamic cone penetrometer.

Cone Penetration (mm/blow)	CBR (%)	Cone Penetration (mm/blow)	CBR (%)
4	50+	16	13
5	50	18	12
6	40	19	11
7	33	20	10
8	29	23	9
9	25	25	8
10	22	28	7
11	20	33	6
12	19	38	5
13	17	45	4
14	16	60-70	3
15	14	80-90	2
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Figure 3. CBR versus depth for a given location.

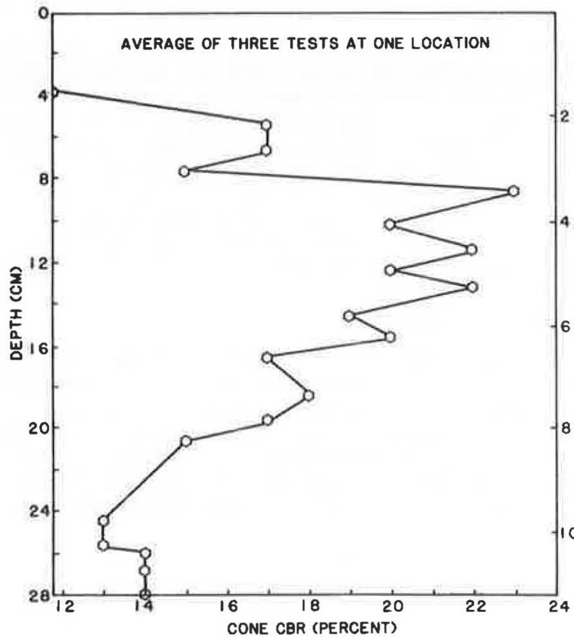


Figure 4. Clegg impact device.

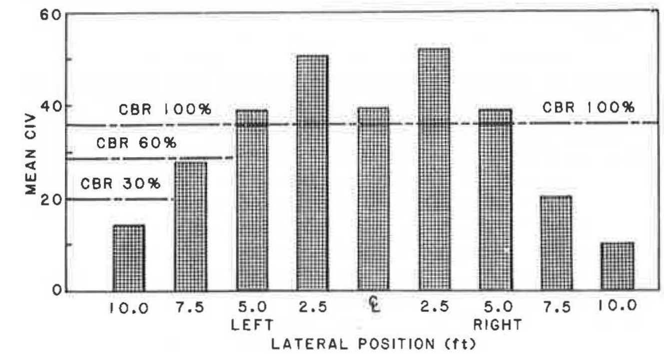
the hammer on hitting the surface is detected by the electronic instrument held in the operator's hand. The hammer is first dropped three times on the surface to ensure a firm seating before a reading is taken.

The Clegg Impact Value (CIV) can be determined in about 2 min, which permits obtaining CIV profiles at close intervals. The relation of CBR and impact values is CBR = $0.07(\mathrm{CIV})^2$.

EXAMPLE USE OF DEVICES

As a part of the research conducted for the counties

Figure 5. CBR across the transverse section of a gravel surface.



in Indiana, extensive laboratory calibrations of the instruments mentioned in previous sections were carried out. In addition, criteria for use of the instruments in the field were developed after selected pavements were tested.

To demonstrate the use of the dynamic cone penetrometer and the Clegg impact meter, data for a typical road are presented in subsequent paragraphs. The road under consideration is a gravel surface that has a nominal thickness of 6 in. The subgrade is a silty clay.

The impact value of the existing gravel surface was obtained by using the Clegg impact device at 50-ft intervals. On the transverse, readings were taken at 2.5-ft intervals, although this was varied depending on the circumstance. Thickness measurements and subgrade CBRs were made at 500-ft intervals.

A principal advantage of using a light, portable, and quick device for evaluating gravel surfaces is illustrated in Figure 5, which shows the variation of CBR with lateral position. For this graph, the longitudinal values were averaged over a uniform section.

Data in Figure 5 are typical of those obtained on most gravel roads in that the wheel tracks (2.5 ft right and left of center line) have higher values than the center and the CBR decreases rapidly toward the edge of the roadway surface. The road in Figure 5 has a usable roadway surface of about 15 ft and the drop-off of values at the edge of the pavement illustrates markedly why so many of the failures during spring break-up take place at the pavement edge.

Thickness of this particular gravel surface was about 6 in at the center line and decreased to about 2 in at the pavement edge. The gravel at the pavement edge is contaminated with subgrade material and, as is noted by the CBR values, the gravel is inadequate for a road of this type.

As a first comment, particular attention would need to be paid to the pavement edges of this surface if an asphalt surface is to be placed or even if additional gravel is to be placed on the road. The hand-carried impact device can detect this situation and can alert the engineer to the need for special attention at these locations during maintenance or rehabilitation.

Further, if this road is to be surfaced with asphalt, a minimum CBR of 80 percent or greater is required and the data show a need to recompact and reshape the road to bring it up to uniform standards.

Recommendations for this road would include cleaning the ditches, dressing up the shoulders, and applying additional material at the edges. It would probably also be necessary to add gravel to increase thickness, depending on the subgrade CBR values.

Figure 6. Longitudinal variation of CIV for a gravel road.

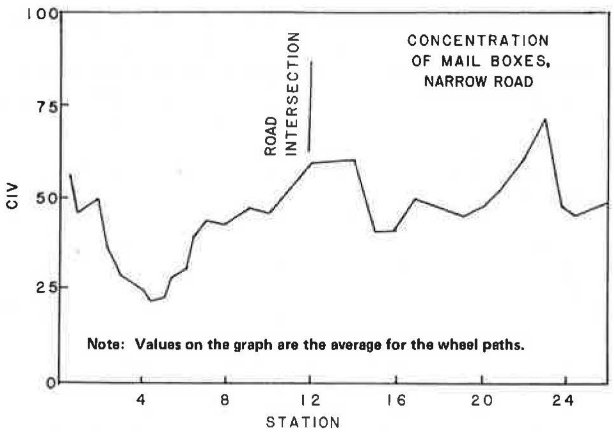
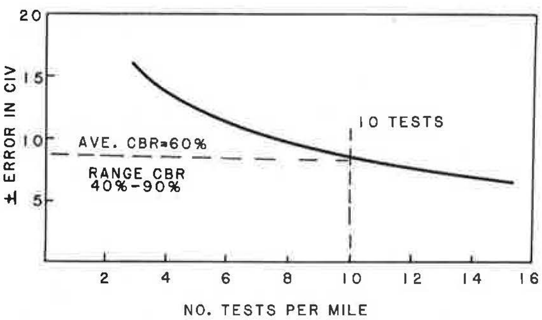


Figure 7. Required number of impact tests versus accuracy.



Longitudinal CIV Profiles

Another use to which the impact device can be put is to detect changes in material along the roadway. Values along the road can be determined by a team of two walking the center line; one operates the device and the other records data.

Figure 6 shows the longitudinal variation of impact values of the gravel surface that was depicted in Figure 5. The following features of the roadway are pertinent. From station 0+00 to station 4+00 there is a distinct downward trend in the CIV values. This drop represents CBR values from greater than 100 percent to less than 40 percent at station 4+00. At station 4+00 the values again increase to the road intersection noted at station 12+00.

Special attention needs to be placed on the section of road up to station 12+00 due to the low and erratic CBR values. Recall that the values shown are wheel track values and thus represent the critical condition. Reasons for the inadequacies need to be determined. Solutions for the problem include stabilization of this short section of road with cement or some other admixture, recompaction, or other alternatives that the engineer might consider.

Note that the detailed evaluation shown in Figures 6 and 7 would not have been possible if conventional techniques had been used.

Testing Frequency

Figure 7 shows the error of measurement of surface CIV as a function of the number of tests per mile. For 90 percent confidence and an assumed limit of accuracy of ±10 CIV, 10 tests/mile are required. At first glance this might seem like an unusually large number of tests, but recall that this is a hand-operated instrument that can be used readily. The actual time required to perform the surface tests, assuming that a complete transverse section of values is required, would be about 15 min at a site and, hence, the actual time required would be this value per site plus the time required for walking the 500 ft between test sites.

Prediction of Spring Values from Fall Values

Ideally, evaluations of this type should be done in the springtime but, due to the short time that the base is in the weakened condition, this often is not possible. Further, during the spring months highway forces are busy maintaining the roads to keep them passable and time for evaluation is rarely available.

The time situation can be circumvented by performing the tests in the summer and fall if the relation between the spring and fall readings is known with some certainty. Figure 8 shows the relation developed for the glacial gravels of central Indiana. On the basis of the reduction values shown in the figure, the spring values can be inferred through the correlations.

Effect of Base Depth and Subgrade CBR on Base CIV

Tests performed by Clegg (5) suggest that depth of base has some effect on the impact values. Likewise, the stiffness of the underlying subgrade also has an effect. Measured surface CIV of the base, depth of base, and CBR of the subgrade are interrelated when considering classification of the base under action of traffic. Soft subgrades yield under load with resultant lower densities of the base than those over stiff subgrades. Likewise, lower in situ CIVs exist at the pavement edge than under the wheel tracks. Results of this study have suggested that depth of surface has a minor effect on the test values. The analysis has indicated that the subgrade has a greater effect on the base CIV than thickness, but recall that the two are interactive and depend on lateral position.

USE OF DEFLECTION MEASUREMENTS FOR EVALUATION

Deflection measuring devices such as the Benkelman beam, Dynaflect, and the road rater can be used for evaluating gravel surfaces. The primary limitation that might exist is cost of equipment and, in certain cases, time involved in making the measurements. Figure 9 shows the results of deflection measurements made on a gravel surface for the purpose of determining corrective measures prior to placing an asphalt surface.

The Benkleman beam can be used with success if done with care. The probe must be placed firmly on the gravel surface, making certain that there are no loose pebbles in the vicinity of the probe. The data are analyzed in the conventional manner and depth of surface is determined by using techniques documented in the literature.

QUALITY CONSIDERATIONS

When evaluating any pavement for rehabilitation, it is necessary to account for the quality of the base and subbase materials, particularly from the standpoint of grain size distribution and plasticity. A danger exists that little or no consideration might be given to the quality of the existing gravel. If low values are found in the field, is the existence of a low CIV on the gravel due to high plasticity, low thickness, or some other factor? In the final

Figure 8. Prediction of mean spring CIV from the mean fall CIV.

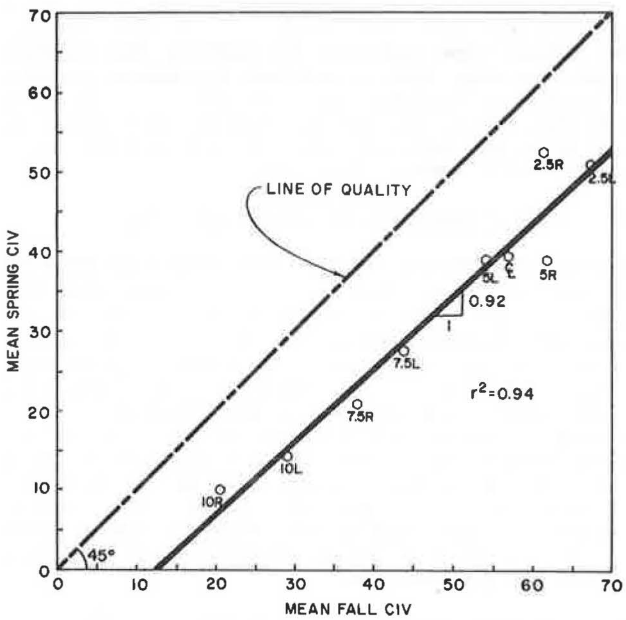
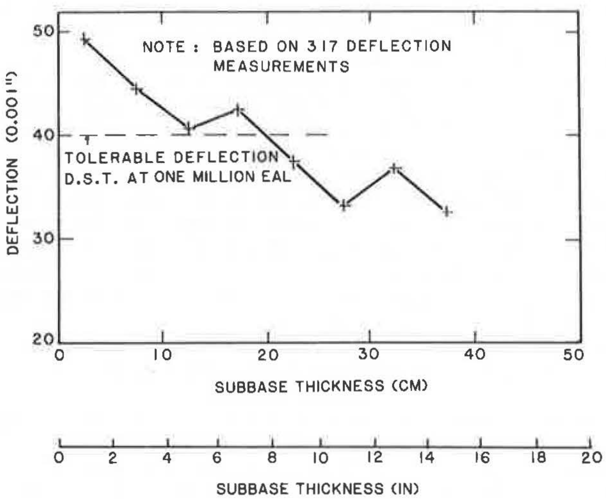


Figure 9. Deflection versus subbase thickness for a gravel road.



analysis, samples must be taken to the laboratory and tested, unless information is available relative to the material itself.

LOCATION AND SPACING OF TESTS ALONG ROADWAYS

The wearing surface and subgrade materials in unpaved county roads are subject to variations in depth and quality of base and variations in the nature of the native subgrade soils. Therefore, a sufficient number of field samples and tests must be made to develop a picture of the range of variations that exist in a given section or roadway.

The complete CIV-transverse section of the roadway should be evaluated at periodic intervals along the length of the road project. The number of tests required is about 10/mile, which sets the frequency of testing at about every 500 ft. One depth measurement and one subgrade CBR value should be taken at various points on each transverse section.

APPLICATION OF FIELD TEST RESULTS

The ultimate use of the field test is to design a road improvement. To accomplish this, it becomes necessary to fall back on standard methods of design of pavements by using documented methods commonly in use. The subgrade CBR will give a check on the required thickness of the finished pavement. The CIV of the wearing course will suggest the quality of the material and whether it should be upgraded by stabilization or whether it is satisfactory as it exists at the present time. For the project illustrated in Figures 5 and 6 the base material has a potential design CBR of >100 percent and it is considered satisfactory for this class of road. If very heavy traffic would be expected, this material should be stabilized or otherwise improved to a minimum CBR of 100 percent for the entire cross section.

Wearing surface materials that have a reasonably uniform in-place CBR value and a reasonably uniform width and depth can usually be incorporated directly into the new pavement design. However, for the illustrated case, additional depth and width of base material are necessary to upgrade the base to meet the pavement design.

For purposes of evaluation, the values in the wheel tracks should be used for redesign and determination of quality and those at the pavement edge should be used as indicators of added needed work that might be carried out at the pavement edge.

SUMMARY

One of the problems that faces the county highway systems in the United States is the time required to make a true evaluation of the road before decisions are made relative to surfacing. This paper presents conceptually the philosophy that, rather than spend a great deal of time on expensive tests at just several locations, it would be better to use simplified test procedures wherein a large amount of data can be obtained in a relatively short period of time. Techniques currently available, which are based on sound engineering principles, can be adopted for this purpose.

This paper has presented concepts for using simple instruments for evaluating gravel surfaces. The utility of these devices lies in their portability and the ability of the investigator to make a large number of tests on which to base conclusions. A word of caution, however, is presented in that a sufficient number of quality tests must be made relative to grain size distribution and plasticity of the base before a solution is arrived at.

The need for providing for adequate drainage and other environmental factors cannot be overstated and it is axiomatic that these be included in the evaluation. It is believed that the use of rapid testing techniques in the field offers the opportunity for the engineer to arrive at an economic redesign of existing gravel surfaces. Testing of the type outlined will suggest alternatives such as application of an asphalt surface, stabilization of the existing base, recompaction and reshaping, or other alternatives that are at the engineer's disposal.

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REFERENCES

1. D.G. Shurig and J.E. Hittle. Field Identifica-

tion of Soils and Aggregates for County Roads. Highway Extension and Research Project for Indiana Counties, Purdue Univ., West Lafayette, IN, County Highway Series No. 13, Dec. 1971.
2. D.G. Shurig, J.E. Hittle, E.J. Yoder, and W.H. Goetz. Field Investigation of County Road Bases and Subgrades. Highway Extension and Research Project for Indiana Highways, Purdue Univ., West Lafayette, IN, County Highway Series No. 14, Jan. 1977.
3. High Load Penetrometer Soil Strength Tester. Commercial Airplane Division, Boeing Company, Renton, WA, Document D6-2455, 1971.
4. D.J. VanVuuren. Rapid Determination of CBR with the Portable Dynamic Cone Penetrometer. The Rhodesian Engineer, 1969.
5. B. Clegg. An Impact Device for In-Situ Base Course Evaluation. Univ. of Western Australia, Nedlands, 1975.

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Surfacings for Low-Volume Roads in the Third World

RAY S. MILLARD

This paper is a review of planning criteria for use in deciding when to provide low-volume roads with permanent surfacings and appropriate techniques for asphalt surfacings on such roads in the third world. Planning criteria involve benefit/cost studies, changes in road user costs (mainly in vehicle operating costs) balanced with the costs of the road improvement, and subsequent changes in road maintenance costs. Pavement deterioration under traffic is influenced considerably by climate, by the road-making materials and methods employed, and by the care taken in pavement maintenance. The choice of bituminous surfacing technique has a considerable influence and the thesis developed is that the pursuit of high stability and good load distribution properties, both desirable in bituminous mixtures on heavily trafficked roads and airfields, has militated against the flexibility that is a prime requirement for asphalt surfacings on low-volume roads. Types of surfacing with this property of flexibility are reviewed.

As the building of main road networks of Third World countries is moving toward completion, emphasis in their road-building programs is changing toward the construction, improvement, and maintenance of secondary and feeder roads that are being built as an essential component of agricultural development schemes. Most of these feeder roads carry very light traffic and are built of earth and gravel. The secondary roads often make up more than half of the total network and generally carry traffic within the range 30-500 vehicles/day. Most of these secondary roads also have surfacings of earth or gravel and there are great pressures to provide them with permanent bituminous surfacings. These pressures are generated by the desire to maintain a free and unencumbered passage for vehicles in wet weather and to remove the nuisances of dust and corrugated bumpy surfacings in dry weather. They are often exacerbated by the incapacity of local road maintenance organizations to provide even minimum routine maintenance on the earth and gravel roads. This paper has two objectives; one is to review criteria for deciding when it is desirable to provide such roads with permanent surfacings of asphalt or concrete and the other is to consider appropriate techniques for bituminous surfacings on such roads.

CRITERIA FOR PROVIDING ROADS WITH PERMANENT SURFACINGS

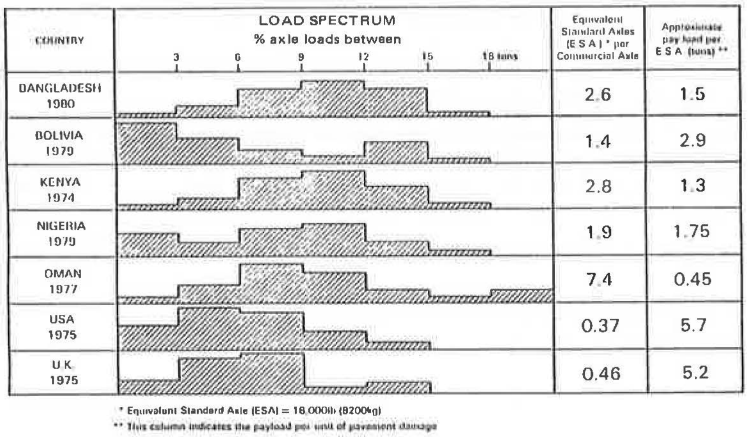
To the transport economist the criterion for providing roads with permanent surfacings is that of minimizing total transportation costs. Vehicle operating costs are normally reduced substantially following the provision of a permanent surface and the calculation is usually undertaken as a benefit/cost study, with the benefits expressed as a rate of return on the investment necessary to upgrade the road. To the politician the criterion may lie at least partly in the perceived gratitude of constituents. Between them there is the uneasy seat of those charged with allotting scarce capital resources among all the different heads of government expenditure. Our stand must be with the transport economist, and in taking this stand we are not functioning as lobbyists--we are seeking to provide a factual basis for making wise decisions on what resources should be devoted to road works and how these resources should be most effectively used.

The basis for benefit/cost calculations concerning the upgrading of roads from earth or gravel to bituminous-surfaced is well established. The basic information required for such calculations consists of data on traffic flow and predictions of likely future growth on costs of the various procedures of road construction and maintenance, on pavement deterioration under the combined effects of traffic and weather, and on the influence of road conditions, particularly of pavement roughness on vehicle operating costs. Typically, in open African savannah such calculations will indicate that rates of return exceed the opportunity costs of capital with present traffic flow between 150 and 400 vehicles/day. Somewhat lower traffic thresholds are to be expected in hilly country and in wet climates. It is always desirable to make sure that local circumstances have been fully considered in assembling the data and in the assumptions that have to be made. Some of the pitfalls are indicated briefly below.

The data on traffic flow must include a realistic estimate of the spectra of vehicle and axle loading. Enforcement of vehicle-loading regulations is generally weak in Third World countries. Overloading beyond the rated capacity of trucks is commonplace and the axle loading spectra are frequently heavier than on main roads in industrialized countries (Figure 1). There is evidence that the fourth power law derived from the American Association of State Highway Officials (AASHO) Road Test--that damage to a pavement is proportional to the fourth power of the wheel load--applies at least approximately on lightly constructed bituminous pavements. Designs on the assumption that legal loading limits are not exceeded can lead to pavement deterioration 10 times more rapid than was intended.

Construction costs will vary from one locality to another. At one locality they will be affected

Figure 1. Typical axle load spectra for commercial vehicles.



considerably by the geometric standards selected for the paved roads and by the extent of road widening and relocation deemed to be necessary. Computer programs are available for the rapid comparison of the rate of return as it is affected by changes in standards of geometric design. The British road transport investment model (RTIM), based on experience in East Africa (1), can be used to compare designs with different specified standards. The World Bank's highway design and maintenance model (HDM) (2) has the potential to make optimum use of specific design components.

Whether such studies are undertaken manually or with the assistance of computer models, it is essential that the relations employed be validated by local experience. This applies with the relation used between vehicle operating costs and road conditions and even more so with the assumptions made on pavement deterioration.

Information on the effects of road conditions on vehicle operating costs is sparse, particularly on the most relevant aspect for this type of study--the effects of road roughness on vehicle operating costs. In effect, at this time only three sources are used: the classic study by Moyer and Winfrey, completed in the 1930s; a synthesis of data and judgment by de Weille published by the World Bank in 1966 (3); and a study undertaken by the British Transport and Road Research Laboratory (TRRL) in cooperation with the Kenya Ministry of Works and the World Bank in the early 1960s (4). Further studies, with World Bank and United Nations Development Program (UNDP) assistance, are in progress in Brazil and India, and these should soon yield useful data. Even on roads that carry no more than 100 vehicles/day, the investment in providing and operating the vehicles considerably exceeds the investment in construction and maintenance of the road. It is odd to find so little research undertaken on the effects of road conditions on vehicle operating costs, particularly in view of the huge and continuing investment in research on pavement design.

Pavement deterioration is affected by many things. Traffic loading is of course always an important factor. On earth and gravel roads the nature of the road material, the climate, and the frequency and nature of maintenance are important influences. These are often known as matters of local experience; but they need to be made numerate in order both to plan the maintenance of such roads in the most effective way and to produce credible cost/benefit studies to establish when the provision of a permanent surface is warranted.

The system can be beaten in various ways, some more laudable than others. For instance, a district engineer who is both competent and cunning will concentrate efforts on improving the drainage and layout of a road and perhaps neglect the maintenance of the gravel running surface. He or she thus generates pressures for the upgrading of the pavement (by both popular clamour and inflated vehicle operating costs) and at the same time completes work that will reduce the capital cost of the upgrading. In the Third World such cunning is as yet rare. It is more common to find highway departments struggling, often with indifferent success, to build up effective road maintenance organizations.

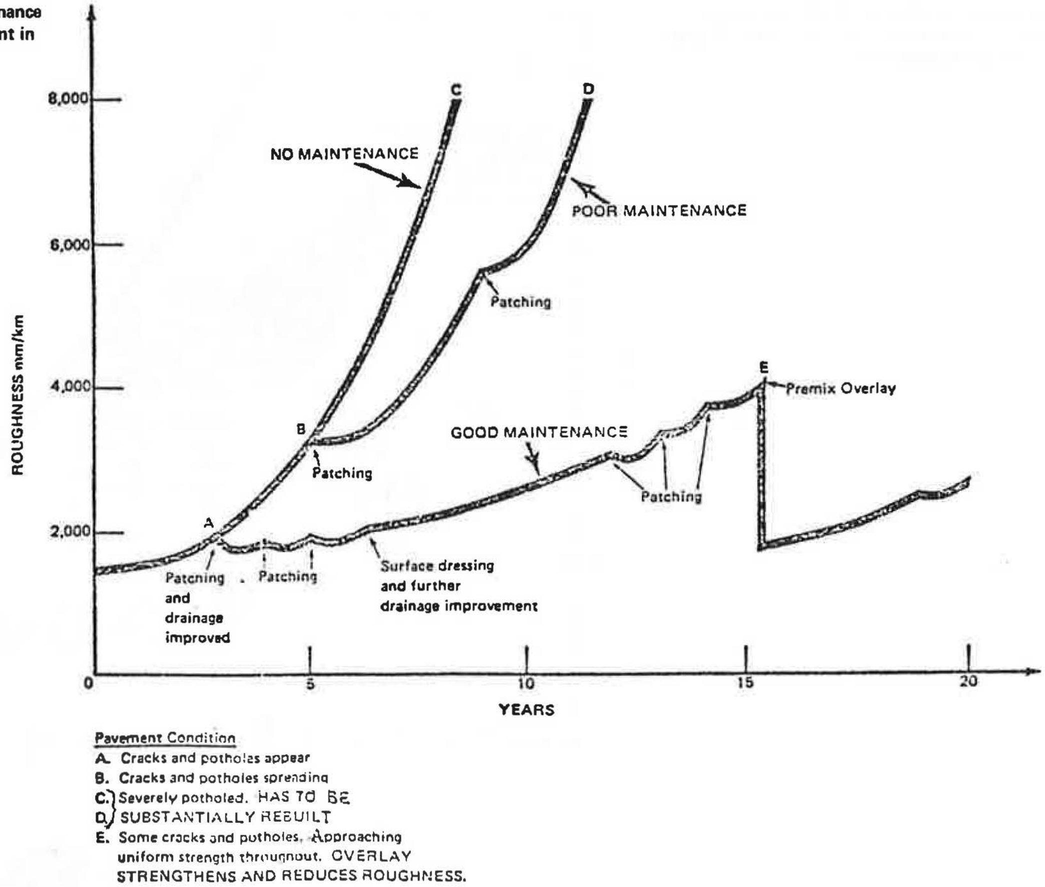
Bituminous pavements are normally designed with the intention that the next major capital investment in strengthening, and possibly widening, will not be required for some 10-20 years. Normal routine maintenance is presumed, and on more lightly trafficked roads that have lightly constructed pavements this maintenance inevitably involves the correction of some faults and omissions in construction; for instance, to correct weaknesses in drainage that become apparent and to repair potholes where construction faults occur. Figure 2 shows the effects of different maintenance policies on pavement deterioration.

The efforts made by both governments and aid donors to build up effective road maintenance organizations have not as yet been markedly successful and it is evident that the neglect of maintenance is leading to considerable waste of resources in both inflated vehicle operating costs and increased expenditure on the roads. This poses a dilemma that is becoming more serious as the focus of effort moves from main roads to secondary roads. We cannot expect the relatively high technologies of detailed soil and hydrological surveys of risk analysis in design and careful control during construction to be commonly available in improving and maintaining secondary roads. With lower levels of technology used in construction, it will be increasingly difficult to keep such roads in serviceable condition. The answer that is sought must be simple and more robust.

BITUMINOUS SURFACINGS

The evolution of bituminous surfacings has taken different paths in different parts of the world. For instance, the paramount bituminous premixes in the United States (asphaltic concrete), Great Britain (rolled asphalt), and the Federal Republic of Germany (Gussasfalt) are different materials in both composition and performance. The easy explanation that they were evolved to meet different needs conceals a fascinating story that is too long to

Figure 2. Effects of maintenance on light bituminous pavement in wet or monsoon climate.



recount here. The relevant point is that the Third World has not had the time or the necessary pressures for bituminous surfacings to be evolved that are manifestly adapted to their needs. They have had to use imported technologies, sometimes inherited from a colonial power, more often nowadays imported by foreign consultants who use experience from their home countries.

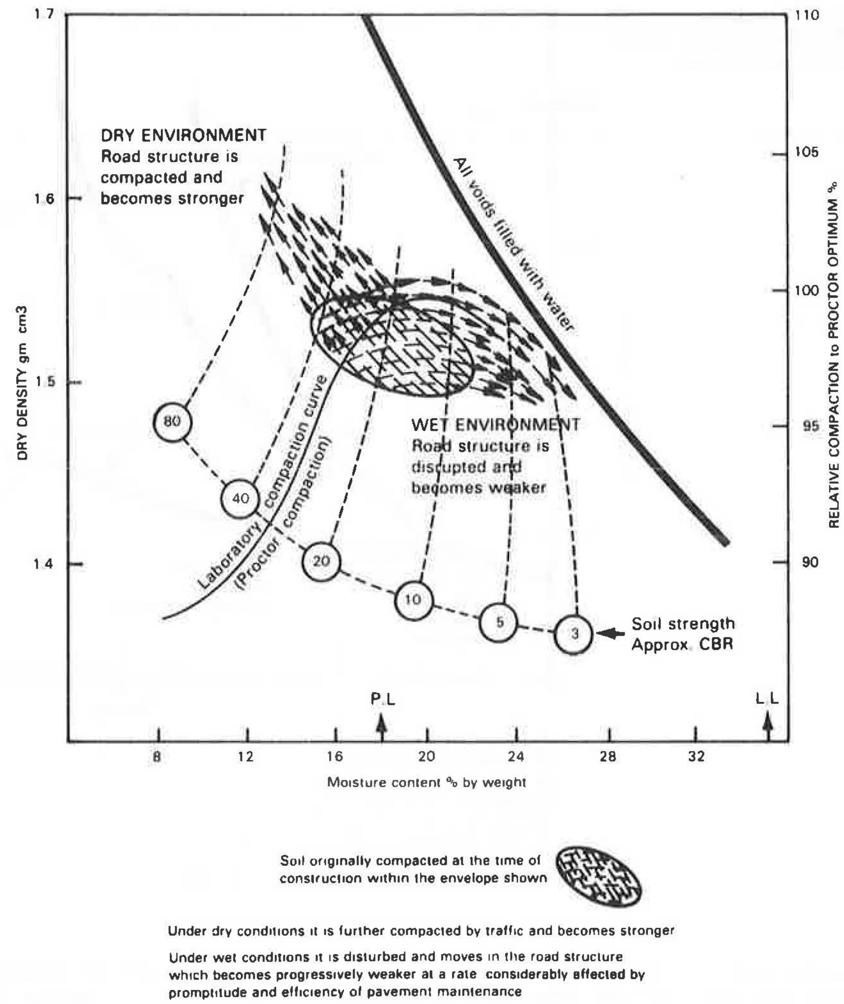
This can produce some odd anomalies. For instance, in Egypt bituminous roads are surfaced with asphaltic concrete and the same material is used for pavement maintenance. Surface dressing (chip and seal) is practically unknown. Twenty degrees of latitude to the west, in Tunisia, almost all the bituminous roads are surfaced with single- or double-surface dressings and the pavements are maintained by further surface seals by using bitumen emulsions. Both countries are evolving bituminous surfacing techniques to suit their needs, but in neither country have local skills evolved to the level that they are able to take full advantage of the variety of bituminous road surfacings. So there are two problems, one to decide which forms of bituminous surfacing are inherently suitable for use on roads in a particular country and the other to establish in the country a corps of people who have the necessary skills in the appropriate road-surfacing processes.

Physical environment has a considerable effect on this choice and the two predominant influences are those of climate and availability of road-making materials. In a dry climate pavements will tend to get stronger under the compacting action of traffic. In a wet climate they may get weaker (Figure 3). An equable climate (i.e., with only very small diurnal and seasonal temperature changes) favors the use of concrete pavements. A climate with large temperature changes is inimical to concrete because an important factor in the deterioration of concrete pavements is thermal expansion and contraction of the concrete in association with traffic loading.

Some countries must adapt their road-making methods to a limited choice of materials. At the extreme are places such as Malta, Bermuda, and some Caribbean islands where the only available material is the soft limestone from which the islands are made. The Deccan area of India is a vast plain of expansive clay without resources of rock for road-making. Similarly, coastal areas of Bangladesh are some 200 miles from the nearest supplies of rock for crushing and in both these areas bricks are used extensively in road-making. In Africa the quality of the indigenous lateritic and other road-making gravels varies considerably between different areas as does the access to rock suitable to be crushed for road works. All these factors have an influence on the choice of techniques for using bituminous materials in road-making, so much so that there are grave risks in attempting to generalize.

Some generalization will be useful, and the first generalization to make concerns the function of a bituminous surfacing on a low-volume road. The intrinsic strength and load-spreading properties required on lightly trafficked roads will normally be provided in the road base and subbase. The functions of the bituminous surfacing are to resist the abrasive action of traffic and to prevent rain from entering the road structure. The primary properties required are stickiness so that it adheres well to road-making aggregates and extensibility so that it can accommodate strains without rupturing as the pavement moves under traffic loads.

Figure 3. Density, moisture content, and strength illustrated by typical sandy clay; plastic limit is 18 percent, liquid limit is 35 percent.



Stickiness can be taken for granted for the time being as one of the recognized attributes of bituminous materials; extensibility needs some further explanation.

The term flexible pavement has long been synonymous with bituminous materials and the word flexible implies extensibility. But, in the search for stronger pavements to accommodate heavier and heavier traffic on roads and airfields, development has moved in another direction and emphasizes stiffness, rigidity, and load-spreading properties as desirable attributes rather than flexibility. In the structural methods of pavement design that are pursued so assiduously high stiffness is sought, with the limiting criteria being defined in terms of critical stress. In the United States the generic name for dense premixed bituminous materials, asphaltic concrete, implies the move from flexibility toward rigidity as a desirable attribute. There is no doubt that this is the right direction in the evolution of heavy duty pavements. But, is it right for the lightly constructed pavements that we have to use? I think not.

In lightly constructed pavements strains develop in the surface layer for many reasons--elastic deformation of the road pavement under traffic loads, permanent deformation because of the compacting action of traffic and, often, more deep-seated movements associated with changes in the moisture regime under roads. Assiduous control of compaction during construction can go some way to reduce these strains but will not eliminate them. In any case, such assiduous control is not likely to be the normal rule in improving secondary roads in the Third World. In providing the initial bituminous surfacing and in subsequent strengthening and surface maintenance, a primary requirement is that the surfacing should be able to accommodate strain without cracking or raveling. Where does this take us in selecting appropriate forms of bituminous surfacing for such roads?

We can get little guidance from the fund of knowledge now available on pavement design. Indeed, concentrating as it has on producing strong pavements for heavily trafficked roads, it may be misleading or inadequate. The aim for high stabilities has produced a trend toward bituminous mixtures in which the bitumen is present in the form of very thin films between tightly packed crushed stone, thus high stability is obtained at the expense of an ability to accommodate strain. The literature is growing on the behavior of dense asphaltic concretes under fatigue loading; it provides some guidance on how to design these mixtures to accommodate transient strains. But, these studies have not included asphaltic surfacings in which the bitumen is present in fairly thick films as these are regarded correctly as intrinsically unsuitable for use on heavily trafficked roads and airfields. And it is these surfacings in which the bitumen is present in fairly thick films that retain the flexibility necessary for use on low-volume lightly constructed roads.

What types of asphaltic surfacing are these? There are broadly two types of surfacing in which relatively thick films of bituminous binder are used:

1. Surface treatments, in which a film of asphalt is applied to the road surface followed by an application of chippings and
2. Mixtures of asphalt and coarsely graded aggregate.

The mixtures can take many forms, from grouting through mix-in-place, to premixes of the open-textured macadam type or denser gap-graded mixtures. All have the attribute that they are able to accept high transient and permanent deformations without cracking or disintegrating.

The two forms of treatment are complementary rather than alternatives. The normal sequence will be to use surface dressing as the first bituminous surfacing on a newly constructed bituminized low-volume road and then, when the road comes to be strengthened or the riding quality improved, to employ a bituminous mixture, either grouting or a premix.

Surface Dressing

Essentially, surface dressing involves application of a uniform film of asphalt to the prepared road base followed by a layer of single-sized stone. The asphalt seals and binds the road surface and the stone provides the running surface. Little quantitative evidence has been gathered on the rate of pavement deterioration of surface-dressed low-volume roads in comparison with such roads surfaced with other bituminous materials. Road Note 31 (5) tentatively suggests that a surface-dressed road will tolerate three times the transient deflection under load and still give the same design life as a similar pavement surfaced with 2 in of asphaltic concrete and there are many examples in practice of roads that consist of surface-dressed granular bases that give good service for many years, with traffic up to and sometimes in excess of 1000 vehicles/day.

A large body of literature gives practical recommendations on how to make good bituminous surface dressings and this paper will not replicate these or even attempt to review them.

Considerable skill is needed in choosing the appropriate specification for given circumstances. The stone chippings should be single-sized and the appropriate size must be chosen in relation to the type of road surface being treated and the traffic--large chippings for soft surfaces and heavy traffic, smaller chippings for harder surfaces and lighter traffic. In double-surface treatments it is appropriate to use two different sizes of chipping for the two treatments so that one can be bedded firmly in the interstices of the other. Opinions vary as to order in which the chippings are to be applied, and this is a point of some subtlety. If for instance the two treatments are to be applied one shortly after the other, it may be best to use the larger chipping (say 3/4-in nominal size) first followed by a further seal and application of a smaller chipping (say 3/8-in nominal size). But, if for any reason the second treatment is to be delayed, a chipping of medium nominal size (say 1/2 in) would probably be best for the first dressing.

Penetration-grade asphalts are frequently used for surface dressing in hot countries. In many such countries road temperatures can be quite low, especially in the early morning, and in these circumstances it is increasingly common practice to make the asphalt more fluid by incorporating small proportions of an oil of medium volatility such as kerosene. If this is done, it is vital that instructions be simple and effective to avoid the risk of fire.

Probably the most important factor in influencing the performance of surface dressings is that the rate of spread of asphalt be even and appropriate for the circumstances. Again, this is a point of some subtlety; the rate of spread of asphalt must be chosen in relation to the absorptive characteristics of the surface being treated, the size of the chippings, and the expected traffic intensity. One of the earliest attempts to rationalize this choice was in Hansen's concept of the average least dimension of the chippings to indicate the thickness of asphalt film that is appropriate for the size and shape of the chippings (6). Most current specifications embody this principle (7-10).

Even so, knowledge on how to specify and lay surface dressings has been derived entirely from carefully observed practical experience. It is none the worse for that. But one result is that it attracts little academic attention either in teaching or research, and many highway engineers never have the opportunity to learn the intricacies involved in securing good surface dressing.

The manufacturing process for surface dressing is undertaken on the road. This militates against the careful control that is necessary to produce a good product, in contrast with premixed bituminous materials in which a large part of the manufacturing process is carried out in a stationary plant on which control of the process is somewhat easier. This is not a disabling disadvantage; witness the consistently high quality of surface dressing done in Australia and New Zealand and the good service from them obtained as a result. But, it is the reason why surface dressing falls into disrepute in some countries where site control has been indifferent. One reason for using double surface dressings in new construction is that the second dressing corrects imperfections that arise from indifferent workmanship in the first. And variants exist that have the same objective, such as the Cape Seal used extensively in southern Africa. In this process the surface voids in the finished surface dressing are filled with an application of coated grit. Special machinery and skilled staff are needed. In industrialized countries these are often provided by specialist firms, a trend that should perhaps be encouraged in Third World countries.

Grouting

Grouting was one of the earliest forms of bituminous surfacing to appear. In it the upper layers of a crushed stone base are grouted with applications of liquid asphalt and the surface voids are then filled with fine aggregate. This process was done by hand, aided by simple machinery often little more than a roller and a simple hand sprayer. Though it did produce a robust surfacing capable of accommodating quite a large strain, construction was tedious and it used a lot of asphalt. In its semi-grout form, in which the aim is for the asphalt to penetrate no more than 2 in, it remains a possible surfacing in areas where skilled hand labor is readily available. It is still in regular use for instance in India, Bangladesh, and Indonesia. But it is subject to vagaries in skill of the workers. Both the spreading of the stone and the application of the bitumen can be mechanized and thus improve output and uniformity. The surface normally has it be sealed with a surface dressing to render it waterproof.

Mix-in-Place

Many thousands of miles of rural road in North America were first surfaced with mix-in-place bitu-

Table 1. Typical bitumen macadams.

Item	Open-Textured Wearing Course, Crushed Rock[a]		Single Course[a]		Open-Graded Base Course, Crushed or Natural[b]	
	Sieve Size (mm)	Percent Passing	Crushed Rock Percent Passing	Gravel	Sieve Size (mm)	Percent Passing
Aggregate	50		100	100		
	37.5		90-100	95-100		
	28		55-90	60-80	25	100
	20	100			19	90-100
	14	90-100	35-55	40-60		
	10	55-75			9.5	20-55
	6.3	25-45	20-30	30-45		
					4.75	0-10
	3.35	15-25	10-20	20-35		
					2.36	0-5
	300	2-10	2-10	5-15		
	75	2-6		2-6	75	0-2
Bitumen content (% total mix by weight)		4.8 ± 0.5, 4.6 for limestone	3.9 ± 0.6, 3.6 for limestone	5.0 ± 0.6	Determined from trial mixes	

Note: Binder for the open-textured wearing course and the single course was medium curing cutbacks and 300 penetration for hand laying; medium curing cutbacks and 300 and 200 penetration for machine laying. Binder for open-graded base course was medium setting, high float medium setting, and cationic medium setting.

[a]From British Standard 4987, coated macadam for roads and other paved areas.
[b]From the Asphalt Institute (7).

minous surfacings, and it is surprising that the process has found little use in the Third World. Perhaps one reason is that roads in developing countries have usually been designed as major new investments rather than being built by stage construction methods. (How would a mix-in-place surfacing be treated in a pavement design analysis?) The process employs natural gravels or sandy soils and cut-back asphalts or slow-breaking emulsions, the materials being mixed in place either with blade graders or with traveling mixers. When slow curing cutbacks are used, the mixed materials remain workable for a considerable time and a bumpy damaged surface can be restored by scarifying and remixing. Such was done, for instance, in Scandinavia in the 1950s by using heavy road oils and glacial gravels to provide thin pavements that could be restored after winter frost heave. It seems likely that this process has application on more lightly trafficked roads in regions of the developing world where there are readily available sources of sandy and gravelly soils of low plasticity.

Bitumen Macadams

Bitumen macadams materials evolved from grouted macadam following the realization that it is more convenient to coat the stone in a mixer than on the road. For some years such mixtures were marketed under proprietary names, but as experience became codified standard specifications were evolved for a wide variety of mixtures that go under this generic name. Essentially they are mixtures of coarse aggregate, fine aggregate, filler (mineral dust), and bitumen. The fine aggregate is not present in sufficient quantity to fill the voids in the coarse aggregate. As an approximate rule, mixtures that contain less than 15 percent passing a No. 8 American Society for Testing of Materials (ASTM) sieve are designated as open-textured bitumen macadams and those with 15-30 percent passing a No. 8 ASTM sieve are designated as a medium-textured.

The original concept of these materials has been modified by the introduction of dense bitumen macadams that have continuous gradings and are nearly but not quite asphaltic concrete. Here we are concerned with the open- and medium-textured macadams. The surface area of aggregate per unit volume is low in comparison with a dense mixture. Hence, with approximately the same amount of bitumen per unit volume the film of bitumen over the aggregate is considerably thicker. The filler has a special function: intimately mixed with the bitumen it produces a binding matrix that is more viscous and greater in volume than the bitumen itself. These mixtures are normally made with cut-back bitumen or with relatively soft penetration-grade bitumen. They can be made with unheated aggregate by using bitumen emulsions. They have two attributes that make them attractive for use in strengthening or correcting the profile of bituminous-surfaced low-volume roads: They are capable of accommodating large strains without fracture and, at least on lightly trafficked roads, they are more tolerant of deviations from specification in grading of aggregate and bitumen content than are dense mixtures. They are also cheaper.

As with surface dressing, specifications have been derived from closely observed field experience and there is no numerate method based on physical tests for designing them. Specifications are based on the grading of the aggregate and the viscosity and proportion of bitumen. Typical illustrative specifications are given in Table 1.

Again, these mixtures have attracted little academic attention in either teaching or research. The absence of a regime of physical tests to design and evaluate them has given such mixtures a psychological disadvantage. A generation of road engineers had been trained on the physical testing regime associated with asphaltic concrete. When confronted with these macadams, the first question they ask concerns the Marshall stability and flow values and the voids contents of such mixtures. When told that they cannot be evaluated in this way, the frequent reaction is to dismiss such mixtures as outlandish and absurd.

Bitumen macadams have a disadvantage in that they are not impervious. This can, under certain circumstances, be turned to advantage as in the use of open-textured macadam as an antisplash, antiskid surfacing. With the American aptitude for turning a happy phrase, these are known as popcorn surfaces. This permeability is not a serious disadvantage when they are used to cover an intact and well-sealed surface. If there is danger that water will penetrate through them to damage the old pavement below, they should be sealed soon after laying with a surface dressing. Indeed, the single-course bitumen macadam (Table 1) is a mixture specifically intended for use on low-volume roads and normally intended to be surface-dressed within a year of laying. Initiates will recognize from the designated composition that this material is cheap and relatively easy to mix and lay.

GAP-GRADED DENSE MIXTURES

Early asphaltic surfacings were frequently made with natural sand. These sand asphalts proved to be extremely durable. They were the basis for the cookery book recipes for premixed surfacings produced by Richardson in the early years of this century (11) and some can still be seen on city streets in the United States and still give good service. But sand asphalts passed out of general use for two reasons: one was the very practical reason that premix producers wanted to use the fines produced in their stone crushing plants and the other was that the use of crushed rock fines proved an advantage in gaining the high stabilities sought in the Marshall testing regime.

In Great Britain another evolutionary line was followed. With sand readily available in most parts of the country, producers continued to use sand asphalts but modified them by incorporating coarse aggregate like plums in a pudding. The resultant mixtures, known generically as rolled asphalt, were dense and impervious--an essential virtue in a wet climate. The coarse aggregate enhanced their resistance to deformation under traffic, though typical mixtures do not have the stability associated with asphaltic concrete. And the sand-asphalt matrix conferred flexibility on the mixture so that it would accommodate strains in pavement structure without cracking.

The next advance with this type of dense asphalt occurred in South Africa in the early 1960s with the development of mechanical testing regimes for their design (12). Stimulus to this development came from the opportunity it gave to employ the slurries from the gold mines, finely ground quartzitic rock, in the manufacture of this dense asphalt. At the same time, the move began in Great Britain to explore the use of mechanical tests for designing these mixtures and the Marshall testing regime is now tentatively in use alongside the proved recipe specifications (13).

These mixtures are mentioned for two reasons. One is the practical one that in many parts of the Third World there are available deposits of natural sand that can be used to make dense and potentially very durable premixed asphalt surfacings. Alluvial and aeolian quartzitic sands, coral sands, and fine volcanic scoria are examples. Where coarse aggregate is available this can be incorporated both to enhance stability and to reduce cost.

The second reason is technical, that where high stability is not a prime requirement such gap-graded mixtures may have advantages in being easier to mix and lay and in producing durable dense asphalts that are not prone to crack so readily when subject to strains as does continuously graded asphaltic concrete. It would have been good to be able to present this thesis fully supported by quantified data; unfortunately, I cannot do so. When I discuss this thesis with asphalt technologists it finds general acceptance. But they have been so busy solving the design problems of heavily trafficked roads and airfields that they have not had time to devote to the development of dense premixes for low-volume roads. I conclude this section by expressing the hope that this paper will prompt more attention to the development of robust and cheap dense asphalts for surfacing low- and medium-volume roads.

CONCRETE ROADS IN THIRD WORLD

In most developing countries the building construction industry absorbs nearly all available supplies of portland cement concrete. Small quantities are used for making soil-cement road bases and bridges but concrete roads as such are as yet little used. In some countries supplies of cement are increasing and price trends are moving in favor of concrete roads. This trend is accentuated if life-cycle costing methods are used because the maintenance needed on a well-made concrete road is normally less than on a bituminous road. This factor is of particular significance in those developing countries that are finding difficulties in building effective road maintenance organizations.

Another potential advantage for roads in the secondary network is that concrete roads are well adapted for construction by skilled and semi-skilled labor and a minimum of machinery. In fact, many examples exist in Europe and America of secondary roads and estate roads built of concrete by hand methods in the 1930s that are still giving excellent service. The advent of large and complex machinery has produced enormous increases in speed of construction and output per person. But, there has been little comparable increase in quality. The problems of joint construction in concrete laid by machine have not been solved satisfactorily. In concrete construction by hand, joint installation is a simple component well within the capacity of the carpenter or other worker who lays the formwork.

A further influence lies in climate. In some regions of the Third World diurnal and seasonal temperature changes are small, for example coastal areas in East and West Africa and the deltaic areas of many large rivers elsewhere, and large peninsulas or islands such as Malaysia, Indonesia, and the Philippines influenced by oceanic climates. In these regions concrete has a technical advantage because the deterioration of concrete pavements is very much influenced by thermal expansion and contraction, manifested by cracking and trouble at the joints. Concrete roads also have an advantage in construction over poorly consolidated soils in that the loads of pavement traffic are equitably distributed over the soils. An example that demonstrates both of these advantages lie in the heavily trafficked road that connects Bangkok with the international airport at Don Muang and the north of Thailand. Here a 20-cm reinforced concrete slab supported by a stabilized subbase lies on some 15 m of unconsolidated silty clay. Some settlement has occurred but the settlement has been even, and apart from work to correct the profile approaches to piled bridges and culverts, the pavement has needed practically no maintenance since it was constructed in the early 1960s.

CONCLUSION

I did not intend to produce a panegyric for concrete roads; the intention was more to illustrate the general thesis of this paper, which by now should be evident. This can be expressed in three propositions.

1. The trend in every country is to evolve toward road-building methods that are optimum for the local environment. These methods vary considerably to reflect both the nature of the available road-making materials and the climate. They may change to reflect changes in the economic and social life of the country.

2. Although countries of the Third World have benefited greatly from the importation of road-

making technologies from industrialized countries, this has sometimes distorted the evolution toward road-building methods that are appropriate to the local environment.

3. The effects of climate and of the nature of available road-making materials on road-building methods are now generally understood; however, this knowledge still needs to be refined and there is the opportunity for countries of the Third World to define the way in which their road-building technologies should be evolving and to make a conscious effort to promote this evolution.

REFERENCES

1. R. Robinson, H. Hide, J.W. Hodges, J. Rolt, and S.W. Abaynayaka. A Road Transport Investment Model for Developing Countries. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Lab. Rept. 674, 1975.
2. C.G. Harral, P. Fossberg, and T. Watanatada; World Bank. Evaluating the Economic Priority of Highway Maintenance. Pan African Conference on Highway Maintenance and Rehabilitation, Ghana, 1977.
3. J. de Weille. Quantification of Road User Savings. World Bank, Washington, DC, Occasional Paper 2, 1966.
4. H. Hide, S.W. Abaynayaka, I. Sayer, and R.J. Wyall. The Kenya Road Transport Study; Research on Vehicle Operating Costs. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Lab. Rept. 672, 1975.
5. A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-tropical Countries. 3rd ed. Transport and Road Research Laboratory; Her Majesty's Stationery Office, London, 1977.
6. F.A. Hansen. Bituminous Surface Treatments on Rural Highways. Proc., New Zealand Society of Civil Engineers, Vol. 21, 1934-1935.
7. A Basic Asphalt Emulsion Manual. Asphalt Institute, College Park, MD, Manual Series No. 1, MS 19, March 1979.
8. A Guide to Surface Dressing in Tropical and Sub-tropical Countries. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Overseas Road Note 3, (in press).
9. G.P. Jackson. Surface Dressing. Shell International Petroleum Company Ltd., London, 1963.
10. Principles and Practice of Bituminous Surfacing: Vol. 1., Sprayed Work. National Association of Australian State Road Authorities, Sydney, 1965.
11. C. Richardson. The Modern Asphalt Pavement. 2nd ed. Wiley, London and New York, 1908.
12. C.P. Marais. Gapgraded Asphalt Surfacings--The South African Scene. Second Conference on Asphalt Pavements for Southern Africa, Durban, South Africa, 1974.
13. Rolled Asphalt (Hot Process) for Roads and Other Paved Areas. British Standards Institution, British Standard 594, 1973.

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Improved Pavement Management System for Low-Volume Roads

B. FRANK McCULLOUGH, DAVID R. LUHR, AND ADRIAN PELZNER

The economic consequences of pavement design and maintenance for low-volume roads need to be evaluated. Because many low-volume road networks have large mileages compared with high-volume road networks, the capital investment in a low-volume network can be high. Low-volume roads are more economically sensitive, on a percentage basis, to pavement design changes; therefore, the use of a pavement management system in the planning, design, construction, maintenance, and management of low-volume road pavement structures is important. An improved version of the computer program, Pavement Design and Management System (PDMS), was developed that greatly increases the capabilities of the system and also improves the rational basis for predicting pavement performance. A structural analysis of American Association of State Highway Officials road test pavement sections was conducted by using a nonlinear elastic layer procedure to characterize the pavement materials for four seasonal periods. A regression analysis was performed to develop a performance-prediction equation. The dependent variable used was the change in present serviceability index (PSI) divided by the change in vehicle applications for each seasonal period. The performance variable is used to predict the PSI-traffic curve for the pavement structure, thereby allowing the evaluation of the performance area under this curve. Because each vehicle type is considered separately, there is no need to consider axle equivalency factors. This is an important advantage because American Association of State Highway and Transportation Officials equivalency factors are found to have serious limitations.

The term pavement management system was first used in the late 1960s and early 1970s to describe a systems engineering approach to the problem of economical design, construction, and maintenance of roads. Since that time many developments have been made in the area of pavement management systems for both rigid and flexible pavements (1).

Interest in pavement management has increased substantially in recent years as transportation agencies look for new methods and tools to use in the efficient management of transportation networks. Pavement management can be a tremendous asset to the engineer faced with budget restrictions, material shortages, and important energy considerations, and at the same time it can meet increasing demands from heavier axle loads and traffic volumes.

NEED FOR LOW-VOLUME-ROAD PAVEMENT MANAGEMENT

The implementation of pavement management is associated with high-cost, high-volume roadways. The use of a pavement management system for low-cost, low-volume roads may have the appearance of applying too much sophistication to a low-cost situation. However, the term low cost may sometimes be a misnomer when applied to low-volume roads. This is because, in many transportation networks, low-volume roads constitute the majority of the network mileage.

In Texas, the Interstate highway system is composed of approximately 4800 km (3000 miles) of roadway. In comparison, the low-volume farm-to-market system contains approximately 66 000 km (41 000 miles) of roadway. Even though the cost per mile of constructing an Interstate highway is 10 times the cost per mile for farm-to-market construction, the difference in mileage results in the Interstate system having only 75 percent of the capital investment of the farm-to-market system.

The roadway system of the U.S. Forest Service has very small traffic volumes and much lower construction costs per mile than the Texas farm-to-market system. However, the size of this system [420 000 km (260 000 miles)] makes the total capital investment very large. Even though 95 percent of these road miles have no bituminous surfacing, approximately $475 million will be spent in 1982 on maintenance, rehabilitation, and new construction for this system.

Also note that small differences in pavement structure design will have a larger effect on the economics of low-volume than high-volume roads. For example, an increase in the surfacing thickness of 1 in for a low-volume road may result in a 30 percent increase in the construction cost per mile. However, the same increase in the surfacing thickness of an Interstate highway may increase the construction cost by only 5 percent because the cost per mile for the Interstate is much higher. When the increased cost per mile for the low-volume road is multiplied by the large mileage in a low-volume network, the increased cost of the small difference in pavement structure is very large.

It is, therefore, obvious that decisions made regarding pavement design and maintenance for low-volume roads are economically very important. Considering the high cost involved with the number of miles in a typical low-volume road network, the use of a pavement management system as a tool in the planning, design, construction, maintenance, and management of the roadway structure should not be overlooked.

Previous Development

The road system of the U.S. Forest Service is made up entirely of low-volume roads that are often in very rugged terrain, are subjected to severe environmental conditions, and vary regionally from the low swamplands of Florida to the mountains of Alaska. Because of the complexities involved in the efficient design, maintenance, and management of pavement structures in such an extensive roadway system, the U.S. Forest Service initiated, with the University of Texas, a cooperative study in 1972 to develop a pavement management system that would be applicable to Forest Service roads. The result of this cooperative effort was the computer program Pavement Design and Management System (PDMS), which was implemented on a trial basis in the Forest Service in 1977 and on a permanent basis in 1979 (2-4).

This pavement management system calculates and optimizes pavement design and rehabilitation strategies on a project level after being given information about available construction materials, material characteristics, expected traffic volumes and loads, various costs, and required pavement performance. Pavement management is accomplished by effectively considering all costs associated with the pavement structure over the entire length of the analysis period. This includes initial construction and subsequent costs related to road maintenance, rehabilitation, and vehicle operation. In this way, the designer is able to make the most efficient use of available resources, in addition to having a tool for future planning purposes.

As a result of the implementation of this pavement management system, it was determined that the system could be revised and improved significantly through a new research and development effort. An initial investigation found that a mechanistic approach to pavement design and prediction of pavement performance would be the best way to revise the system (5). As a result of this initial investigation, plans and objectives were developed that would result in a new and improved version of PDMS.

Summary of Objectives

The objective of the research and development effort to improve PDMS was to enhance the capabilities of the pavement management system and also to improve the fundamental approach of certain performance models within the system. Specifically, the primary objectives were the following:

1. Develop a design algorithm based on a rational characterization of the pavement structure and pavement response,
2. Develop the capability to consider the seasonal characteristics of pavement materials as well as seasonal changes in traffic volume,
3. Develop the capability to consider the stress-sensitive characteristics of unbound materials in the pavement structure,
4. Develop the capability to rationally evaluate the effects of heavy loads or new axle configurations, and
5. Improve the capability to consider the effect of pavement performance on vehicle operating and user costs.

STRUCTURAL ANALYSIS

In designing a pavement structure for a low-volume road, remember that its structural behavior is no different from that of a pavement structure for a high-volume road. That is, the laws of physics that govern the structural characteristics of pavement materials do not change. Subtle differences may exist, but within the realm of a pavement management system all flexible pavements can be structurally analyzed in the same way. For this reason it was decided to structurally analyze the pavement sections at the American Association of State Highway Officials (AASHO) road test, even though the level of traffic at the road test was very high (6).

An earlier analysis of AASHO road test pavement sections indicated that a simple linear-elastic evaluation of the pavement sections by using layer theory worked fairly well in correlating pavement performance with compressive subgrade strain (5). Considering this, and also that the AASHO road test data provide some of the best information available on pavement performance, it was decided to conduct a more detailed structural analysis of the AASHO road test sections by using a non-linear-elastic methodology.

In the non-linear-elastic analysis of the pavement structures, the stress-sensitive nature of the resilient moduli for the unbound pavement materials (granular base and subbase, fine-grained subgrade) were characterized. This stress-sensitive characterization, as well as modulus values for the asphalt layer, was developed by Finn and others (7) for four different seasonal conditions at the AASHO road test. In this way, it was possible to study the seasonal differences in pavement behavior. The structural analysis of the pavement sections was carried out by using a modified version of the elastic-layer program bitumen structures analysis in roads (BISAR); the original BISAR program was developed by Shell Research (8). The development of the modified BISAR program, as well as the details of the structural analysis methodology, is discussed elsewhere (9).

PERFORMANCE MODEL DEVELOPMENT

The structural analysis of AASHO road test pavement sections was completed by analyzing all the combinations of pavement structures and axle loads that exist at the road test by using the modified BISAR program. Several stress-strain parameters were calculated for each pavement section and are discussed in the following sections on performance variable, independent variables, and regression analysis.

Performance Variable

Once the pavement behavior of the AASHO road test sections had been mechanistically characterized by the structural analysis, the development of a performance model required the correlation of the mechanistic pavement parameters with a measure of pavement performance. In selecting the pavement performance parameter to be used, it was decided that the use of a macroperformance variable, such as present serviceability index (PSI), would be better than the use of microperformance variables (e.g., rut depth, cracking, and raveling). There were two major reasons for this decision: (a) the microperformance variables are not mutually exclusive but are highly interrelated and difficult to predict and (b) the PSI, which is primarily a roughness index, represents the performance parameter that most affects the road user.

In order to achieve the objectives mentioned at the beginning of the paper, a performance variable needed to be selected that would change with seasonal conditions. The performance variable selected was, therefore, the rate of PSI deterioration with traffic, or

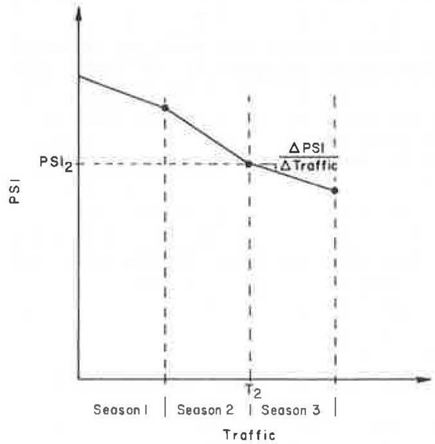
$$\text{Performance variable} = \Delta\text{PSI}/\Delta\text{TRAFFIC} \quad (1)$$

where ΔPSI is the change in PSI during a given season and ΔTRAFFIC is the change in vehicle applications during the same season. Note that, in this analysis, no seasonal weighting function was used to transform the traffic data. However, seasonal weighting of traffic data was used in the development of the American Association of State Highway and Transportation Officials (AASHTO) performance equation (6).

This performance variable concept is illustrated in Figure 1, which shows the PSI versus traffic curve divided into straight-line segments for each season. The performance variable is predicted for each season, thereby proceeding along the performance curve in a step-by-step manner until the PSI reaches some terminal, or failure, level. This type of performance prediction variable has important advantages:

1. Pavement performance is predicted for different seasons, which allows the consideration of seasonal pavement behavior or seasonal traffic conditions;
2. The performance variable is affected by the pavement performance history; for example, in Figure 1 the PSI at the end of season 2 (PSI_2) and the cumulative traffic at the end of season 2 (T_2) are used as independent variables to predict the performance variable for season 3; and
3. This procedure predicts the PSI-traffic performance curve, and the area under this curve can be used as an indicator of pavement performance that can be compared with performance areas of other pavement structures or used in analyzing other important pavement factors, such as vehicle user costs.

Figure 1. Illustration of performance variable.



Independent Variables

The independent variables used in the performance prediction model fall into three categories: vehicle type, pavement history, and pavement mechanics.

Vehicle Types

Vehicle type variables describe the vehicle being considered and include number of tires on the load axle, tire pressure, load per tire (steering and load axles), and number of load axles per vehicle. In this way, the entire vehicle is considered as one unit instead of being divided into axle applications.

Pavement History

As mentioned during the discussion of Figure 1, the values of PSI and cumulative traffic at any given time are used in the model to predict the performance variable for the next seasonal period. This causes a different rate of deterioration for a new pavement than for a pavement that has been in service for a period of time.

Pavement Mechanics

Pavement mechanics variables reflect the change in pavement performance due to seasonal conditions, pavement structures, and axle loads. The variables include tensile strain at the bottom of the asphalt layer, subgrade compressive strain, subgrade shear strain, and subgrade strain energy. Pavement deflection was specifically not included because of difficulties in predicting pavement deflection when the depth to rigid foundation (depth of roadbed) is not known (10).

Regression Analysis

During the regression analysis it was found that the best prediction equation occurred when the dependent variable was transformed by using a square-root function. A statistical summary of the prediction equations developed is given in Table 1. Several equations were developed that can be used separately for different pavement conditions and axle types. The general equations given below can be used in all

Table 1. Statistical summary of performance equations.

Equation	R^2	Standard Error	Coefficient of Variation	No. of Terms in Equation	No. of Data Observations
Full-depth asphalt[a]	0.931	0.0028	0.30	6	33
All other asphalt pavements[a]	0.565	0.0031	0.69	10	1469
Two and three layer					
Single axle, single tire	0.712	0.0017	0.78	4	165
Single axle, dual tire	0.510	0.0033	0.73	6	677
Tandem axle, dual tire	0.637	0.0029	0.58	10	627

[a]General equation.

cases and are primarily for considering conditions excluded in the other equations (i.e., different axle configurations).

For full-depth asphalt,

$$\begin{aligned}\Delta PSI/\Delta TRAFFIC = {} & 1.751(C) + 5.772 \times 10^{-4}(I) + 1.264 \times 10^{1}(D \cdot G) \\ & - 8.946 \times 10^{1}(J \cdot H)^2 + 1.715 \times 10^{4}(H)^3 \\ & + 1.677 \times 10^{-3}\{\log_{10}[(5.0 - L)/(M + 1)]\} \\ & + 8.449 \times 10^{-3}\end{aligned} \tag{2}$$

For all other asphalt pavements,

$$\begin{aligned}\Delta PSI/\Delta TRAFFIC = {} & 1.054 \times 10^{-4} + 0.9763\{-4.168 \times 10^{-4}(K) - 9.951(B) \\ & + 2.941 \times 10^{-1}(A \cdot C) - 6.919 \times 10^{-1}(D \cdot E) \\ & + 2.922(J \cdot F) - 3.630 \times 10^{-6}(\sqrt{M}) \\ & - 1.985 \times 10^{1}(J \cdot H)^2 + 6.037 \times 10^{-3}[(5.0 \\ & - L)/(M + 1)] + 3.785 \times 10^{-4}(J)^2 + 9.849 \\ & \times 10^{1}(I \cdot G)^2 + 4.050 \times 10^{-3}\}\end{aligned} \tag{3}$$

where

A = steering axle load per tire (kips);
B = steering axle asphalt strain;
C = steering axle subgrade strain energy;
D = load axle load per tire (kips);
E = load axle asphalt strain;
F = load axle subgrade strain;
G = load axle subgrade shear strain;
H = load axle subgrade strain energy;
I = 1 for single axle, single tire, 2 for single axle, dual tires, and 4 for tandem axle, dual tires;
J = number of load axles per vehicle;
K = asphalt layer thickness;
L = existing PSI; and
M = existing cumulative traffic.

A separate general equation was developed that does not contain any asphalt strain terms. This general equation is used to predict the performance variable for aggregate-surfaced roads and will be calibrated or replaced in the future when adequate performance data are obtained for aggregate-surfaced roads.

In the meantime, two other performance models are used when designing aggregate-surfaced roads. One model involves the accumulation of rutting caused by cumulative traffic and was developed by the Waterways Experiment Station (11). The second model concerns the reduction of the pavement structure through surface aggregate loss. This term is either input directly by the user as a function of traffic or is calculated by a regression equation (4). The PDMS program considers the aggregate-surfaced road to have failed when any of these three models fails.

In almost all equations for bituminous pavements, the subgrade stress-strain variables contributed more to the accuracy of the equation than did the asphalt tensile strain. This trend was also found in the preliminary performance equation developed during this project (5). The reason for this trend is that the subgrade parameters give a better indication of the response of the entire pavement structure, whereas the asphalt parameter tends to be dominated by conditions in the asphalt layer. In a small analysis of simple correlation coefficients, it was even found that subgrade parameters correlated better with the percentage of cracking in the asphalt than did asphalt tensile strain (12).

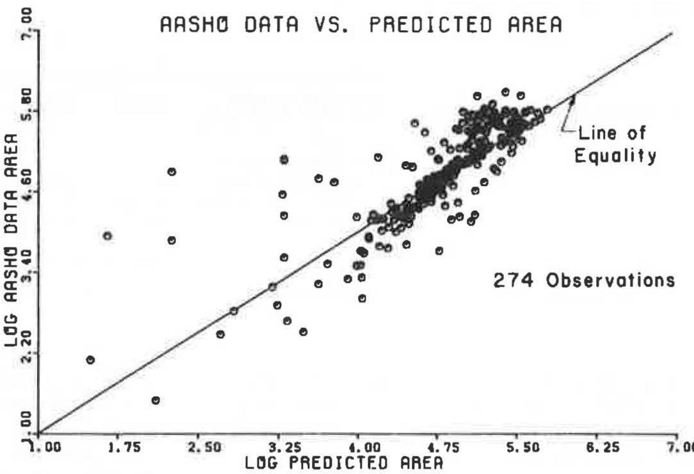
Perhaps the best way to examine the accuracy of the prediction equations is to compute the performance area under the predicted PSI-traffic curves and compare it with the performance area under the actual PSI-traffic curves from AASHO road test data. This was done for those sections that reached a PSI of 3.0 or below and the results are plotted in Figure 2. The coefficient of determination (R^2) for this set of 274 observations was 0.65, with a root-mean-square error of 0.43 ($10^{0.43} = 2.69$). An important statistic to compare with the root-mean-square error of the predicted area under the performance curve is the root-mean-square error for the replicate data sections. These replicate data give an indication of the repeatability of the measurements made at the road test. In the case of the areas under the performance curves, 24 replicate sections had a root-mean-square error of 0.22 ($10^{0.22} = 1.66$). The root-mean-square error of 0.43 for the predicted areas appears very reasonable when compared with the replicate value.

PAVEMENT MANAGEMENT SYSTEM FRAMEWORK

The PDMS program evaluates and optimizes pavement structure design and rehabilitation strategies on the basis of total overall cost for the life of the desired analysis period. This total overall cost includes the initial construction cost, cost of routine maintenance, rehabilitation costs, user vehicle and delay costs, and salvage value, all considered on a net-present-value basis.

The program inputs indicate what pavement materials are available and at what cost, the traffic loads that the pavement structure will be required to carry, constraints with respect to pavement thickness and initial construction funds, performance requirements, and user costs. PDMS evaluates candidate pavement structures to determine if they satisfy the designer's constraints. The candidate structures are first analyzed structurally to calculate the stress-strain parameters used in the prediction equations. This is accomplished by using regression equations that model the modified BISAR program, rather than the entire modified BISAR program, so that computation time is not excessive. The performance prediction equations are then used to predict the change in PSI due to each vehicle type for each season. Since each vehicle type is considered separately, there is no need to use axle equivalency factors. The total change in PSI due to all vehicle types in the traffic stream is accumu-

Figure 2. Measured versus predicted performance areas from AASHO road test sections.



lated for each year, proceeding until the terminal, or failure, level of PSI is reached. This process of repeatedly adding the change in PSI caused by each vehicle type eliminates the need to use Miner's rule in determining the cumulative pavement damage.

When the terminal serviceability level is reached the program will consider a range of overlay thicknesses, by using the procedure above, until the pavement again deteriorates to the terminal PSI level or the end of the analysis period is reached (Figure 3). The program, therefore, has the capability of considering hundreds of different pavement design and rehabilitation strategies, including the use of different pavement materials and variations in the layer thicknesses and number of layers in the structure. This procedure allows the comparison of a thick (and expensive) initial construction that requires little maintenance with a thin initial construction that requires frequent rehabilitation (stage construction).

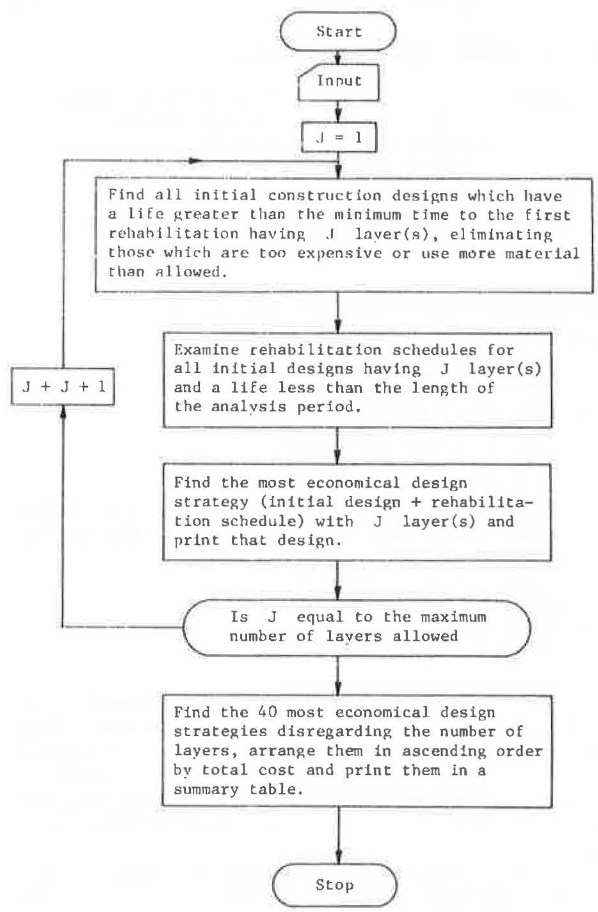
All pavement designs and rehabilitation strategies that satisfy the designer's constraints are sorted according to total overall cost, and the 40 designs that have the lowest cost are printed. In addition, the cost/area ratio (calculated by dividing total cost by the performance area under the PSI-traffic curve) is calculated for each design. This parameter assists the designer in evaluating the 40 lowest-cost designs, because the expenditure of an additional amount of money may decrease the cost/area ratio significantly. The designer may then decide that the optimum pavement design and rehabilitation strategy is not the one that has the absolute minimum cost but rather the one that has the minimum cost/area ratio.

SIGNIFICANCE OF PDMS IMPROVEMENTS

Stress Sensitivity

One reason that PDMS considers the stress sensitivity of unbound pavement materials is for the rational economic comparison of materials. The condition may occur where two materials, for example an expensive high-quality granular material (with high stiffness) and an inexpensive low-quality granular material (with low stiffness), would be compared as possible construction materials for a pavement structure. By assuming constant stiffness of the materials, the expensive material would always provide a stronger pavement structure, assuming that each material is of equal thickness. However, considering the situation of a very weak subgrade and heavy wheel loads, the high tensile strains in the

Figure 3. Simplified flowchart of PDMS.



granular layer would have the effect of reducing the stiffness of the granular layer. This could reduce the effectiveness of the expensive material until it was only slightly stiffer than the inexpensive material. Therefore, for economic comparisons, it is important that PDMS characterize the stress-sensitive nature of pavement materials. This is especially true when the use of a subbase material to build-up the structure from a weak subgrade is being evaluated. This fact, along with the relatively good correlation obtained between predicted and measured performance areas under the PSI-traffic curve

Figure 4. Measured and AASHTO 80-kN equivalence factors for single axle loads.

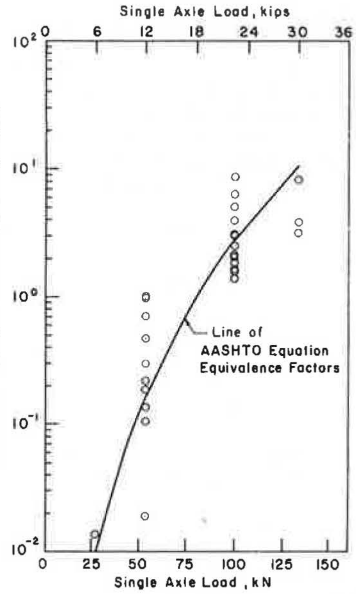
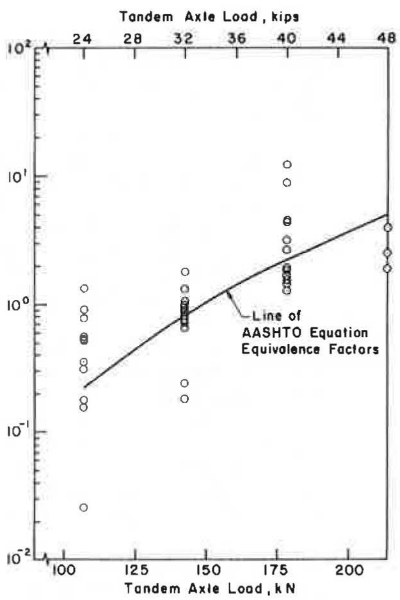


Figure 5. Measured and AASHTO 80-kN equivalence factors for tandem axle loads.



(Figure 2), helps to show the benefit of characterizing the stress-sensitive nature of unbound pavement materials.

Axle Equivalence Factors

An earlier development of an AASHO road test algorithm indicated that axle equivalence factors may also be affected by stress sensitivity in the pavement structure, because different equivalence factors were computed for different pavement structures (5). Axle equivalence factors developed by the AASHTO performance equation and tabulated in the AASHTO Interim Guide indicate that equivalence factor values seldom change by more than 10 percent from a thin pavement structure to a thick structure. Stated more simply, the AASHTO equivalence factor values are nearly constant for a given axle load (13).

The accuracy of axle equivalence factors may be evaluated by analyzing the identical pavement structures that carried different loads at the AASHO road test. For example, eight different test sections were built with identical 7.5-cm (3-in) asphalt, 7.5-cm base, and 10-cm (4-in) subbase pavement structures. Each of these test sections carried one type of axle load; they included loads of 9-kN (2-kips), 53-kN (12-kips), 80-kN (18-kips), 100-kN (22.4-kips) single axle and 27-kN (6-kips), 107-kN (24-kips), 142-kN (32-kips), 178-kN (40-kips) tandem axle.

Since an 80-kN (18-kip) equivalence factor is computed by

$$e_x = W_{t80}/W_{tx} \qquad (4)$$

where

e_x = 80-kN equivalence factor for axle load x,
W_{t80} = applications of 80 kN necessary to reach a given PSI level, and
W_{tx} = applications of axle load x necessary to reach a given PSI level,

actual equivalence factors from the AASHO road test data can be computed for those identical pavement sections with different axle loads. To examine specifically the 80-kN equivalence factor, those identical sections at the road test that had 80-kN loads in addition to at least one other load were found. With these data, it was possible to calculate 80-kN equivalence factors for 28 single-axle cases and 49 tandem-axle cases. The results are shown for single axles in Figure 4 and for tandem axles in Figure 5. On both figures, because a logarithmic scale is used, a single solid line represents the AASHTO equivalence factors. From these figures it can be seen that the equivalence factors from the AASHTO performance equation do not satisfactorily explain the variation in equivalence factors from the road test data, especially when it is considered that the equivalence factors are plotted on a logarithmic scale. Because of the error involved in using equivalence factors, that PDMS considers vehicle types separately and does not use equivalence factors is important.

Seasonal Variation

Pavement performance is affected by seasonal changes that occur in the pavement structure. Seasonal conditions change from one region to another, therefore, it is necessary to characterize the seasonal influence on performance. PDMS does not consider non-load-associated environmental distress (i.e., temperature cracking) in a direct manner; however, the environmental effects on load-associated distress are considered in the way pavement performance is modeled separately for each season.

The U.S. Forest Service and some other transportation agencies in northern climates that manage low-volume roads have the authority to impose roadway axle load limits during certain times of the year when the pavement structure is in a very weak condition. This situation usually occurs in the springtime, when melting in the upper layers of the pavement structure causes water to accumulate above the still frozen lower layers. The PDMS program has the capability to evaluate the economic trade-off of increasing road restrictions or paying higher pavement costs.

User Cost

The improved prediction of the PSI-traffic curve enables PDMS to increase considerably the analysis of vehicle user cost. Little is known about the relation between user cost and pavement performance. However, substantial research and development work is currently being done in this area (14,15). The improved framework of PDMS allows the designer to define user cost as a function of PSI for each vehicle. Since the PSI level is predicted for every season, the user cost can be computed and included in the total overall cost. This procedure is an optional feature of PDMS because the designer may not have accurate user data or simply may not wish to consider user cost. If user cost data are not available and the designer would still like an indication of the level of pavement performance, the performance area under the PSI-traffic curve can be used as an indicator of which pavement design and rehabilitation strategy provides the highest level of service (and, therefore, the lowest overall user costs).

Extrapolation of Performance Models

Because of constantly changing conditions, pavement materials, axle loads, and axle configurations, any model of pavement performance developed will almost certainly be subject to use outside the data range from which it was developed. In addition, moving the performance model from one region or country for use in a different environment will result in the same type of extrapolation. This is a serious problem for strictly empirical performance models such as the AASHTO performance equation, which relies on subjective parameters such as the material strength coefficient, soil support value, and regional factor to extrapolate beyond the data at the AASHO road test. However important these parameters are when used in the AASHTO equation, no quantitative methods are available to determine their values.

Perhaps the most-important improvement of PDMS comes from the mechanistic basis of the new performance model. Parameters similar to the material strength coefficient or soil-support value are not necessary because the pavement and roadbed materials are characterized by using the resilient modulus. No regional factor terms are necessary because the pavement performance is directly a function of the seasonal characteristics of the material. This greatly improves the ability to use this pavement management system in a wide range of circumstances, including aggregate-surfaced roads. However, a comprehensive data base of pavement performance information is necessary to calibrate or revise the performance models in PDMS, particularly for the aggregate-surfaced roads. The U.S. Forest Service appreciates this and, in the near future, plans to develop a data base system for its road network (16).

CONCLUSIONS

This paper presents the results of a research and development effort to improve PDMS. Based on the information presented, the following conclusions are made:

1. A pavement management system is an important tool in evaluation of the significant economic effects of low-volume road design;
2. The mechanistic approach used in developing the performance model provides a good foundation for the inevitable extrapolation outside the range of original data; however, it is no substitute for a comprehensive data base;
3. The performance variable is used to predict the PSI-traffic curve, thereby allowing the evaluation of the performance area under this curve;
4. Pavement deterioration is calculated by adding the change in PSI due to each vehicle type instead of using Miner's rule to estimate cumulative damage;
5. The consideration of the stress sensitivity of unbound pavement materials is important in a pavement-management system;
6. The AASHTO equation equivalence factors have definite limitations, and that equivalence factors are not used is an advantage of PDMS;
7. Seasonal variation of pavement and traffic conditions is important and is characterized in PDMS; and
8. PDMS has the capability to consider user cost, which will be important in the future when more data are available, and is capable of considering user preferences through the cost/area ratio.

ACKNOWLEDGMENT

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REFERENCES

1. R. Haas and W.R. Hudson. Pavement Management Systems. McGraw-Hill, New York, 1978.
2. T.G. McGarrah and W.R. Hudson. A Pavement Design and Management System for Forest Service Roads--Conceptual Study. Council for Advanced Transportation Studies, Univ. of Texas at Austin, Austin, Res. Rept. 12, July 1974.
3. F.L. Roberts, B.F. McCullough, H.J. Williamson, and W.R. Wallin. A Pavement Design and Management System for Forest Service Roads--Working Model. Council for Advanced Transportation Studies, Univ. of Texas at Austin, Austin, Res. Rept. 43, Feb. 1977.
4. B.F. McCullough and D.R. Luhr. A Pavement Design and Management System for Forest Service Roads--Implementation. Council for Advanced Transportation Studies, Univ. of Texas at Austin, Austin, Res. Rept. 60, Jan. 1979.
5. D.R. Luhr and B.F. McCullough. Development of a Rationally Based AASHO Road Test Algorithm. TRB, Transportation Research Record 766, 1980, pp. 10-17.
6. The AASHO Road Test: Report 5--Pavement Research. HRB, Special Rept. 61E, 1962.
7. F. Finn, C. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abdullah. The Use of Distress Prediction Subsystems for the Design of Pavement Structures. Proc., 4th International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Aug. 1977.
8. D.L. DeJonk, M.G.F. Pentz, and A.R. Korswagen. Computer Program BISAR, Layered Systems Under Normal and Tangential Surface Loads. Shell Research, Amsterdam, External Rept., July 1972.
9. D.R. Luhr and B.F. McCullough. Structural Analysis of AASHO Road Test Flexible Pavements for Performance Evaluation. TRB, Transportation Research Record, 1982 (in preparation).
10. B.F. McCullough and K.J. Boedecker. Use of Linear-Elastic Layered Theory for the Design of CRCP Overlays. HRB, Highway Research Record 291, 1969, pp. 1-13.
11. V.C. Barber, E.C. Odom, and R.W. Patrick. The Deterioration and Reliability of Pavements. U.S. Army Waterways Experiment Station, Vicksburg, MS, Tech. Rept. S-78-8, July 1978.
12. D.R. Luhr. Comparison of Subgrade Strain Equation with Shell Pavement Design Procedure.

Center for Transportation Research, Univ. of Texas at Austin, Austin, Forest Service Project Tech. Memorandum 44, Jan. 1981.
13. AASHTO Interim Guide for Design of Pavement Structures--1972. AASHTO, Washington, DC, 1974.
14. R.J. Wyatt, R. Harrison, B.K. Moser, and L.A.P. de Quadros. The Effect of Road Design and Maintenance on Vehicle Operating Costs--Field Research in Brazil. TRB, Transportation Research Record 702, 1979, pp. 313-320.
15. J.P. Zaniewski and others. Vehicle Operating Costs, Fuel Consumption, and Pavement Type and Condition Factors. Texas Research and Development Foundation, Austin, Final Rept. FHWA 11 9878, March 1982.
16. J. Hernandez, B.F. McCullough, and W.R. Hudson. A Data Base for the U.S. Forest Service Pavement Management System. Center for Transportation Research, Univ. of Texas at Austin, Austin, Res. Rept. 66, May 1981.

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Performance of Various Thicknesses of PCC Pavement

JOHN I. HELMERS AND VERNON J. MARKS

If adequately designed and high-quality material and good construction practices are used, portland cement concrete is very durable. This is demonstrated by the oldest pavement in Iowa (second oldest in the United States), which was paved in 1904. It performed well for 70 years without resurfacing. The design thickness is an important factor in both the performance and cost of pavement. The objective of this paper is to provide a 30-year performance evaluation of a pavement constructed to determine the required design thickness for low-volume secondary roadways. In 1951 Greene County and the Iowa Highway Research Board of the Iowa Department of Transportation initiated a 4-mile (6.4-km) demonstration project to evaluate thicknesses that ranged from 4.5 to 6 in (11.4-15.2 cm). The project, which consisted of 10 research sections, was formed pavement placed on a gravel roadbed with very little preparation except for redistribution of the loose aggregate. Eight sections were nonreinforced except for centerline tie bars, and no contraction joints were used. Mesh reinforcing and contraction joints spaced at 29 ft 7 in (9.02 m) intervals were used in two 4.5-in (11.4-cm) thick sections. The only air-entrained section was nonreinforced. The pavement performed well over its 30-year life of carrying a light volume of traffic and did not require major maintenance. Cracking was substantial; average slab length varied directly with thickness. The 4.5-in-thick nonair entrained, mesh-reinforced pavement with contraction joints has performed the best.

Iowa's portland cement concrete (pcc) paving began in 1904 with one-half block in the town of LeMars (second oldest pavement in the United States). This two-lift pavement was 6.5 in (16.5 cm) thick, and the top 1.5 in (3.8 cm) had a greater cement factor than the bottom 5 in (12.7 cm). The joints were formed at 6-ft (1.83-m) intervals skewed 45° from each side, which created a diamond pattern. The texture was obtained by scoring the surface in 4-in (10.2-cm) squares. This pavement performed well for 70 years without resurfacing and demonstrates the potential of pcc pavement. Many miles of pcc pavement were constructed in the late 1920s and early 1930s during a campaign to get Iowa out of the mud.

Iowa is a state of 56 290 miles^2 (145 791 km^2), and only eight urban areas have a population greater than 50 000. It has 112 257 miles of roadway (180 660 km), the surface types of which are given in Table 1. In 1951, excluding municipal roads, there were 8248 miles (13 274 km) of hard surfacing, 58 598 miles (94 304 km) of gravel, and 35 523 miles (57 169 km) that had no surfacing. The 94 121 miles (151 473 km) of roadway without hard surface and the belief that adequate design, high-quality material, and good construction are essential for durable concrete were the impetus for Iowa Highway research project HR-9.

Substantial research has been conducted into structural requirements. Studies of flexural fatigue as a function of design thickness were completed in the 1920s. This research was used in the development of the 1933 Portland Cement Association (PCA) design curve for pavement. The Iowa Department of Transportation is currently using the 1966 PCA design procedure.

Table 1. Miles of Iowa highway by surface type, 1981.

	Iowa Highway by Surface Type (miles)			
Surface Type	Primary	Secondary	Municipal	Total
Portland cement paved	4557	4 536	3305	12 463
Asphalt concrete paved	5465	8 936	5879	20 379
Bituminous treated	88	1 514	1058	2 694
Gravel	15	68 942	1701	70 767
Not surfaced, dirt	0	5 825	127	5 954

The objective of the research project was to determine to what thickness pcc pavement could be reduced, with corresponding cost reduction, and still provide a high-quality surface of long life for low-volume secondary roadways. The objective of this report is to provide a 30-year performance evaluation of the experimental pcc roadway.

PROJECT IDENTIFICATION

Greene County is located in central Iowa approximately 50 miles northwest of Des Moines. The project is 4 miles (6.4 km) long on County Road E-33 from IA-4 to Farlin.

Signs that show the thickness and reinforcing of the pavement were installed along the north right-of-way line of the project. They were placed at the ends of the sections, and arrows on the signs pointed to the section to which the information applied. These signs are still present on the project as an aid to observers in locating the various sections and evaluating the present condition.

A nonconformity of the signs' text and the terms used in this report is that the nonreinforced sections are listed as dowel reinforced. The dowel term noted on the signs refers to the centerline tie bars.

Preconstruction Testing

In the spring of 1951 soil borings were taken and load-bearing tests were performed by the plate-bearing method to determine the suitability of the existing roadbed as a base for the pavement. Bearing values under a 12-in diameter plate at yield point ranged from 58.5 to 9 lb•f/in² (403-62 kPa); 30 lb•f/in² (207 kPa) is considered adequate. Based on this criteria, load-bearing tests showed 4100 ft (1250 m) of unstable base, and the soil borings indicated some areas in which there was a high water table and a subgrade that consisted mainly of clay loam (U.S. Bureau of Public Roads subgrade group A-6).

Base Drainage

In the areas identified as being unstable, vertical sand drains were constructed to provide for moisture movement. These drains were 6 ft (1.8 m) deep, 7-in (17.8-cm) diameter holes filled with clean sand and a solution of calcium chloride and water compacted with a mechanical vibrator. They were located on 5-ft (1.5-m) centers in five parallel lines in a checkerboard pattern. There were 4064 drains constructed in the following locations (note that the project is stationed east to west):

From station 3+50 to 11+25,
From station 26+00 to 31+00,
From station 47+00 to 53+00,
From station 62+00 to 65+50,
From station 87+00 to 99+50, and
From station 99+50 to 105+60.

Soon after the project was paved, the county engineer questioned the effectiveness of these drains because no horizontal interconnecting blanket or outlets through the earth shoulders were provided. The first winter after construction produced severe frost action and resulted in minor heaving of two areas where there had been vertical sand drain treatment.

Table 2. Design and construction summary.

Section No.	Location[a] From Station	To Station	Thickness (in)	Reinforcement	Contraction Joint Spacing (ft)
1	0+10	18+00	5	None	-[c]
2	18+00	27+00	4.5	Mesh	29.58
3	27+00	35+00	4.5	None	-[c]
4	35+00	53+00	5.5	None	-[c]
5	53+00	71+00	5	None	-[c]
6	71+00	80+00	4.5	Mesh	29.58
7	80+00	89+00	4.5	None	-[c]
8	89+00	106+00	5.5	None	-[c]
9	106+00	159+00	6	None	-[c]
10	159+00	211+15	6[b]	None	-[c]

[a]Project is stationed east to west. [b]Air entrained. [c]No contraction joints.

Grade Preparation

The original intent was to use the existing gravel-surfaced roadway with very little preparation except for uniform distribution of the loose aggregate on the surface. A profile grade tolerance of 0.15 ft (4.6 cm) was established (the allowable variation between the finish grade and the existing grade). This tolerance presented a challenge to the contractor because the roadbed had been constructed 12 years earlier.

DESIGN AND CONSTRUCTION

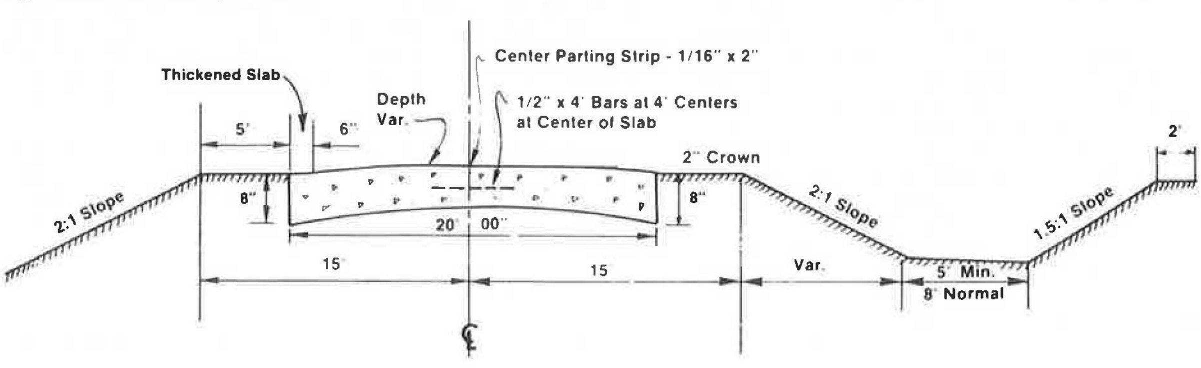
The 4-mile (6.4-km) project was divided into 10 sections of various lengths, given in Table 2. The pavement thicknesses were arbitrarily selected to range from 4.5 to 6 in (11.4-15.2 cm) and were not based on the plate-bearing results or the PCA design formula. Substantial engineering judgment is used with the modulus of subgrade reaction for Iowa pavement thickness design yet today. The concrete proportions were specified as Iowa State Highway Commission Mix 4A. (Note: Aggregate absolute volumes and batch quantities were adjusted for the air-entrained concrete.)

Item	Absolute Volume	Batch Quantities (lb)
Cement, min	0.096 419	510
Water, approx.	0.161 201	272
Aggregates		
Fine, approx. (Sp.Gr. = 2.66)	0.371 190	1664
Coarse approx. (Sp.Gr. = 2.69)	0.371 190	1682

The cement was type 1 from Penn Dixie in Des Moines, Iowa, and the sand and gravel aggregates were produced by Ferguson Diehl Company of Jefferson, Iowa. The air-entraining agent used in section 10 was a commercially available liquid product (Darex) added at the mixer. Air entrainment was not a common practice in 1951.

Paving operations began in September 1951. The 20-ft (6.1-m) wide pavement was built by using the conventional equipment of that time. The concrete was dry-batched at a plant located in Farlin. The dry-batched concrete was mixed on site and deposited on subgrade paper between the fixed forms. These forms were 8 in (20.3 cm) high, and since the pavement thicknesses specified were less than 8 in, the outer 6 in (15.2 cm) of the base on each side was sloped to the bottom of the form to yield a thickened edge of slab. Figure 1 shows a typical cross section of the pavement.

Figure 1. Cross section of pavement.



All of the nonreinforced pavement designs have 4-ft (1.2-m) long no. 4 (1.27-cm diameter) deformed steel rebars placed on 4-ft centers across the centerline as tie bars. Two of the four 4.5-in (11.4-cm) thick sections were also reinforced with welded wire mesh. The layouts for the nonreinforced and reinforced pavements are given in Figures 2 and 3, respectively.

The joints in the slab were formed by placing premolded bituminous parting strips in the fresh concrete. A longitudinal joint was formed along the center of the slab. Other joints included days-work joints and contraction joints at the ends of the mats of reinforcement [29-ft 7-in (9.02-m) spacing] in the 4.5-in mesh-reinforced sections.

TESTING AND EVALUATION

Both beam [6x6x33 in (15.2x15.2x83.8 cm)] and cylinder [6x12 in (15.2x30.5 cm)] test specimens were made to test concrete strength during construction. Cores were drilled at 260 days and 28 years. A summary of concrete strengths is given in Table 3.

Crack surveys have been conducted 14 times since construction, more frequently in the first two years. The last three were 6, 14, and 28 years after construction. The length of individual sections is divided by the total transverse cracks plus transverse joints to yield an average slab length. A summary is given in Table 4. At one month, the average slab length of pavement without contraction joints ranged from 81 to 192 ft (24.7-58.5 m). The 5-year range is from 17 to 33 ft (5.2-10.1 m) and the 28-year range is from 13 to 22 ft (4.0-6.7 m) (on the jointed slab).

A summary of the longitudinal cracking is given in Table 5. Very little longitudinal cracking occurred during the first year but it increased steadily thereafter. Some variations are somewhat different than expected, both between repeated sections and averages of sections that have different thicknesses. These differences may be due to the variations in stability of the grade.

A test of the longitudinal profile was not made until 1955, when the sections were tested with a Bureau of Public Roads (BPR) roughometer. Testing with the BPR roughometer was conducted in 1955, 1979, and 1981. The results are summarized in Table 6.

Maintenance

Maintenance of this pavement has been minimal; little more than crack sealing has been done for most of its life. A crack-sealing effort in 1980 deposited enough sealant material on the surface to

Figure 2. Layout for nonreinforced pavement.

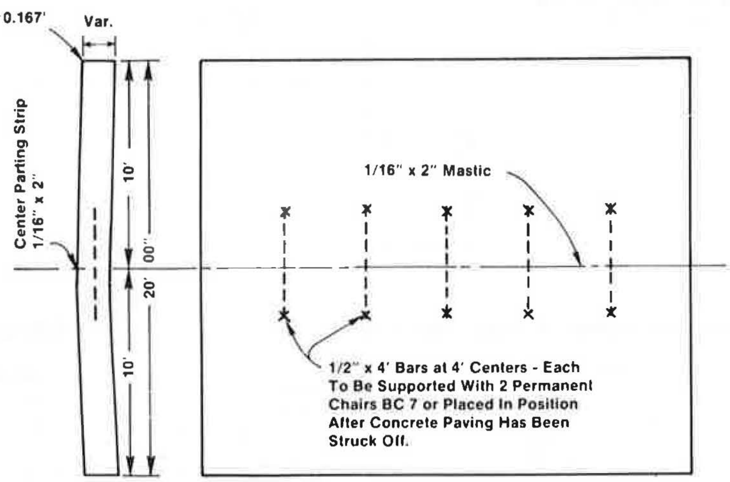


Figure 3. Layout for 4.5-in mesh-reinforced pavement.

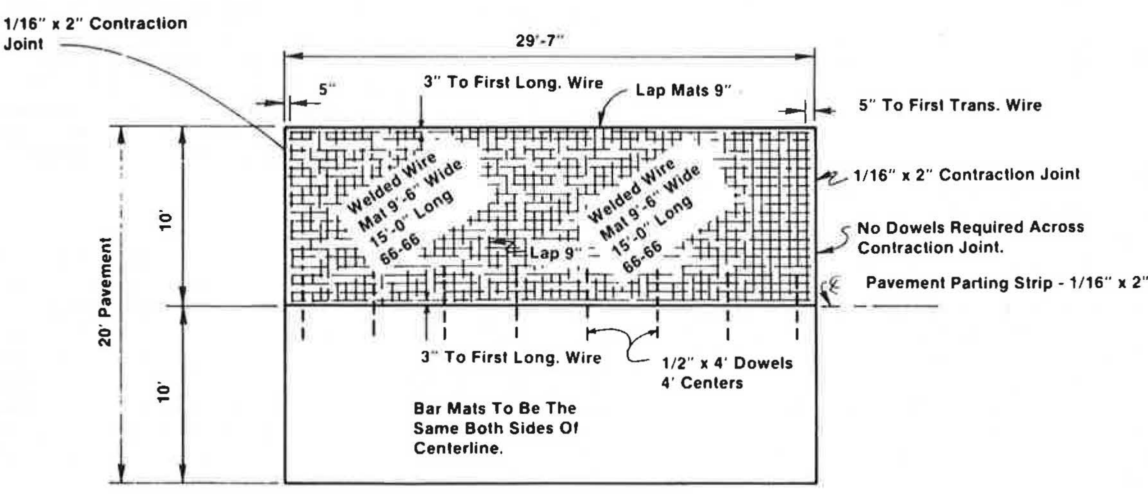


Table 3. Concrete strengths.

Section No.	Thickness (in)	Air Content by High-Pressure Test (%)	Modulus of Rupture, 28-Day Beams (lb·f/in²)	Compressive Cylinders, 28 Days (lb·f/in²)	Compressive Cores, 260 Days (lb·f/in²)	Compressive Cores, 28 Years (lb·f/in²)
1, 5	5	3.5	800	5370	6060	8090
2, 6	4.5	2.6	780	5520	6210	8070
3, 7	4.5	3.2	790	5850	6220	8100
4, 8	5.5	3.4	800	5490	6240	7500
9	6	3.5	810	5870	6580	7820
Avg		3.2	800	5620	6260	7920
10	6[a]	6.6	770	5290	6080	7540

[a]Air entrained.

Table 4. Transverse crack summary.

Section No.	Thickness (in)	Average Slab Length (ft)					
		1 Month	1 Year	2 Years	5 Years	14 Years	28 Years
1, 5	5	92	26	19	17	15	15
2, 6	4.5	30	29	28	26	22	22
3, 7	4.5	81	29	21	18	15	13
4, 8	5.5	192	33	25	22	19	17
9	6	123	56	39	33	26	18
10	6[a]	177	59	36	27	21	18

Note: Slab length = length of section/(no. of transverse cracks + no. of joints).
[a]Air entrained.

Table 5. Longitudinal crack summary.

Section No.	Thickness (in)	Avg Longitudinal Cracking (ft/station)					
		1 Month	1 Year	2 Years	5 Years	14 Years	28 Years
1, 5	5	0	2	11	14	47	67
2, 6	4.5	0	0	11	12	46	81
3, 7	4.5	0	2	7	12	36	80
4, 8	5.5	0	5	6	8	13	25
9	6	0	3	7	8	20	NA
10	6[a]	0	2	5	11	42	NA

[a]Air entrained.

Table 6. Profile variation, BPR-type roughometer.

Section No.	Thickness (in)	Profile Variation (in/mile)			Longitudinal Profile Value[b]		
		1955	1979	1981	1955	1979	1981
1, 5	5	112	129	158	3.5	3.3	3.0
2, 6	4.5 mesh	107	110	129	3.6	3.5	3.3
3, 7	4.5	107	132	156	3.6	3.2	3.0
4, 8	5.5	109	123	143	3.5	3.3	3.1
9	6	110	127	144	3.5	3.3	3.1
10	6[a]	105	121	131	3.6	3.4	3.2

[a]Air entrained.
[b]Longitudinal profile value is obtained by correlation with the CHLOE profilometer and use of AASHO road test present serviceability index (PSI) formula. It is the PSI without deduction for cracking and patching.

result in a significant decrease in riding quality. In recent years, full-depth patches have been placed to restore some small broken and distorted areas.

A Greene County cost-accounting program provides information to obtain the cost per mile for both pcc pavement and gravel-surfaced roadways. The maintenance costs per mile for paved and gravel-surface roadways are $1034 and $1365, respectively. A higher level of service (signing, mowing, and winter maintenance) is provided on paved roads than for gravel roads. If signing, mowing, and winter maintenance are not included, the basic maintenance costs for paved and gravel roadways are $483 and $1108, respectively.

Aggregate Demand

Calculation shows that it will take many years for a paved road to result in a reduction of aggregate use. The construction of this roadway required 2450 tons/mile (1 381 061 kg/m) for 4.5-in (11.4-cm) thick and 3270 tons/mile (1 843 294 kg/m) for 6-in (15.2-cm) thick pavement.

Typical Greene County gravel road construction uses 700 tons/mile (394 589 kg/m) the first year, 600 tons/mile (338 219 kg/m) the second year, and 45 tons/mile (25 366 kg/m) each year thereafter. With these figures, it would take 27.6 years for a gravel roadway to use 2450 tons/mile (1 381 061 kg/m) and 45.8 years to use 3270 tons/mile (1 843 294 kg/m).

A paved road must yield a greater than normal life to result in a true reduction of aggregate demand. The paving aggregate, however, is not lost and provides an excellent base for future overlays.

PERFORMANCE

The volume of traffic over this project has been fairly constant over the years. The average daily traffic from 1957 to 1981 was about 260 vehicles/day. Traffic volumes for the 24-year period are given in Table 7. A grain elevator in Farlin increased the amount of truck traffic during harvest season during its operation from 1951 through 1976. A gravel pit operation 0.5 mile (0.8 km) east of Farlin also produced heavier loads on the road (1951-1977).

Table 7. Average traffic.

	Avg Traffic by Mile Number (vehicles/day)				
Year	East 1	2	3	West 4	Avg
1957	250	241	241	265	249
1962	299	294	249	262	276
1967	353	329	271	295	312
1972	236	286	218	188	232
1976	315	272	238	258	271
1981	292	248	195	167	226
Avg	291	278	235	239	261

The 6-in (15.2-cm) diameter cylinders of non-air-entrained concrete averaged more than 5600 lb•f/in² (38.61 MPa) when tested at the age of 28 days. The 28-day modulus of rupture of the beams was 800 lb•f/in² (5.52 MPa). The concrete is of excellent quality 28 years after construction; compressive strength averages 7920 lb•f/in² (54.61 MPa).

The high-pressure air content was determined on 28-year-old cores. The non-air-entrained concrete averaged 3.2 percent and the air-entrained section averaged 6.6 percent.

Typical surface appearance of the pavement is shown in Figure 4 [4.5-in (11.4-cm) nonreinforced] and Figure 5 [6-in (15.2-cm) nonreinforced]. Even though transverse joints were not sawed, the random cracking in the 6-in pavement produced a relatively uniform spacing and an orientation nearly perpendicular to the centerline. Most of the transverse cracking occurred early in the life of the pavement. In the thinner sections, little additional cracking developed after the first two years. Transverse cracks continued to develop in the 6-in pavement through 28 years but at a declining rate.

The 4.5-in (11.4-cm) mesh-reinforced sections with contraction joints exhibit the longest average slab length [22 ft (6.7 m)] after 28 years. The nonreinforced pavements have slab lengths that range from 13 to 18 ft (4.0-5.5 m). The average slab length for the nonreinforced pavement varies directly with the thickness. The 4.5-in pavement has the shortest slab length [13 ft (4.0 m)]; the 6-in pavement has slab lengths of 18 ft. These lengths are just less than the current Iowa Department of Transportation design of 20 ft. Unfortunately, jointed nonreinforced pavement was not included for comparison.

The longitudinal cracking generally varies inversely with thickness, but there are irregularities. These irregularities may be attributed to variations in the grade from inadequate support or grade settlement. The 4.5-in thick pavement, both reinforced and nonreinforced, developed the most longitudinal cracking, but the 81 ft/station average for the mesh-reinforced design results from widely differing data from the two sections of 19 ft/station and 144 ft/station. This, and that the 5 ft 0.5 in (14.0 cm) section exhibits the least longitudinal cracking, would indicate that the longitudinal cracking was dependent on the base. No correlation was obvious between sand-drain locations and subgrade with longitudinal cracking. The cracking at 0.25 point has not contributed to any substantial degree in a loss of service, though it may result in increased maintenance.

The profile variation at 30 years of age ranged from 129 in/mile (204 cm/km) to 158 in/mile (249 cm/km). This is rough for a primary or Interstate route, but is quite adequate for a secondary route. The data exhibit a substantial increase in roughness from 1979 to 1981. Visual observations indicate an extensive sealing effort during this period (1980) that deposited enough material on the surface to adversely affect the riding quality. The material on the surface can be expected to be worn and bladed (winter maintenance) away and a longitudinal profile comparable to 1979 will result. The 4.5-in mesh-reinforced section exhibits the smoothest profile. No joint heaving or slab warping is apparent.

Figure 4. Typical pavement condition of 4.5-in nonreinforced slab in section 3.

Figure 5. Typical pavement condition of 6-in nonreinforced slab in section 9.

Table 8. Pavement costs in 1955 bid prices.

Section No.	Thickness (in)	Cost per Yard² ($)
1, 5	5	3.15
2, 6	4.5 mesh	3.42
3, 7	4.5	3.04
4, 8	5.5	3.26
9	6	3.38
10	6[a]	3.38

[a] Air entrained.

COSTS

The cost of each section is listed in Table 8. These prices were greater than normal costs of pavement at the time of construction because of the extra work involved due to the research and short sections.

CONCLUSIONS

The conclusions drawn from this research are as follows:

1. All design thickness, from 4.5 to 6 in (11.4-15.2 cm), provided quite adequate service for a low-volume secondary roadway with minimal maintenance for 30 years.
2. The 4.5-in (11.4-cm) thick pavement has resulted in a slight reduction of aggregate use when compared with requirements for an unpaved gravel roadway, but the 6-in (15.2-cm) pavement results in a substantial increase in aggregate use.
3. The 4.5-in thick mesh-reinforced section has provided the best overall performance.
4. Slab lengths of the nonreinforced sections without contraction joints vary directly with thickness and are all just less than the current design length of 20 ft (6.1 m).
5. The amount of longitudinal cracking varies inversely with the thickness of pavement.
6. The cost of maintaining a paved road is less than that for a gravel roadway. If only basic maintenance is provided, the cost for a paved road may be less than half that for a gravel-surfaced roadway.

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Pavement Evaluation and Upgrading of Low-Cost Roads

JACOB GREENSTEIN

Design, construction, and performance experience on nonsurfaced gravel low-cost roads show that, even when proper design and construction procedures are followed, unexpected early failure often occurs. These modes of failure generally occur after the wet season and are normally attributed to the development of unexpected, very high moisture content in the subgrade. This moisture buildup in localized subgrade areas cannot be predicted and is frequently attributed to changes in topography, drainage, and environmental factors that take place during and after construction. This paper presents a new methodology to minimize unexpected pavement failure and to upgrade the underdesigned road sections. The methodology is based on the subgrade strain criterion. The strain characteristics developed on top of the subgrade are calculated for a given dynamic loading and therefore permit the determination of the location of these high-moisture and low-strength sections of the pavement. The proposed methodology also permits the calculation of the amount of roadway reinforcement required to achieve uniform roadway strain characteristics. The uniform strain should obviously result in a uniform life expectancy of the roadway. The automated instrumentation for making this strain inventory is now being used in Thailand, several countries in Latin America, and elsewhere. It has proven to be simple, economical, and practical.

Many unsurfaced roads constructed in accordance with the best design technology have failed well before their predicted life expectancy. Such an unexpected failure is shown in Figure 1. Normally, these failures are attributed to the development of very high moisture content in the subgrade, moisture substantially greater than the in situ moisture content found in the natural subgrade prior to construction. This moisture buildup in localized subgrade areas cannot be predicted; it is frequently attributed to changes in topography, drainage, and environmental factors that develop during and after construction. It is rarely assumed that all of the natural subgrade would become saturated because this would lead to conservative, low-strength values for the natural subgrade that would generate thick and costly pavement.

In conventional design methods for unsurfaced roads, the stability of the pavement is ensured through material quality requirements for the subbase or the base materials. By contrast, only the subgrade strain criterion is represented qualitatively in the determination of pavement thickness. In other words, under given traffic and environmental loadings, the subgrade strain developed at the top of the subgrade must be identical along the road. The magnitude of the subgrade strain is affected strongly by environmental factors, which corresponds to the conclusions of numerous field studies: The principal failure cause, particularly in areas of high rainfall, is due to underground springs that pump moisture from a high-water table, which causes saturation of the subgrade due to flooding of side ditches during the rainy season.

The equipment and methodology have now become available for determining the strain characteris-

Figure 1. Failure of unsurfaced road.

tics. High-moisture, high-deflection, or high-strain areas can be located and the roadway reinforcement required to achieve uniform roadway strain characteristics can be determined.

DESIGN CONSIDERATIONS

The basic objective of the design procedure is to provide a structure made of local materials that will be suitable to carry the anticipated traffic loadings under a given environmental condition. To achieve this goal, the following five phases must be completed.

1. Design criteria--limited strain, limited stress, and smooth ride;
2. Data collection--traffic and environmental loadings and subgrade and material properties;
3. Calculation and evaluation of the pavement system--subgrade and material properties, thickness, and stresses and strains;
4. Design conclusion--structural section, lifetime, and material specifications; and
5. Early performance study--immediately after the construction stage (generally 0.5-2 years after completion of construction).

Most design methods of low-volume roads (1-6) include only the first four phases; that is, the structural section and lifetime performance curves are analyzed before the construction stage. Thus, unexpected environmental conditions and material performance after the completion of the project may cause early and unexpected failure of the pavement (see Figure 1).

DESIGN CRITERIA

Use of Subgrade Strain Criteria in Conjunction with California Bearing Ratio Methodology

Most design methods are variations of the California bearing ratio (CBR) method. The basic concept is an empirical application of the Boussinesq theory of homogeneous media to prevent shear failure in the subgrade (7-10). CBR methodology is widely used, and its values are simple to measure in the field or laboratory. Subgrade strain is difficult to measure.

Since subgrade strain is used in this paper as the principal criterion of design, the ratio between CBR and vertical strain must be shown to be constant for any material (e.g., for a given modulus E). The fundamental CBR formula stipulates that the ratio between the allowed maximum shear stress (τ_{max}) in the subgrade and its CBR should be constant:

$$\tau_{max}/CBR = D \quad (1)$$

Results of studies that use this methodology indicate that the relation between flexible pavement thickness (H) to wheel load (P) and the contact pressure (p) is as indicated in Equation 2 (10).

$$H = \left\{P[(0.75/\pi D) \cdot (1/CBR) - (1/p\pi)]\right\}^{0.5} \quad (2)$$

The factor $0.75/\pi D$ is determined empirically as 1/8.1 for most materials that have a CBR less than 12. Therefore, the constant D in Equation 1 is equal to 1.93.

In an elastic linear material that has Poisson's ratio of 0.5 and for a state of axisymmetric loading there exists a simple relation between the vertical strain (ϵ_z) and the maximum shear stress (τ_{max}) (10):

$$\tau_{max}/\epsilon_z = E/2 \quad (3)$$

where E denotes the modulus of elasticity.

Substitution of Equation 3 into Equation 1 gives:

$$CBR/\epsilon_z = E/2D \quad (4)$$

Equation 4 indicates that for a given media (e.g., E = constant) the ratio between the CBR and the vertical strain is constant.

Subgrade Strain Criterion

Most design methods that incorporate the results from the American Association of State Highway Officials (AASHO) road test of 1962 (11) use rutting as a failure criterion of flexible pavement. Rutting is considered to occur only on the subgrade and is controlled by limiting the value of the vertical compressive strain of the top subgrade. This implies that the pavement layers above the subgrade will be structurally adequate so that only negligible plastic deformation will occur with each layer. The Shell method (12) is associated with ultimate rut depths on the order of 0.75 in (19 mm).

The subgrade strain criterion was verified by the Waterway Experiment Station (WES) after analysis of pavement performance of test sections (13,14), and the range of the WES criterion is given in Figure 2, which presents a comparison of other strain criteria (12,15-18). Figure 2 shows that the Shell strain criterion (12) is in the range of the WES criterion. On the other hand, the criterion developed by Brabston and others (15) shows lower strain values for the same number of repetitions than recommended by the WES. The differences in the recommended criteria are mainly due to the following reasons.

1. The dynamic modulus of the subgrade and the granular materials is determined by different approaches, as shown in Figure 3, which presents the relation between the dynamic modulus and the subgrade CBR according to WES (19), Shell (20), and Wiseman and others (21). For example, a subgrade CBR of 5 percent, the dynamic modulus ($\times 10^3$ lb•f/in^2) is 17.0 (19), 7.8 (20), and 7.0-10.0 (21).
2. Local environmental conditions affect the design parameters differently.

METHODOLOGY OF PAVEMENT EVALUATION

To a certain extent, the structural evaluation of a pavement system is an inverted design process. If the pavement cross section and properties of the paving materials and subgrade soil are known, it is possible to compute pavement responses (stresses, strains, and deflections) under a given load at any point within the structure. In the evaluation process, the response of the pavement is observed and material properties are back-calculated. Of the different responses of the pavement to load, the only practical measurements are deflections. For structural evaluation the real pavement-subgrade system is replaced by a mathematical model, and the measured surface deflections are used as input to back-calculate the model's parameters. Two mathematical models are used in order to evaluate the elastic modulus of the subgrade and the pavement.

First Mathematical Model

The first mathematical model is a thin plate (to represent the pavement) supported on an elastic foundation (the subgrade). By using this theory (21,22), the pavement and its supports are expressed in terms of the following basic pavement parameters.

Flexibility

The pavement flexibility (F) is the center deflec-

Figure 2. Comparison of subgrade strain criteria (ϵ_z).

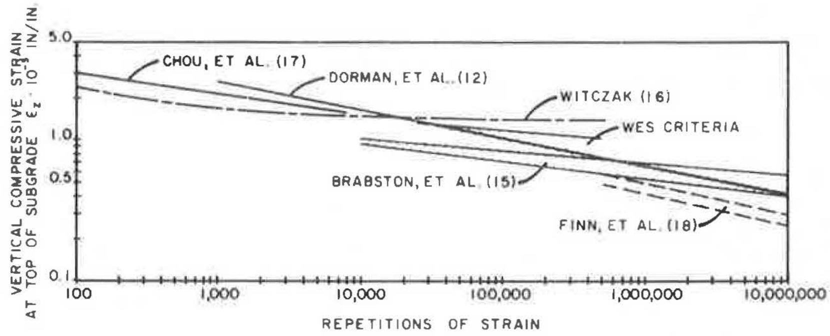
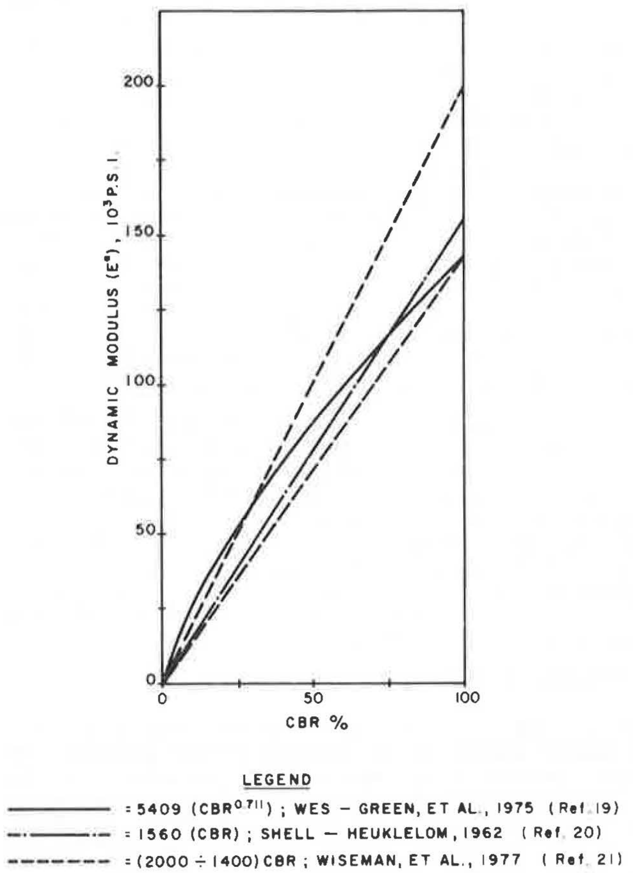


Figure 3. Relationships between dynamic modulus and CBR.



tion per unit force and is reported in micrometers per kiloNewton (μm/kN).

$$F = \Delta_0/P \tag{5}$$

where Δ_0 is the center deflection (μm) and P is the total load (kN), which equals peak to peak force when a dynamic device is used.

Characteristic Length

This characteristic length (ℓ) presents the shape of the deflection basin and is determined according to Figure 4, which shows the relation between ℓ, the offset distance r, and the deflection ratio (Δ_r/Δ_0).

Subgrade Modulus

The subgrade modulus (E_s) can be computed by using the measured pavement flexibility (F) and the characteristic length (ℓ) by the following equation (in megaNewtons per square meter):

$$E_s = 10^6/4.14\,F\ell(S/S_0) \tag{6}$$

where

F = measured flexibility (μm/kN),
ℓ = characteristic length (mm), and
(S/S_0) = ratio of the measured pavement stiffness (S) to the theoretical point load stiffness (S_0), which is a function of ℓ and geometry of load application, stiffness being defined as the required load to cause unit deflection.

For the 18-in (46-cm) diameter plate used in this study, S/S_0 is determined by the following equation (22):

$$S/S_0 = 68\,(0.1\ell + 5)^{2.06}/\ell^2 \tag{7}$$

Substitution of Equation 7 and the results of Figure 4 for r = 50 cm in Equation 6 results in the relation among E_s/p, F, and Δ_{50}/Δ_0 (p denotes the contact pressure). This relation is presented in Figure 5a. According to this model, the relation between the characteristic length per pavement thickness (ℓ/H) and E_p/E_s is determined by the following equation (21,22)

$$\ell/H = [E_p\,1.5/E_s\,12\,(1 - \mu^2)]^{1/3} \tag{8}$$

Equation 8 and Figure 4 are applied in this study to determine the relation among the deflection ratio (Δ_{50}/Δ_0), E_p/E_s, and H/a. This relation is presented in Figure 6a. E_p and E_s denote the elastic modulus of the pavement and subgrade, respectively. H is the pavement thickness, and a is the radius of the circular area.

Second Mathematical Model

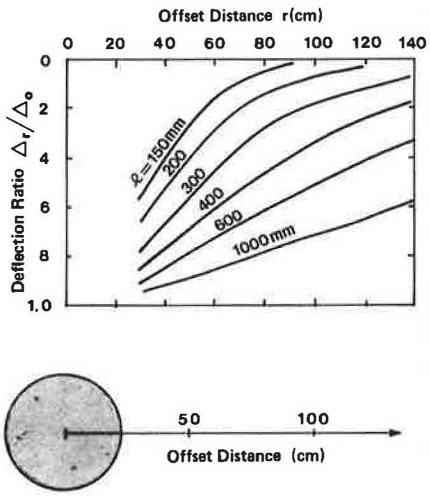
The second mathematical model used in this study is based on a two-layer linear elastic model by Burmister (23). This model is used to determine the vertical strain (ϵ_z) on top of the subgrade. The results are presented in Figure 6b in the form of a nondimensional relation between the subgrade strain factor ($\epsilon_z E_s/p$), the modulus ratio (E_p/E_s), and H/a. Thus, by combining Figures 6a and b, it is possible to determine the relation among Δ_{50}/Δ_0, H/a, and $\epsilon_z E_s/p$. In a similar way, the combining of Figures 5a and b enables the determination of the relation among the deflection ratio Δ_{50}/Δ_0, the flexibility (F), and the vertical strain on top of the subgrade (ϵ_z).

The following example is presented to demonstrate how to determine the subgrade strain by means of Figures 5 and 6:

$F = 12\ \mu m/kN$ and $\Delta_{50}/\Delta_0 = 0.41$ were determined under an 18-in (46-cm) plate loaded with a dynamic contact pressure of $p = 67.77\ kN/m^2$. The total thickness of the pavement is 32 cm. By using Figure 6 (see example 1) for $\Delta_{50}/\Delta_0 = 0.41$ and $H = 32$, the following two nondimensional parameters can be determined: $E_p/E_s = 2.15$ and $\varepsilon_z E_s/p = 0.29$. The second step is to determine the subgrade modulus and the vertical strain on the top of the subgrade by means of Figure 5. In example 1, the result is

$$E_s/p = 1.07 \times 10^3 \text{ (or } E_s = 1.07 \times 10^3 \times 67.7 = 72.5\ 10^3\ kN/m^2\text{) and } \varepsilon_z = 2.7\ 10^{-4}.$$

Figure 4. Deflection bowl, Hogg model–18-in diameter plate.



Note that the principles of the methodology presented in this section might be applied to different nondestructive equipment, such as the road rater, Benkelman beam, Dynaflect, and falling weight deflectometer. Nevertheless, in this paper, only the application of the road rater is presented (see Figure 7).

EVALUATION AND UPGRADING OF UNSURFACED ROADS

The design lifetime of unsurfaced pavements in developing countries such as Ecuador, Bolivia, and Thailand varies between 5 and 8 years. The lower limit is used for laterite roads and the upper limit for granular-based roads. In order to eliminate unexpected early failure performance, analysis is conducted in these countries during the early service period to locate the underdesigned road sections.

Performance Analysis and Upgrading of Laterite Roads

In this case, 18 cm of laterite were designated (2) to carry 25 000 standard axles/lane over a silty subgrade that has a design CBR of 6.5 percent in Thailand. About 8 months after the completion of the project and after the major rainy season, a dynamic deflection, which used a pavement profiler with 46-cm diameter plate, was conducted along this unsurfaced road. The following representative deflection parameters were determined for two sections of the road:

1. Section A (about 75 percent of the length of the road)--Representative value of flexibility (F) = 13.2 μm/kN; deflection ratio (Δ_{50}/Δ_0) = 0.185; and pavement thickness (H) = 18 cm.
2. Section B (about 25 percent of the length of

Figure 5. ε_z versus E_s/p, F, Δ_{50}/Δ_0.

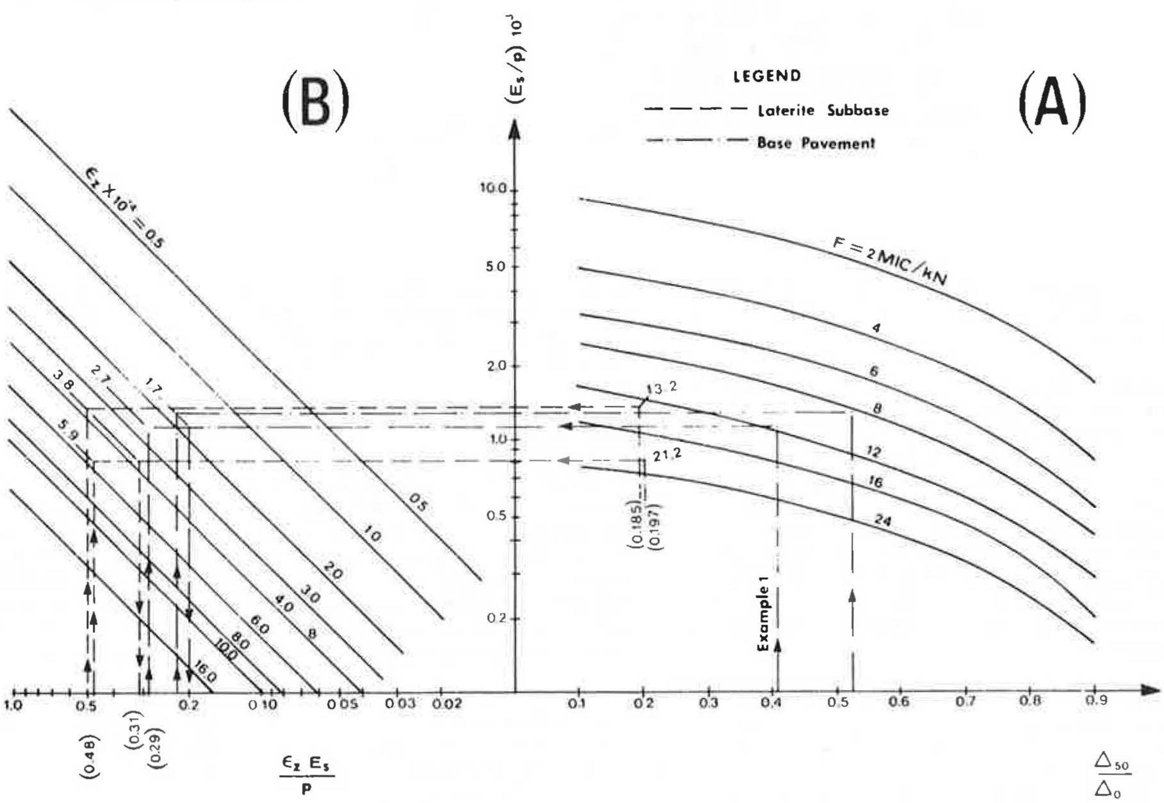
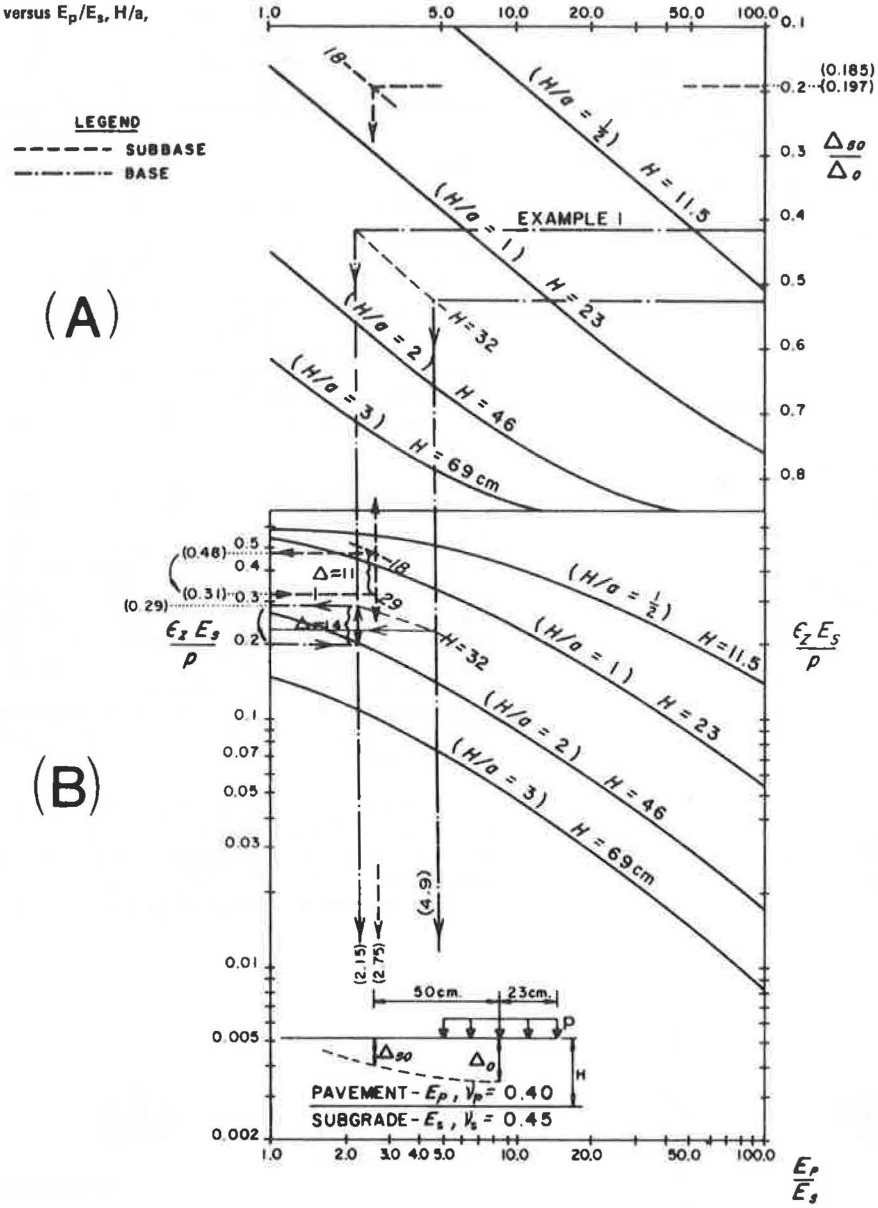


Figure 6. $\epsilon_z E_s/p$ versus E_p/E_s, H/a, Δ_{50}/Δ_0.



the road)--Representative value of flexibility (F) = 21.2 μm/kN; deflection ratio (Δ_{50}/Δ_0) = 0.197; and pavement thickness (H) = 18 cm.

By using the charts in Figures 5 and 6, the design parameters shown in Table 1 were determined.

These results emphasize that the subgrade modulus in Section B is lower than that in Section A. Therefore, the subgrade strain is significantly higher ($\epsilon_z = 5.9 \times 10^{-4} > 3.8 \times 10^{-4}$). In order to avoid an earlier unexpected failure of Section B, the subgrade strain determined by the pavement profiler must be reduced from 5.9×10^{-4} to 3.8×10^{-4}. This reduction can be achieved by increasing the pavement thickness according to Figures 5 and 6 in the following way:

1. Use Figure 5 to determine the subgrade strain factor ($\epsilon_z E_s/p$). When $E_s/p = 0.81$ and ϵ_z is reduced from 5.9×10^{-4} to 3.8×10^{-4}, the subgrade strain factor varies from 0.48 to 0.31.
2. Use Figure 6 to determine the required pavement thickness (H). When $E_p/E_s = 2.75$, and the strain factor varies from 0.48 to 0.31, H varies from 18 to 29 cm. Therefore, the additional thickness is 29 cm - 18 cm = 11 cm.

Performance Analysis and Upgrading of Granular Base Roads

A 32-cm section of granular base that has CBR > 80 percent was designated to carry 100 000 standard axles/lane over silty clay subgrade (3). During the first rainy season, a short section (about 500 m in length) started to fail; consequently, a dynamic deflection survey was conducted on the failed and unfailed sections of the road. The following representative deflection parameters were described:

1. Failed section (about 5 percent of the road)--Flexibility (F) = 12.0 μm/kN; deflection ratio (Δ_{50}/Δ_0) = 0.41; and pavement thickness (H) = 32 cm. From Figures 5 and 6, $\epsilon_z = 2.7 \times 10^{-4}$, $E_s/p = 1.07 \times 10^3$, and $E_p/E_s = 2.15$.

Figure 7. Pavement evaluation of unsurfaced roads.

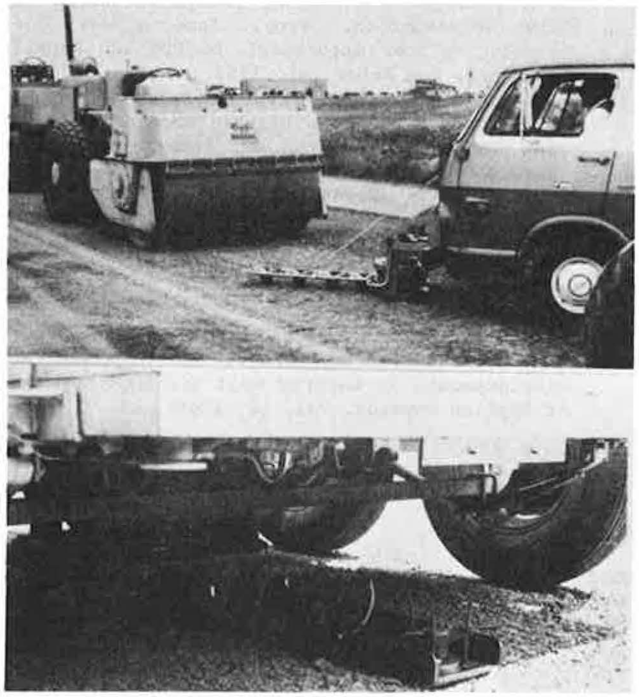


Table 1. Analysis of deflection survey of laterite road.

	Design Parameter	
Road Section	After Figure 6	After Figure 5
A	$\epsilon_z E_s/p = 0.5$ $E_p/E_s = 2.70$	$\epsilon_z = 0.000\ 38$ $E_s/p = 1320$ $E_s = 89\ 500\ kN/m^2$
B	$\epsilon_z E_s/p = 0.48$ $E_p/E_s = 2.75$	$\epsilon_z = 0.000\ 59$ $E_s/p = 810$ $E_s = 54\ 900\ kN/m^2$

Notes: E_z = vertical strain, E_s = elastic modulus of the subgrade, p = contact pressure, and E_p = elastic modulus of the pavement.

2. Unfailed section--Flexibility (F) = 8.5 µm/kN; deflection ratio (Δ_{50}/Δ_0) = 0.52; and pavement thickness (H) = 32 cm. From Figures 5 and 6, ϵ_z = 1.7×10^{-4}, E_p/E_s = 4.9.

In order to upgrade the failed section, additional pavement thickness is required to reduce the subgrade strain from 2.7×10^{-4} to 1.7×10^{-4}. This can be achieved in the following way:

1. Use Figure 5 to determine the subgrade strain factor $\epsilon_z E_s/p$. When $E_s/p = 1.07 \times 10^3$ and ϵ_z is reduced from 2.7×10^{-4} to 1.7×10^{-4}, $\epsilon_z E_s/p$ varies from 0.29 to 0.20.
2. Use Figure 6 to determine the required pavement thickness (H). When E_p/E_s = 2.15 and the subgrade strain factor varies from 0.29 to 0.20, H varies from 32 to 46 cm. Therefore, the additional required thickness is 14 cm of granular-base course.

CONCLUSIONS

Early performance study of low-cost roads has proved to be an economical and practical means to avoid unexpected early failure. A pavement evaluation process is used to determine the response of the pavement and subgrade system to dynamic loading.

The two-layered pavement-subgrade system is replaced by a simple mathematical model. The deflection basin is determined by nondestructive means and the results are used as input to back-calculate the model's pavement.

The equipment and methodology have now become available for determining the strain characteristics. This methodology permits a precise determination of where the high-moisture, high-deflection, or high-strain areas are located.

The subgrade strain criterion is used to determine the minimum overlay thickness required to achieve uniform roadway strain characteristics. This technique is being applied in Thailand, Nepal, and Latin America to laterite and gravel roads that have a pavement design CBR of 30 to 80 percent.

REFERENCES

1. Road Note 31--A Guide to the Structural Design of Bituminous Surfaced Roads in the Tropical and Subtropical Countries. Department of the Environment, Road Research Laboratory, Crowthorne, Berkshire, England, 1977.
2. J. Greenstein and M. Livneh. Pavement Design of Unsurfaced Roads. TRB, Transportation Research Record 827, 1981, pp. 21-26.
3. J. Greenstein and M. Livneh. Design Thickness of Low-Volume Roads. TRB, Transportation Research Record 702, 1979, pp. 39-46.
4. M. Attal and others. Manual on the Planning of Labour-Intensive Road Construction. International Labour Organization, Geneva, Switzerland, 1977, p. 38.
5. L. Odier and others. Low-Cost Roads: Design, Construction and Maintenance. Butterwords, London, 1971, p. 79.
6. R.G. Ahlvin and G.M. Hammitt, II. Load-Supporting Capability of Low-Volume Roads. _In_ Low-Volume Roads, TRB, Special Rept. 160, 1975, pp. 198-203.
7. W.J. Turnbull and R.G. Ahlvin. Mathematics Expression of the CBR Relations. Proc., 4th International Conference on Soil Mechanics and Foundation Engineering, London, 1957, pp. 178-180.
8. G. Wiseman and J.G. Zeitlen. A Comparison Between the CBR and the Shear Strength Methods in the Design of Flexible Pavements. Proc., 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, 1961, pp. 359-363.
9. R.G. Ahlvin, Y.T. Chou, and R.L. Hutchinson. Structural Analysis of Flexible Airfield Pavements. Proc., ASCE Transportation Engineering Journal, Vol. 100, TE3, 1974, pp. 625-641.
10. J. Uzan, M. Livneh, and I. Ishai. Thickness Design of Flexible Pavements with Different Layer Structures. Australian Road Research, 1980.
11. The AASHO Road Test. HRB, Special Rept. 73, 1962, 438 pp.
12. G.M. Dormon, J.M. Edwards, and J.E.D. Kerr. The Design of Flexible Pavements. Presented at the Annual General Meeting of the Engineering Institute of Canada, Banff, Alberta, Canada, May 1964.
13. R.G. Ahlvin and others. Multiple-Wheel Heavy Gear Load Pavement Tests: Basic Report. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Tech. Rept. S-71-17, Vol. 1, Nov. 1971.
14. C.D. Burns and others. Comparative Performance of Structural Layers in Pavement Systems: Design, Construction, and Behavior Under Traffic

of Pavement Test Sections. Federal Aviation Administration, Rept. FAA-RD-73-198, Vol. 1, WES TR S-74-8, June 1974.
15. W.N. Brabston, W.R. Barker, and G.G. Harvey. Development of a Structural Design Procedure for All-Bituminous Concrete Pavements for Military Roads. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Tech. Rept. S-75-10, July 1975.
16. M.W. Witczak. Design of Full-Depth Asphalt Airfield Pavements. Asphalt Institute, College Park, MD, Res. Rept. 72-2, April 1972.
17. Y.T. Chou, R.L. Hutchinson, and H.H. Ulery, Jr. Design Method for Flexible Airfield Pavements. TRB, Transportation Research Record 521, 1974, pp. 1-13.
18. F.N. Finn, K. Nair, and C.L. Monismith. Applications of Theory in the Design of Asphalt Pavements. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, 1972.
19. J.L. Green and J.W. Hall, Jr. Nondestructive Vibratory Testing of Airport Pavements; Experimental Test Results and Development of Evaluation Methodology and Procedure. Federal Aviation Administration, Rept. FAA-RD-205, Vol. 1, WES TR S-75-14, Sept. 1975.
20. W. Heukelom and A.J.G. Klomp. Dynamic Testing as a Means of Controlling Pavements During and After Construction. Proc., International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, MI, 1962.
21. G. Wiseman, J. Uzan, M.S. Hoffman, I. Ishai, and M. Livneh. Simple Elastic Models for Pavement Evaluation. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Vol. 2, Ann Arbor, MI, 1977.
22. G. Wiseman. The Interpretation of Surface Deflection Measurements Using the Model of an Infinite Plate on an Elastic Foundation. Symposium on Nondestructive Test and Evaluation of Airport Pavement, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Nov. 1975.
23. D.M. Burmister. The Theory of Stresses and Displacements in Layered Soil Systems. Journal of Applied Physics, Vol. 16, 1945.

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Pavement Design for a 3.5-Million-Pound Vehicle

WILBUR CHARLES GREER, JR.

The investigative and analysis procedures used to evaluate roadways for the movement of two nuclear reactor vessels from Knoxville, Tennessee, to the Tennessee Valley Authority's (TVA) Phipps Bend power generation plant are presented. The route included 74 km (46 miles) of state, county, and private roadways, both paved and unpaved. The total weight of each reactor pressure vessel and its transport trailers was approximately 13 345 kN (3 million lb). The five prime movers brought the total weight for each move to approximately 15 569 kN (3.5 million lb). The transport trailer was supported on 24 axle lines with 16 wheels/axle line. The methods of compiling existing construction data, field testing, and analysis procedures are discussed. The application of proofrolling to verify the results of the analyses is also presented. Deflection and density testing, both before and after the moves, were performed by the Tennessee Department of Transportation. An analysis of these data indicates no serious effects due to the two moves that occurred in 1980 and 1981. A comparison between the results of the U.S. Army Corps of Engineers procedures originally used to evaluate pavement thickness requirements and the results of layered elastic computer program analyses after the second move is presented. The results of the two methods compare favorably.

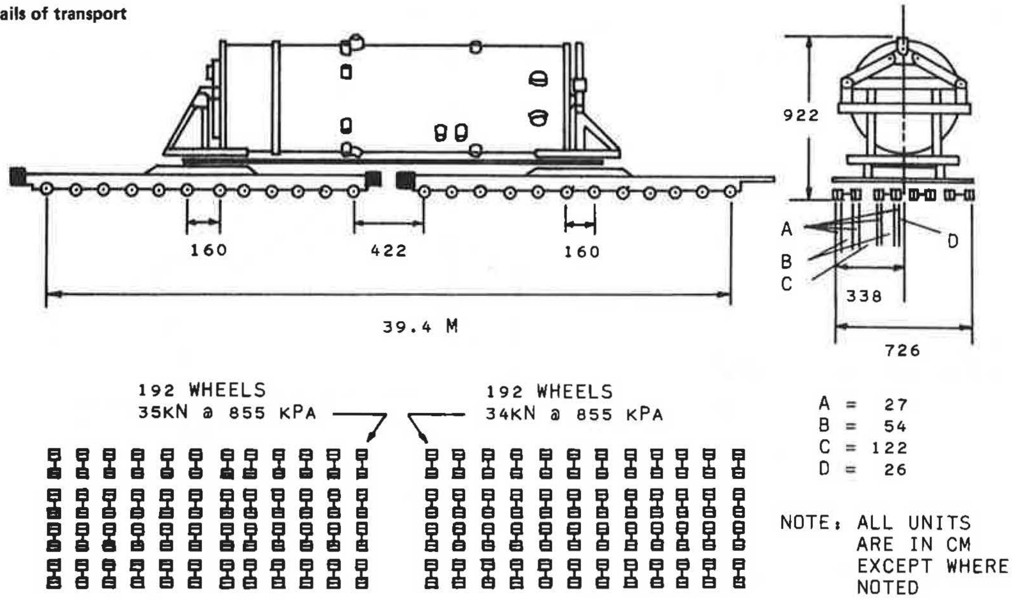
The construction of large industrial manufacturing plants and power-generation facilities may require that very heavy mechanical items be brought to the site in one piece. Items such as generators and nuclear reactor pressure vessels may weigh a few thousand kilonewtons (several hundred thousand pounds) to in excess of 9000 kN (2 millions lbs). Most projects may have only one or two such pieces of equipment to move. Where possible, these items can be brought to the site by water or rail transportation; however, the equipment has to be transported across paved roads to many sites. The vehicles used to move the equipment often have many axles and numerous tires per axle in order to spread the load and reduce individual tire loads to acceptable levels. The high axle loads, numerous tires, and very low number of applications complicate the analysis of the pavement structures.

The Tennessee Valley Authority's (TVA) Phipps Bend nuclear power generation plant is located in northeast Tennessee. The plant, currently under construction, will have two nuclear units when completed. The steel reactor pressure vessel (RVP) for each unit was fabricated in Memphis nearly 692 km (430 miles) away. Each RPV weighs 9190 kN (1033 tons) and thus just cannot be loaded on a normal truck and hauled to the plant site. Plans for the transport of the RPVs to the plant site called for each RPV to be barged to Knoxville and then hauled over 74 km (46 miles) of state, county, and private roadways to the plant site. These are believed to be some of the heaviest, if not the heaviest, loads ever moved over U.S. roads.

The VSL Corporation, the transport contractor, authorized an investigation and evaluation of the 74 km (46 miles) of roadways to be traversed relative to each pavement's ability to withstand the loads imposed through the transport trailers. The objectives of the investigation were to investigate and develop recommendations with regard to pavements and to the geotechnical aspects of the travel route, of planned detours, and of planned road widenings. The basic criteria for a successful move were defined as the following:

1. Prevent, as economically as possible, the transport trailers from becoming stuck; stoppage could result in enormous delay costs; and
2. Minimize visible and measurable damage to the roadways that could result in significant damage charges against the contractor.

Figure 1. Details of transport assembly.



The investigation concentrated on identifying and evaluating potential critical pavement sections along the route that might impede the transport of the RPVs or that might experience intolerable damage due to the transportation effort. The first RPV was moved successfully in July 1980, and the second RPV was moved successfully in July 1981.

TRANSPORT ASSEMBLY

The transport assembly for each RPV was comprised of two main trailer sections. Each main trailer section was approximately 19.5-21.3 m (64-70 ft) long and 7.6 m (25 ft) wide. When assembled, the transport assembly was 47 m (154 ft) long with 24 axle lines and 16 tires/axle line (384 tires in total).

The transport trailers and RPV together weighed 13 247 kN (1489 tons). This load was transferred to the pavement surface through the 384 tires. The inflation pressure of each tire was 855 kPa (124 lb•f/in²). The load on each tire on the front trailer was approximately 34 kN (7590 lb•f) and the load on each tire on the rear trailer was approximately 35 kN (7890 lb•f). Details of the transport assembly and tire spacings are shown in Figure 1.

The transport trailers were pulled by three tractors and pushed by two more tractors. The typical tractor type (Michigan) had four wheels and weighed 436 kN (49 tons). The tires had an inflation pressure of 234 kPa (34 lb•f/in²). The axle configuration was single axle-single wheel. The wheel spacing was 274 cm (9 ft) and the axle spacing was 330 cm (10 ft 10 in). The speed of the transport assembly was a maximum of 4.8 km/h (3 mph).

INVESTIGATIVE PROCEDURES

In order to evaluate the existing pavements, it was necessary to determine the existing thickness of each pavement and the subgrade support conditions. Because of the relatively long total length of roadways to be investigated, typical pavement thickness profile data for both the traveled way and shoulders were compiled for each roadway by reviewing the available construction plan drawings from the files of the agencies responsible for each roadway.

The thickness profiles were supplemented by drilling 49 soil test borings along the proposed route to verify that the in-place thicknesses for each pavement section generally corresponded to the construction plan drawings. The borings generally were extended to a depth of 3 m (10 ft) below the ground surface. The borings also were used to develop pavement thickness profiles where little or no information from construction plan drawings was available.

A total of 26 different pavement sections were identified along the proposed route. The pavement thickness profiles ranged from as little as 2.5 cm (1-in) of bituminous concrete over 10 cm (4 in) of crushed stone on county roads to as much as 38 cm (15 in) of bituminous concrete over 30 cm (12 in) of crushed stone on the state routes. The state routes had been overlaid numerous times in the past.

Field California bearing ratio (CBR) tests were performed near the top of the subgrade in 42 of the borings to determine the in-place CBR values.

THICKNESS ANALYSIS PROCEDURES

The pavement thickness design procedures considered in the premove analysis are discussed elsewhere (1-6). The procedures for flexible pavements based on the U.S. Army Corps of Engineers design equation (1, Equation 1) were chosen as the basis for the analysis and evaluation:

$$t = [(0.23 \log C) + 0.15] \sqrt{P\{[1/8.1(\mathrm{CBR})] - (1/p\pi)\}} \qquad (1)$$

where

- t = thickness of total pavement structure (in),
- C = a measure of traffic called coverages,
- P = equivalent single wheel load (lb),
- CBR = a measure of soil strength or support capability, and
- p = tire contact pressure (lb•f/in²).

The equation shown is not the latest one available but, as will be discussed later, it provided acceptable and more economical pavement sections than did the latest equation.

Subgrade Strength (CBR)

As would be expected along a number of different

Figure 2. Frequency histogram of in-place CBR values.

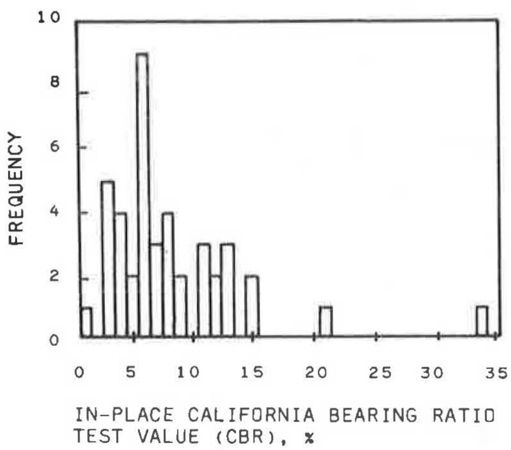
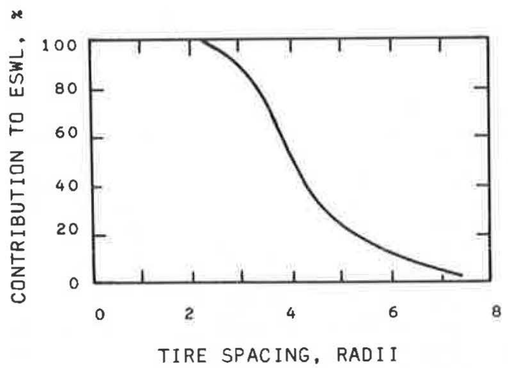


Figure 3. Contribution of individual wheel loads to ESWL.



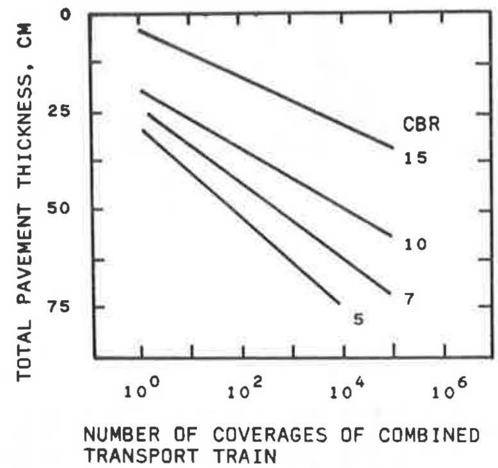
roadway pavement sections, particularly for the length in this project, a wide range of in-place CBR values were obtained. Figure 2 shows the frequency and range of CBR values.

Because of the wide range in the CBR values and that only one or two CBR values generally were available for each of the 26 sections of roadway, a design CBR value based on the data for the complete length of roadway was chosen for evaluation rather than trying to develop a design CBR value for each section. It was thought that the variation in CBR values for the whole route would be indicative of the variation in CBR values within any of the 26 sections of roadway. A CBR value for which 75 percent of the test values was equal to or greater than that value was chosen as the design value. The 75 percent figure is less conservative than the 85-90 percent figure normally used by most designers (6). However, I felt justified in using the less conservative CBR value because the entire route was to be proofrolled with one simulated axle line in order to detect exceptionally soft areas. Based on these criteria, a design CBR value of 5 was selected.

Equivalent Single Wheel Load

Equivalent single wheel loads (ESWLs) were calculated for the push-pull vehicles and the interior and edge wheels of the main trailers based on the tire loads, spacings, and pressures previously discussed and the influence chart in Figure 3 (1). The edge wheels of the main trailer were evaluated to

Figure 4. Coverages of transport train versus total pavement thickness.



determine shoulder requirements should the vehicle inadvertently be steered onto the shoulder. The calculated ESWLs are given in the table below (note: 1 lb•f = 4448 kN; 1 lb•f/in² = 2143 kPa).

Vehicle	Wheel	Actual Tire Load (lb•f)	Tire Pressure (lb•f/in²)	ESWL (lb•f)
Michigan push-pull	NA	24 000	34	25 325
Main trailer	Interior	7 890	124	19 830
Main trailer	Edge	7 890	124	18 720

The ESWL varies with depth and thickness profile of the pavement as well as with the criteria (equal surface deflections, strains, stresses or volumes of deflection basin) for the ESWL determination. However, because of the large number of greatly different pavement sections, the simplified procedures based on Figure 3 for determining an ESWL were considered reasonable, particularly when one considers that a doubling or tripling of the pavement thickness may only change the ESWL by 20-25 percent. This 20-25 percent change in ESWL will only change the thickness requirement for the pavement by approximately 10 percent.

Thickness Requirements Versus Coverages

Graphs of thickness versus allowable coverages of each type of ESWL were developed for the design CBR value of 5. Trial and error procedures then were used to determine the thicknesses necessary to allow various coverages of the transport assembly and prime movers. One pass of the transport assembly and prime movers included 24 coverages of the main trailer ESWL and 10 coverages (5 vehicles, 2 axles) of the Michigan ESWL. Figure 4 presents the required total pavement thickness versus coverages of the transport train for various CBR values.

REQUIRED PAVEMENT THICKNESS AND COMPOSITION

For new pavement construction (detours and widenings), county and private roads, and shoulders, the required pavement thickness was determined based on three passes (factor of safety = 1.5 for the two actual passes) of the transport trailer and prime movers. For state routes, it was assumed that pavement thicknesses capable of carrying many passes (10 or more, factor of safety = 5 for the two actual passes) of the transport trailers and prime moves were necessary because of the large volume of traffic they would have to carry once the moves of the RPVs were completed. The required total thicknesses determined for a CBR value of 5 are presented in the table below (note: 1 in = 2.54 cm).

Roadway Type	Required Total Pavement Thickness (in) Traveled Way	Shoulder
County, private, detours, and widenings	14	14
State	16	14

The U.S. Army Corps of Engineers has updated Equation 1; however, an analysis of the project with this updated procedure resulted in pavement thicknesses 5-8 cm (2-3 in) greater than those for the older procedure when the number of passes of the combined transport train was less than 100. When the number of passes exceeded 100, the pavement thicknesses with updated procedure were less than those for the older procedure. If the updated procedure had been used the contractor would have had to expend money to build pavements thicker than necessary.

The U.S. Army Corps of Engineers procedure also yields the total thickness of the cover required over the subgrade regardless of the type of materials used as cover. Based on my practical experience, it was concluded that the cover could consist of well-compacted, well-graded crushed stone, provided the crushed stone was overlain by a minimum of 10 cm (4 in) of well-compacted bituminous concrete. This results in basic pavement thickness profiles of 10 cm of bituminous concrete over 25 cm (10 in) of crushed stone and 10 cm of bituminous concrete over 30 cm (12 in) of crushed stone for the two types of roadways. The relatively small difference in thickness is due to the fact that the required thickness is a function of the log of the traffic. The bituminous concrete was recommended in order to minimize slippage when the tractor wheels started from a dead stop. However, several temporary detour areas were crossed by the transport train, where the pavement consisted of only crushed stone and no problems were encountered.

For those pavements evaluated that had more than the minimum of 10 cm (4 in) of bituminous concrete, it was judged that the total thickness requirement (i.e., crushed stone thickness) could be reduced. Based on my practical experience, it was decided to count 2.5 cm (1 in) of bituminous concrete as 5 cm (2 in) of crushed stone when calculating equivalent pavement thickness. The required equivalent pavement thicknesses are presented in Table 1. The bituminous concrete to crushed stone conversion was not considered valid when the thickness of crushed stone was less than 15 cm (6 in).

It was recommended that all roads that had actual thicknesses or equivalent thicknesses less than those indicated in Table 1 be overlaid to bring them up to the necessary thickness requirements. This involved only the county and private roadways. The state routes generally had thicknesses far in excess of those deemed necessary. Shoulder thicknesses were less than required, and the contractor was cautioned not to steer the vehicle onto shoulders.

Table 1. Equivalent pavement thickness requirements when bituminous concrete thickness exceeds 4 in.

Required Total Pavement Thickness	Required Equivalent Pavement Thickness (in) Bituminous Concrete	Crushed Stone
16 in—state roadways	4	12
	5	10
	6	8
	7	6
14 in—county, private, detours, and widenings and shoulders	4	10
	5	8
	6	6

Note: 1 in = 2.54 cm.

PROOFROLLING

In order to evaluate the analyses and recommendations from a practical standpoint and to identify potential localized problem areas, it was recommended to the contractor that the entire length of roadway be proofrolled with a simulation of at least one axle line. This was considered to be mandatory because of the uncertainties involved with the design analyses and the limited amount of data for the roadways involved. The entire route was proofrolled with a simulated axle line prior to any upgrading of roads. The visible deflection response of the pavements was visually observed and documented. The use of a single axle was justified because the analysis procedure indicated no significant interaction between the axles.

The deflections of the surface of the state routes generally were not observable with the unaided eye. The county roads consisted of only 2.5 cm (1 in) of asphalt and 7.5-10 cm (3-4 in) of crushed stone and often pumped and began to break up under just one pass of the one simulated axle. These observations were used to develop the final recommendations for the project.

FINAL THICKNESS RECOMMENDATIONS

The county roads were to be upgraded to 7.5 cm (3 in) of bituminous concrete and 25 cm (10 in) of stone. The reduction to 33 cm (13 in) of total pavement thickness reduced the theoretical number of passes of the transport assembly to approximately two. After the second move, 2.5 cm (1 in) of bituminous concrete were to be placed to smooth out any areas of county roads that might be slightly damaged by the two moves. The recommended thickness requirements for the state routes were left unchanged.

THICKNESS BY TENNESSEE DEPARTMENT OF TRANSPORTATION

As part of the permitting process, the Tennessee Department of Transportation (TDOT) performed its own testing at 15 locations. This testing consisted of the following:

1. Transverse elevation cross sections,
2. Benkelman beam deflections,
3. Determination of the in-place density of the asphalt pavement, and
4. Determination of the thicknesses of the asphalt and base course.

This testing was done prior to each move and items 1, 2, and 3 were repeated shortly after each transport move. TDOT also measured pavement surface temperatures during the move at several locations.

Benkelman Beam Deflections

An analysis of the TDOT reported data was performed to evaluate the effects of the move of the RPV. Of the 15 test locations selected by TDOT, all were

Figure 5. After-move Benkelman beam deflection versus before-move Benkelman beam deflections, second move.

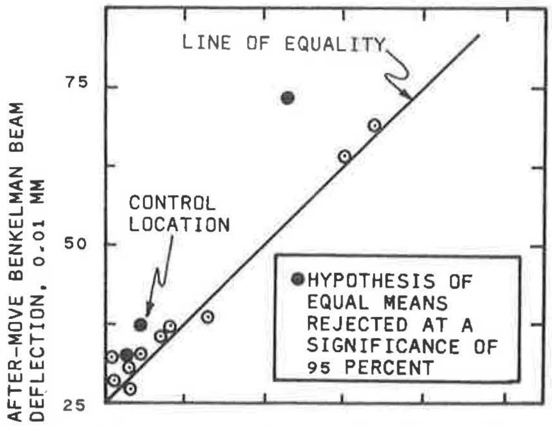


Table 2. Input parameters for ELSYM5 analysis.

	Case		
Parameter	1	2	3
Layer thickness (in)			
Bituminous concrete	4	4	3
Crushed stone	10	12	10
Subgrade	Infinite	Infinite	Infinite
Main trailer wheel loads			
Wheel load (lb·f)	7 890	7 890	7 890
Tire contact pressure (lb·f/in^2)	124	124	124
Tire contact radius (in)	4.50	4.50	4.50
Michigan tractor wheel loads			
Wheel load (lb·f)	24 500	24 500	24 500
Tire contact pressure (lb·f/in^2)	34	34	34
Tire contact radius (in)	15.14	15.14	15.14
Elastic modulus (lb·f/in^2)			
Bituminous concrete	100 000	100 000	100 000
Crushed stone	15 000	15 000	15 000
Subgrade	7 500	7 500	7 500
Poisson's ratio			
Bituminous concrete	0.35	0.35	0.35
Crushed stone	0.35	0.35	0.35
Subgrade	0.40	0.40	0.40

Note: 1 in = 2.54 cm; 1 lb·f = 4448 kN; 1 lb·f/in^2 = 2143 kPA.

traversed by the transport assembly except location 12. This location was used as a control section when it was learned that it would not be traversed. The surface deflection of a pavement is accepted as an indicator of the pavement's strength and integrity. Therefore, it was thought that if the transport move did any significant damage, then the effect would be to weaken the pavements, which would result in higher Benkelmen beam deflections.

A plot of the average after-2nd-move (A2M) Benkelman beam deflections versus the before-2nd-move (B2M) deflections at each of the 13 test locations for which temperature-corrected deflection data are available is shown in Figure 5. This plot shows that 11 of the 13 average A2M deflection readings were greater than the average B2M deflection readings. The average A2M deflection readings increased from only slightly to 0.18 mm (0.007 in) over the average B2M deflection readings. All but one of the locations exhibited increases in Benkelman beam deflections of less than 0.08 mm (0.003 in) and two locations actually exhibited a decrease in deflection. The average A2M deflection readings at location 12 (which was not traversed) increased by 0.05 mm (0.002 in) over the B2M deflection readings. The overall average increase was 0.03 mm (0.001 in) for all 13 locations.

In order to evaluate the B2M and A2M deflection readings, statistical procedures concerning hypotheses of equal means by using paired observations were used to analyze the data (7). The t-statistics for paired observations (individual B2M deflection paired with individual A2M deflection at each measurement location) were calculated for the 13 test sections. Of the 13 test locations analyzed, only 3 were found to have A2M average deflections that were statistically different from the B2M average deflections at a 99 percent confidence level. One of these 3 locations included the control section that was not traversed by the transport vehicle.

An analysis of the paired average B2M and average A2M deflections for all 13 test locations indicated that the hypothesis that the average A2M deflections were equal to the B2M deflections would not be rejected at a 99 percent level of significance. Thus, the second move did not appear to weaken the pavements at the test locations to the point where significantly greater deflections would be observed under the standard load.

Asphalt Density

With the exception of two locations, the average asphalt density generally increased from 16 to 32 kg/m^3 (1-2 lb/ft^3). The average B2M and A2M asphalt density data were analyzed on a paired basis similar to that for the deflection readings. The hypothesis of equal asphalt densities before and after the second move was rejected at a confidence level of 99 percent.

Transverse Cross Sections

The cross section data [surveyed to 3 mm (0.01 ft)] generally indicated changes in cross section elevation ranging up to 12 mm (0.04 ft). The changes in cross section readings indicated a general settlement rather than shear failure with resultant heave. The changes in cross section elevations are probably a result of consolidation in both the subgrade and the asphalt layers. Sufficient information is not available to allow an allocation of the settlement between consolidation of the asphalt and consolidation of the subgrade.

POST-MOVE ANALYSIS WITH LAYERED ELASTIC THEORY

As a follow-up evaluation after the completion of the second move, analyses were made of the recommended pavement thickness sections by using a layered elastic computer program. The program used was ELSYM5, and it was run by ARE, Inc. (8).

Selection of Input Parameters

An elastic modulus of 52 MPa (7500 lb·f/in^2) was chosen for the subgrade based on the widely accepted relation of subgrade modulus (lb·f/in^2) = 1500 x CBR and a design CBR of 5. For the crushed stone base course, an elastic modulus of 103 MPa (15 000 lb·f/in^2) was chosen. This value is slightly less than the 138-172 MPa (20 000-25 000 lb·f/in^2) that is recommended in some published correlations; however, it is considered reasonable (9,10). An elastic modulus of 690 MPa (100 000 lb·f/in^2) was used for the bituminous concrete. This value is based on a relation by Barker and others (9) and a pavement surface temperature in the range of 38°-49°C (100°-120°F). Similar values are reported by

Shook and Kallas (11). The TDOT reported that temperature of the pavement surface at selected locations during the moves ranged from 27° to 66°C (80° to 150°F); most of the values were in the 35°-49°C (95°-120°F) range. Due to the wide range in pavement sections and temperature conditions, the assumed elastic moduli are considered representative of an average condition during the moves.

Table 2 presents the three thickness cases analyzed and the input parameters for the main trailer calculations and the Michigan tractor calculations. Cases 1 and 2 represent the original thickness recommendations and case 3 represents the slightly reduced final thickness recommended for the upgrading of county roads.

Results of ELSYM5 Analysis

Figure 6 presents the typical results of computer program analysis in graphical form for a single-wheel load on the main trailer for case 1. The deflection of the pavement surface, the tensile strain at the bottom of the bituminous concrete (ε_t), and the vertical compressive strain on the top of the subgrade (ε_c) are presented. These two strains are generally recognized as the controlling strains in the thickness design of flexible pavements. To determine the maximum deflections and strains, the principle of superposition was used. The maximums for the main trailer were found to occur beneath the center point between either set of the center sets of dual wheels. The strains beneath one wheel of the Michigan tractor were basically unaffected by the other wheels on the tractor. Table 3 presents the maximum strains and deflections calculated for the combined wheel loads for the three cases analyzed.

As can be seen from Figure 6, the tensile strain at the bottom of the bituminous concrete (ε_t) and the vertical compressive strain on top of the subgrade (ε_c) dissipates to virtually nothing at a distance of 50-75 cm (20-30 in: 4.4 to 6.7 radii) from the center of the loaded areas for the transport trailer wheels. However, the deflection of the pavement surface is affected to a distance in excess of 254 cm (100 in; 22.2 radii) from the center of the loaded area. This indicates that the extent of effect of one wheel on the strains in the pavement is similar to that in Figure 3.

The maximum strains calculated and presented in Table 3 were compared with published correlations for strain and repetitions to failure. Figure 7 presents typical curves for ε_t versus number of strain applications to failure (12,13). Figure 8 presents typical curves for ε_c versus number of strain applications to failure (9,14). The theoretical number of repetitions for the combined transport train are presented in the table below for both the elastic analysis and the U.S. Army Corps of Engineers procedures.

	Case		
Combined Transport Train	1	2	3
No. of theoretical repetitions of transport train to failure based on ε_v (critical criteria)	7-15	10-28	3-5
No. of theoretical repetitions of transport train to failure based on Corps of Engineers' procedure	3	10	2

Figure 6. ELSYM5 strains and deflections, main trailer, case 1: H1 = 4 in and H2 = 10 in.

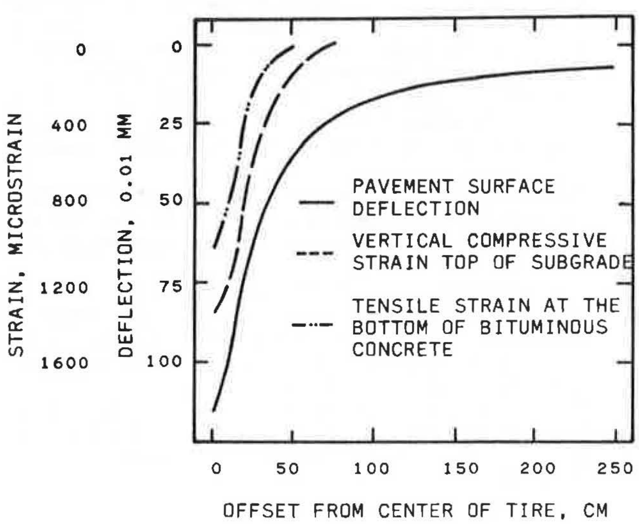


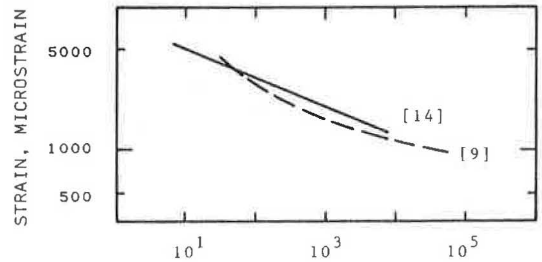
Table 3. Maximum strains and deflections for main trailer and Michigan tractor.

Condition	Case 1	2	3
Main trailer			
Maximum surface deflection (in)	0.170	0.169	0.178
Maximum tensile strain at bottom of bituminous concrete, microstrain	1400	1380	1560
Maximum vertical compressive strain on top of subgrade, microstrain	2280	2000	2880
Michigan tractor			
Maximum surface deflection (in)	0.112	0.110	0.117
Maximum tensile strain at bottom of bituminous concrete, microstrain	260	250	280
Maximum vertical compressive strain on top of subgrade, microstrain	2150	1930	2340

Note: 1 in = 2.54 cm.

Figure 7. Strain repetitions to failure versus tensile strain at bottom of bituminous concrete.

Figure 8. Strain repetitions to failure versus vertical compressive strain on top of subgrade.



There was also a reasonably good agreement between the CBR equation procedure and the strain procedure based on the elastic analysis, particularly when one considers that a slight error in the strain curves (log-log plot) can greatly affect the number of allowable strain repetitions.

SUMMARY

The movement of the RPVs can be considered a success in terms of the basic criteria established at the start of the project. The transport trailer did not become stuck nor did it experience any delays due to inadequate pavement sections on roadways I evaluated.

As for visible or measurable damage, the move of the RPVs did not appear to affect the roads significantly where TDOT performed tests. The increase in asphalt density could be construed to mean that the move actually improved the pavement structure since higher densities normally mean higher strength and stability for a given mix. The only detrimental effect of this increase in density would be if the density increased to the point where the surface was bleeding asphalt and reducing skid resistance. However, this bleeding has not been evident. The major visible damage done by the move occurred by inadvertent travel onto inadequate shoulder sections. Localized scarring of the pavement surface occurred due to slippage of the tires on the prime movers when they started from a dead stop.

An overlay patch on a road in an industrial park was peeled up during the first day of the first move. The area affected was approximately 46 m (150 ft) long. The overlay was only 2.5 cm (1 in) thick and the bond between the overlay and underlying layer was observed to be very poor due to water and dirt at the interface. There was no failure in the underlying asphalt layers. A patch of asphalt of apparently similar age several hundred meters down the road appeared to be unaffected by the move.

There was reasonably good agreement between the thickness requirements for the U.S. Army Corps of Engineers procedure (relatively simplistic) and the layered elastic theory (relatively sophisticated). However, the close agreement achieved in the subject case may have been fortuitous and may not occur in other cases. Also note that the CBR procedure and strain procedure were developed in different manners. It would appear that practicing engineers who do not have access to sophisticated computer programs can use the U.S. Army Corps of Engineers procedures (1) to evaluate heavy vehicle moves and the results can be expected to be reasonable.

This project also indicates that if the vehicle for heavy transports is properly designed in terms of numbers of wheels and axles then flexible pavements of moderate thickness can be used. Most high-traffic state and Interstate routes (flexible pavements) can probably handle a few repetitions of a properly designed transport vehicle without suffering significant damage.

ACKNOWLEDGMENT

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REFERENCES

1. Transporting Abnormally Heavy Loads on Pavements. Highway Research Circular 156, May 1974, 14 pp.
2. A Guide for the Design of Full-Depth Asphalt Pavements for Heavy Wheel Loads. Asphalt Institute, College Park, MD, MISC-75-5, Sept. 1975.
3. D.J. Van Vuren. The E.S.W.L. Concept and Its Application to Abnormally Heavy Vehicles on Roads. Die Siville Ingenieur in Suid-Afrika, Aug. 1969, pp. 191-203.
4. Full-Depth Asphalt Pavements for Air Carrier Airports. Asphalt Institute, College Park, MD, MS-11, Jan. 1973.
5. E.J. Yoder. Principles of Pavement Design. Wiley, New York, 1959.
6. E.J. Yoder and M.W. Witczak. Principles of Pavement Design, 2nd Ed. Wiley, New York, 1975.
7. W.J. Dixon and F.J. Massey, Jr. Introduction to Statistical Analysis. McGraw-Hill, New York, 1957.
8. H.J. Treybig, B.F. McCullough, F.N. Finn, R. McComb, and W.R. Hudson. Design of Asphalt Concrete Overlays Using Layer Theory. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Vol. 1, 1977.
9. W.R. Barker, W.N. Brabston, and Y.T. Chou. A General System for the Structural Design of Flexible Pavements. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Vol. 1, 1977.
10. H.F. Southgate, R.C. Deen, J.H. Havens, and W.B. Drake, Jr. Kentucky Research: A Flexible Pavement Design and Management System. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Vol. 1, 1977.
11. J.F. Shook and B.F. Kallas. Factors Influencing Dynamic Modulus of Asphalt Concrete. Proc., Assn. of Asphalt Paving Technologists, 1969.
12. P.C. Pretorius. Design Considerations for Pavements Containing Soil-Cement Bases. Univ. of California, Berkeley, Ph.D. dissertation, 1969.
13. C.L. Monismith, D.A. Kasianchuk, and J.A. Epps. Asphalt Mixture Behavior in Repeated Flexure--A Study of an In-Service Pavement Near Morro Bay, California. Univ. of California, Berkeley, IER Rept. TE 67-4, 1967.
14. Y.T. Chou, R.L. Hutchinson, and H.H. Ulery, Jr. Design Method for Flexible Airfield Pavements. TRB, Transportation Research Record 521, 1974, pp. 1-13.

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Discussion of Aggregate Properties for Untreated Road Surfaces

MARTIN C. EVERITT

Historically, engineers have assumed that an untreated aggregate road surface behaves in substantially the same way as a bituminous paving mixture. Test criteria and construction specifications have been derived by loosening normal bituminous pavement criteria. The paper argues that, although most or all of the properties of aggregate that are important for bituminous applications are also important in untreated surfaces, the relative degrees of importance are probably different. Some additional properties, such as natural cementation used in the design of early macadam pavements, should be revived and studied. A comprehensive review of aggregate properties has never been made to determine whether assumptions based on flexible pavement practices are valid for untreated aggregate surfaces. Some agencies and individuals have made piecemeal investigations. Practical experience suggests that, for most low-volume roads, present practices are only marginally adequate. Present specifications and test criteria are reviewed and commonly specified values are compared with experience, primarily in the Rocky Mountain Region of the U.S. Department of Agriculture Forest Service. Suggestions for revised criteria are offered. The utility of some of the present tests or criteria, as applied to untreated aggregate surfaces, is questioned. Attention is called to the sometimes complex interrelations among several properties.

Most of the road mileage in the world is surfaced with either native soil, naturally occurring sand and gravel, or crushed aggregate. Even in the United States, a major fraction of the road mileage is not hard surfaced. Yet there is almost no technology for the design of untreated aggregate (or gravel) surfaces. Before World War II, aggregate surfaces were often maintained into existence. A minimum thickness of aggregate was placed. When traffic produced mudholes or ruts, these were improved with additional aggregate until the problem was cured. Design was primarily derived from experience and judgment.

More recently, the need for some sort of design approach was recognized. Engineers seem to have begun with the assumption that an aggregate surface is essentially the same as a bituminous concrete surface, except for the color. Flexible pavement design techniques have been applied. Construction specifications are derived from flexible pavement base or surface course specifications, usually by loosening the quality requirements. All of the properties considered important for bituminous mixtures, such as abrasion and gradation, are assumed to be equally important for untreated aggregate surfaces. A systematic effort to determine whether this is actually true has never been made.

On the basis of long experience with the low-volume road system of the Rocky Mountain Region of the U.S. Forest Service, I have concluded that these assumptions are not completely valid. All of the properties usually considered are important; however, their relative degrees of importance are evidently different from those in bituminous surfaces. There is a property, herein called natural cementation, of which little is known at present, that appears to be very significant in the performance of untreated aggregate surfaces. Other important properties or characteristics may currently be little known or as yet undiscovered. In addition, many of the limit values frequently specified are too conservative; others may not be sufficiently restrictive.

This paper breaks no new ground. It is offered to stimulate discussion and, possibly, some new research.

FUNCTIONS OF ROAD SURFACE

A road surface, of any type, must perform several functions. Some of these are as follows:

1. Load distribution and load transfer to the lower layers and the roadbed; surface layer must be strong enough to prevent overstressing of the lower layers;
2. Stability under the three-dimensional loads applied; surface must resist raveling, shoving, rutting, and consolidation caused by vertical, longitudinal, and lateral forces applied by tires;
3. Reasonable smoothness without excessive maintenance;
4. Skid resistance; in aggregate surfaces, this requires both friction between the tires and the surface and the absence of loose gravel that might roll under the wheels;
5. Reasonable control of dust;
6. Maintainability by using normal technique; and
7. Surface drainage to sides or ditches.

A properly designed aggregate surface can fulfill all of these functions economically. Of course, different kinds or volumes of traffic require different designs or points of emphasis. Where high-quality aggregates are in limited supply, they should be reserved for the most heavily used roads and lower-quality material used for the lower-class roads.

PROPERTIES OF AGGREGATES

Standard specifications for aggregates are used by many agencies for some or all of the following reasons:

1. Material that conforms to the specification will usually perform satisfactorily in normal use situations, provided the proper specification is chosen for the project;
2. Sophisticated design or quality assurance testing is not usually feasible for various reasons;
3. Specifications are balanced between ease of production and the ideal; tolerances are broad enough to cover normal production variability and testing accuracy;
4. Items or tests specified are simple enough for easy control in the field; and
5. Materials must not be unusually difficult to handle, place, and compact by using normal construction practices.

Adherence to standard specifications is logical so long as high-quality materials are plentiful in the work area. However, many materials that would perform satisfactorily in service will be rejected or overlooked because they do not completely fit the standard specifications. Some roads will be severely overbuilt with respect to the real need.

Grading

Gradation is perhaps the most important single property of an aggregate because it directly or indirectly affects several other properties and behaviors. Many textbooks present the advantages and disadvantages of the open-graded, dense-graded, gap-graded, and over-sanded (excessive fines) cases (1).

Most modern specifications establish target values for each sieve size and allow a tolerance on each side of the target value. The target values must be selected in relation to the function of the aggregate and the material available in the proposed source. These relations are often overlooked when standard specifications are used arbitrarily. The tolerance should consider the degree of control that can be reasonably expected from a normally well-run production operation and on the skill and intensity of the quality assurance effort. However, the tolerance must also preserve the intent of the designer. For example, the U.S. Forest Service standard specifications (2), which are typical, allow tolerances of 4-15 percentage points on either side of the target value for surfacing aggregates. Many engineers consider a tolerance of 15 percentage points to be excessive.

Of course, individual producers vary in efficiency and one might suspect that those usually associated with low-volume road projects are less efficient than larger operators, on the average. If one tightens the tolerance, he or she should expect to pay for it, because all but the most efficient producers will need to change their operations to conform to the tighter limits. An accurate estimate of the cost per ton of aggregate for each percentage point by which the tolerances are tightened beyond the generally used criteria is difficult, especially when dealing with portable operations and on-site pits. Many engineers think that the improvement in performance derived from aggregates that are very close to the theoretically perfect gradation is worth a significant price increase. Reliable data on production costs relative to specification tolerances for field operations would be valuable.

Likewise, few or no reliable data exist that relate performance and maintenance costs to relatively more or less restrictive construction specifications. The logical assumption is that lower construction quality leads to higher maintenance costs and vice versa. Reasonably accurate data would be valuable.

Cementation

Some aggregates possess a property herein called natural cementation. Cementation was recognized before 1912 (3, pp. 1-3) when a test called the Page Cementing Value was employed in the design of macadam pavements. In a 1916 paper, Lord (3, pp. 1-3) stated that, "The cementing value of road materials is conditioned chiefly by the colloidal products of rock decay and increases in a general way proportionately with these products, reaching a maximum in rocks free of quartz."

After World War I, rubber tires and motorcars rapidly replaced steel tires and wagons, and engineers shifted their attention to the new bituminous and concrete pavements. Cementation apparently disappeared from the literature. Because of a continued lack of attention, the property is not now well understood. The extensive literature on aggregate degradation suggests that the entire subject is extremely complex.

Two cementation test procedures are currently available. Both are inexpensive and simple to perform. Neither is completely satisfactory. Aggregates sent to the Federal Highway Administration (FHWA) laboratory in Denver are tested by using a procedure that is loosely derived from AASHTO T106 (4). A representative portion of the sample passing the number 10 sieve is compacted in three 2-in cube molds at optimum moisture and AASHTO T99 maximum density. After drying to constant weight at 230°F, the three cubes are broken in a compression machine and the average of the strengths is taken as the cementation value, expressed in pounds per square inch.

This test is simple and uses a small sample. However, it tests only the material that passes the number 10 sieve, not the total aggregate. Thus, one must extrapolate from the cementation value to the behavior of the total aggregate. A cementation value of 200 $lb{\cdot}f/in^2$ has been found to be generally satisfactory, but lower values may prove to be acceptable, especially when the total aggregate is very well graded.

The Willamette National Forest uses a procedure based on AASHTO T99, D (5). Fresh portions of the total aggregate are compacted in three T99 cylinders at optimum moisture. After drying to constant weight, they are broken in a compression machine and the average strength taken as the cementation value. A strength of 75 $lb{\cdot}f/in^2$ is considered satisfactory. This test requires more material than the cube test but uses the total aggregate and so should be more representative. However, the FHWA laboratory in Denver has observed severe problems with sample segregation and very wide scattering of data in a trial use of the procedure in a production laboratory. Testing problems could indicate that the gradations used are faulty; however, the procedure should be more fully tested.

Limited attempts have been made to correlate the cementation value with other properties, including the Atterberg limits, sand equivalent, percentage passing the number 40 and 200 sieves, and the dust ratio. No strong correlation was found except that, in about 100 samples, all high cementation values occurred with sand equivalents less than 45. The presence of high cementation values in a number of aggregates that have low or unmeasurable liquid limit and plasticity index (PI) values is conspicuous. The reverse has also been occasionally noted. There may be a weak correlation with the dust ratio. The performance of the aggregate on the road has often been more accurately predicted by the cementation value than by PI.

The evidence and experience strongly suggest that natural cementation is an independent property that could be useful in the design of untreated aggregate surfaces. Research is needed to confirm this, determine the mechanism involved, develop a simple and more reliable test procedure, and establish desirable values based on the performance of road surfaces.

Binder

The term binder has different meanings in different localities. Generally, it can be defined as one or more properties, usually found in soil and aggregate particles finer than the number 200 sieve, that tend to hold the coarser particles in place. Natural cementation just described is probably only one of several such properties.

The need for a binder in surfacing aggregates has been recognized and many specifications define it in terms of a range of desired PI values. PI requirements of 2-4 on the low end of the scale or 9-10 on the upper end are common in the United States, though in other nations much higher upper limits are used. For example, British engineers working for the United Nations Educational, Scientific, and Cultural Organization (UNESCO) in Africa (6) recommend the following PIs for surfacing aggregates:

Climate	PI
Moist temperate and wet tropical	4-9
Seasonal wet and tropical	6-15
Arid	15-30

I think that PIs less than 6 are not significant and values as high as 12-15 are not harmful in most cases. The binding power defined by PI seems to be different from natural cementation, since no correlation between the two has been found.

Abrasion

In the early bituminous pavements, it was thought that abrasion was not important, but research in the last 20 years has shown that all aggregates degrade and the mechanism and products are extremely variable and complicated (3). The Los Angeles abrasion test, AASHTO T96, has evolved and is accepted as a reliable indicator of aggregate response to abrasion and impact, especially under construction handling, placement, and compaction equipment. It does not adequately predict the later chemical degradation and production of plastic fines that occur in some aggregates in the presence of water. Many agencies require a Los Angeles abrasion loss of 40 percent or less for surfacing aggregate. In many areas, this quality is not hard to achieve.

Four major faults with aggregates that have higher abrasion losses are as follows:

1. Shorter service life due to physical degradation; this can be compensated by greater thicknesses or more frequent replacement;
2. Excessively smooth surfaces, sometimes leading to slippery conditions in wet weather; this is also noticed in high PI aggregates; it does not seem serious so long as vehicle tires have a fairly good tread;
3. Excessive dust, which can be controlled with various palliatives; and
4. Surface erosion during rains, which results from the disappearance of coarse aggregate; this can be a local maintenance problem.

In general, the 40 percent maximum for Los Angeles abrasion loss seems valid for arterials and heavily used collector roads. For local roads and collector roads that have small traffic volumes, an acceptable loss of 50 percent is more reasonable. Aggregates that have losses in excess of 50 percent have performed satisfactorily in some cases. When use of such aggregates is planned, the gradation should be made coarser than usual to compensate for degradation of the coarse fragments. Crushing on the road by grid roller is economical and frequently adequate.

In the early years of road building, when dry macadam pavements were common, a certain amount of aggregate abrasion was considered essential to the performance of the pavement. Continued production of fines replaced those lost to dust and rainwash and so maintained the desired dense gradation at the surface. This reasoning, and the frequently observed poor performance of very hard aggregates, suggest that minimum, as well as maximum, Los Angeles abrasion losses should be specified for untreated aggregate surface materials.

Abrasion is influenced by the grading and binding capability of the aggregate. Both open-graded and over-sanded aggregates will experience greater abrasion losses than will the same aggregate in the dense-graded case because the particles in a dense gradation are more tightly bound than those in either of the other cases, and so are less subject to wear.

Durability

Early in the Interstate program, engineers became aware that many pavements were failing long before their anticipated life span, even though they had been well designed and built by using aggregates that met all of the quality standards then in use. An enormous amount of research has shown that all aggregates degrade differently, and some may produce different residues or a greater volume of residue in the presence of water than they do in the dry state. Rocks that produce small amounts of nonplastic fines in the Los Angeles abrasion test may produce a higher volume or highly plastic fines if water is present. These fines are a major cause of bituminous pavement failures, but these same fines usually will improve the performance of an untreated aggregate surface. Wet abrasion tests are still undergoing development, but AASHTO T210 seems to be gaining general acceptance. More work is necessary to establish limit values for acceptable performance, but it seems likely that a maximum durability index, together with a range of abrasion losses, should be specified for untreated surfaces. The limit values remain to be determined.

Dust Ratio

The dust ratio is defined as the ratio of the percentage passing the number 200 sieve to the percentage passing the number 30 sieve. It is a means of controlling the shape of the grading curve in the fine end, since there are often no sieves specified between the number 30 and the number 200. It is possible for an aggregate to be comparatively high in material passing the number 30 and low in the percentage passing the number 200; in other words, gap graded in the area where good grading is important to stability. The usually specified dust ratio of two-thirds helps ensure that the fine fraction of the aggregate is well graded. However, it is often considered to be an unimportant specification and is very frequently ignored in quality assurance testing. The dust ratio should be given more attention.

Sand Equivalent

The sand equivalent (SE) test, AASHTO T176, began as an attempt to develop a shortcut for the Atterberg limit tests in aggregate. It is quick, easy, and reasonably accurate. Though not an exact substitute for the Atterberg limits, an SE value greater than 35 usually correlates with a PI of less than 6. When this is the only item of concern, as in an aggregate for bituminous paving mixture, it is a satisfactory substitution.

However, it is not possible to develop a broad correlation between definite values of PI and SE. Thus, when a minimum PI is specified, a sand equivalent cannot be substituted. The test is not of much use at this time in specifying aggregates for untreated surfaces except that the technique is a part of AASHTO T210. Some evidence exists, though not conclusive as yet, that good values of natural cementation, based on the FHWA test, occur with sand equivalents less than 45 (4).

The sand equivalent also correlates with the percentage passing the number 200 sieve.

Compaction

Engineers working under nonengineering managers are often asked to defend the practice of thorough compaction of untreated aggregate surfaces during construction. We confidently reply that it is well worth the cost because compaction preserves the distribution of fines, prevents segregation, and improves the structural support of the layer--all arguments firmly based on bituminous pavement experiences.

Yet, most aggregate surfaces will fluff or loosen with frost cycles or wet seasons, then settle down again under traffic; this behavior continues throughout the life of the surface. I believe that compaction is beneficial but find it hard to convince skeptics in the absence of positive data. Indeed, many low-volume logging roads are built with only traffic compaction of the surfacing aggregate and seem to perform as well as others built with controlled compaction. If the performance of untreated aggregate surfaces built with and without controlled compaction could be compared with type and volume of traffic, the resulting data would be very useful to engineers and managers.

CONCLUSION

The foregoing shows that the properties of aggregates for untreated road surfaces are complex. Specifications must be influenced not only by the characteristics of the rock but also by the climate, the purpose of the road, and nature and volume of traffic. Many complex interactions occur that are not predictable on the basis of laboratory test procedures. Even the characteristics defined by tests are not independent of each other. Yet, there is a strong tendency for engineers who are not well trained in materials to look at each property in a list of specifications as an abstract value and discard or modify those that do not seem satisfactory without consideration of the effects of that property on the overall behavior of the aggregate. This discussion presents no firm recommendations for numeric values because I do not have access to research facilities and personnel. The values cited are based on observation and experience in a particular environment. The objective of this discussion has been to call attention to some of the considerations involved and to stimulate some systematic investigations directed specifically at untreated aggregate road surfaces.

Safety, maintenance cost, user cost, economy, and riding quality all are affected by the aggregate used. Current research, particularly the huge Brazil project (7, pp. 304-340), is showing that fuel consumption and user costs in general are more strongly related to road surface conditions, especially smoothness, than has been recognized up to this time even with untreated surfaces. Long experience has proven that small investments in investigation, design, and quality assurance pay big dividends. However, many road-building agencies are caught in a personnel and budget squeeze that makes such work difficult or impossible. With renewed interest in low-volume roads, rapidly inflating costs, decreasing availability of quality aggregates, and tight environmental controls, it is no longer reasonable to ignore the design factors involved in untreated aggregate surfaces. We must understand the properties of aggregates and derive highly efficient specifications for even very low standard roads.

REFERENCES

1. E.J. Yoder and M.W. Witczak. Principles of Pavement Design. 2nd ed. Wiley, New York, Chapter 11, 1975.
2. Standard Specifications for Construction of Roads and Bridges. U.S. Forest Service, Section 703, 1979.
3. Degradation of Aggregates--1911-1968, Annotated. HRB, Bibliography No. 51, 1970.
4. M.D. Everitt. In Proc. of Forest Service Geotechnical Workshop, Internal Rept., Aug. 14-17, 1979.
5. W. Vischer. In Proc. of Forest Service Geotechnical Workshop, Internal Rept., Aug. 14-17, 1979.
6. Road Gravels. TRB, Compendium 7, 1979, 195 pp.
7. Low Volume Roads: Second International Conference. TRB, Transportation Research Record 702, 1979.

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Fabric-Reinforced Aggregate Roads—Overview

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The purpose of this paper is to present an overview of the use of fabrics in aggregate-surfaced roads. Specific areas addressed in the paper are (a) general performance characteristics of aggregate-fabric-soil (AFS) systems, (b) mechanisms that contribute to the fabric-related benefits, (c) various factors that exert a major influence on the performance of AFS systems, and (d) methods of analyzing and designing AFS systems. Data and information sources used in the discussion include pertinent literature and results from a study being conducted in the School of Civil Engineering at the Georgia Institute of Technology. Based on the discussion presented in this paper, use of an interlayer of fabric in an aggregate-surfaced road can lead to either better performance or to substantial reductions in aggregate layer thickness. It is also shown that the behavior of AFS is complex and difficult to analyze with theoretical models. Although numerous thickness design methods are available, most are for specific commercial fabrics and are empirical in nature. No general design procedure is available that can accommodate a variety of fabrics of widely differing properties and, thus, it is difficult for potential fabric users to make economic decisions in selecting fabrics and design thicknesses for various job requirements.

Synthetic engineering fabrics or geotextiles have become increasingly important in civil engineering applications in recent years. The main applications include drainage, erosion control, separation, and reinforcement. Fabrics that perform these functions are termed geotextiles and are defined by the American Society for Testing and Materials (ASTM) as any permeable textile used with geotechnical materials as an integral part of a man-made project, structure, or system.

In railroad and highway support systems, fabrics are used to provide separation between subgrade soil and ballast, subballast, base, or subbase layers and to provide tensile reinforcement to the system. The pumping action of traffic loading combined with high levels of saturation would allow an intermixing of these dissimilar materials were it not for the interlayer or fabric. The fabric provides a reinforcement to the support system by giving tensile resistance and confinement to granular materials. In addition, when large deformations occur, a membrane effect will provide improved load support capabilities.

Fabrics are also being used as an interlayer between cracked, deteriorated pavement surfaces and new asphaltic concrete overlays in order to reduce the rate of reflective crack occurrence.

One of the most common uses of geotextiles is in road construction and area stabilization, where soft, low-strength soil conditions prevail. In this application the geotextile is generally used in conjunction with a locally available aggregate such as crushed stone, shotrock, gravel, or sea shells to develop a structural support layer. For example, roads surfaced only with aggregate are continually being built to provide access to and around construction sites, logging operations, mining and quarrying operations, and as planned stage construction for higher-type roads. Experience with these types of support systems has shown that geotextiles can be cost effective and may allow substantial reductions in the quantity and possibly even the quality of aggregate used.

The purpose of this paper is to provide an overview of the use of fabrics in aggregate-surfaced roads. Specific areas to be addressed are (a) general performance characteristics of aggregate-fabric-soil (AFS) systems, (b) general mechanisms that contribute to the fabric-related benefits, (c) various factors that exert a major influence on the performance of AFS systems, and (d) methods of analyzing and designing AFS systems.

GEOTEXTILES

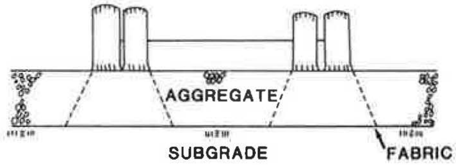
A large selection of fabric products is available commercially. These synthetic fabrics are commonly categorized based on construction and fiber composition. Basically, the two categories of construction are woven and nonwoven; however, the fiber composition may be polypropylene, polyester, nylon, or polyethylene. Polypropylene and polyester are the most common.

Cricital and optimum properties and characteristics of fabric for use in roadways have not been firmly established. Bell and others (1) suggest that tensile strength, modulus, friction-adhesion, creep, bond strength, fatigue, failure elongation, and burst strength are important mechanical properties. Lavin and others (2), Robnett and others (3), and Lai and Robnett (4) have shown the importance of fabric modulus on the performance of AFS systems. Giroud and Noiray (5) in their design method show an effect of both modulus and percentage elongation at failure; Bell and others (1) also suggest that chemical stability, durability, hydraulic conductivity, and constructability considerations are important. Space limitations do not allow extensive discussion of fabrics but Bell and others, Koernes and Welsh, and Rankilor (1,6,7) are excellent sources of information relative to fabric composition and manufacturing processes, fabric properties, test methods, and end-use requirements.

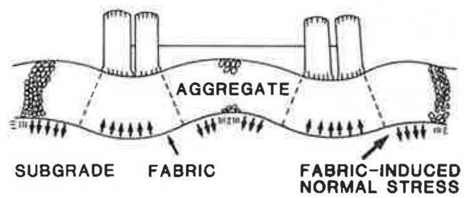
BENEFIT MECHANISMS

Fabrics are used in road construction with a locally available aggregate such as crushed stone, quarry or shotrock, sand, gravel, or sea shells to develop a structural layer. Figure 1a depicts the general geometry of such a system. In this application, the fabric provides reinforcement and separation benefits to the system (1,5,8).

Figure 1. Schematic of aggregate-fabric-soil subgrade system.



a) INITIAL STATE AFTER CONSTRUCTION



b) STATE AFTER SUBSTANTIAL HEAVY TRAFFIC

Reinforcement Function

In the reinforcement function, it is postulated that the fabric serves to improve the performance (often measured by resistance to permanent deformation or rutting) of the AFS system under repetitive vehicular loading due to a number of mechanisms including (a) restraint effect of the fabric on the aggregate and subgrade layer, (b) membrane effect, (c) friction developed at the fabric interfaces that creates a boundary layer effect, and (d) local reinforcement effect.

Restraint Effect

Two types of restraint effects should occur in the AFS systems. The first is related to the reverse curvature of the fabric outside the wheel path and the resultant downward pressure or apparent surcharge applied to the soil (Figure 1b). Such an effect increases the bearing capacity or resistance to shear flow of the soil from the wheel path. A second type of restraint effect occurs when the aggregate particles at the soil-aggregate interface tend to move from under the loaded area but are restrained or given a tensile reinforcement due to the presence of the fabric (9). The strength and modulus of aggregate material are beneficially affected by this increased confinement. The increased aggregate modulus decreases the compressive stress on the soil under the wheel load.

Membrane Effect

As the roadway undergoes large deformation (Figure 1b), the fabric is stretched and develops in-plane tensile stress, the magnitude of which depends on fabric strain and fabric modulus. A stress perpendicular to the plane of the fabric will be induced, the magnitude of which at any point equals the in-plane stress divided by the radius of curvature of the fabric at that point. The net effect is a change in the magnitude of stress imposed on the

Figure 2. Schematic diagram of 8-ft pit test apparatus.

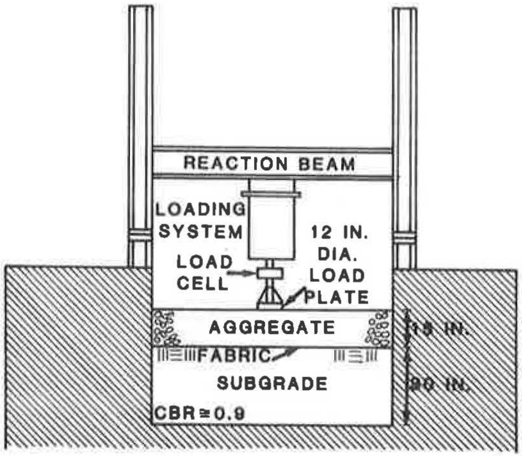


Table 1. Stresses measured by pressure cells in 8-ft pit tests.

Pressure Cell No.	Depth from Surface (in)	Offset from ℄ of Load (in)	Stress Normal to Pressure Cell (psi)	
			With Typar 3401	Without Fabric
9	16	0	12.0	15.5
3	20	0	11.0	15.5
4	30	0	-	-
7	42	0	5.5	6.5
8	20	6	8.2	13.0
6	20	12	7.0	7.5
5	20	18	1.6	0.8
2[a]	16	18	3.4	5.0
1[a]	32	30	3.4	3.4

[a]Radial stress; all other stresses are vertical.

subgrade (a reduction under the wheel load and an increase outside of the wheel path). Kinney (9) calculated a reduction of 18 and 37 percent for the effective load transmitted to the subgrade for two fabrics tested.

Lai and Robnett (4) report a change in measured compressive stresses (measured with special pressure cells by using diaphram wire resistance strain gage) under simulated repetitive wheel loading when fabric is included in the aggregate-soil system. Figure 2 shows the test equipment; the cell positions and outputs of the sensitive pressure cells are summarized in Table 1. Pressure cell readings were obtained for surface rutting of about 3-4 in under repetitive loading. Results from pressure cells 9, 3, 7, and 8 show a reduction in the vertical compressive stress directly under the loaded area for the system that contains a fabric compared with an aggregate-soil (AS) system without a fabric. An increased stress was measured for cell 5 for the system with fabric. This cell was located at an offset position approximately where maximum subgrade heave occurred. The membrane curvature at this location would be expected to increase the subgrade stress.

The reduction of the subgrade stress for the system that contains a fabric appears to be due to the membrane effect, although it could also be partly due to an increased load-spreading capability of the confined aggregate. As a result of the reduced subgrade stress, a reduction in the rate of rut formation in the subgrade for a given vehicular loading condition should be expected.

In order to develop fabric-induced stress, substantial vertical deformations, proper geometry, and fabric anchorage are generally required. Barvashov (10) and Raad (11) have suggested prestressing the fabric in order to reduce the system deformation required to get the fabric in substantial tension.

Friction and Boundary Layer Effect

Friction developed along the interface between aggregate-fabric and friction-adhesion of the fabric-soil interface create a boundary-layer or composite material of aggregate and soil immediately adjacent to the fabric. The composite material created due to the presence of the fabric should possess more favorable properties of ductility and tensile strength. The effectiveness of this phenomenon is closely related to the magnitude of friction-adhesion developed at the interfaces. Fabrics capable of developing high friction-adhesion appear to be desirable.

Local Reinforcement

Concentrated stresses due to imposed vehicular loading can cause a punching or local bearing capacity failure at the points of contact between the aggregate and subgrade. Use of fabric between the aggregate and soft soil will serve to distribute the load, reduce localized stresses, and, in general, provide increased resistance to vertical displacement. Bell and others (12) suggested this as a possible mechanism in the stabilization of a road constructed over muskeg.

Separation Function

In the separation function, the fabric serves to prevent the fine-grained subgrade soil from intermixing with the coarse-grained aggregate material and reducing its shear strength and stability. Depending on aggregate gradation, 10-20 percent additional plastic fines can cause a substantial reduction in shear resistance (13,14). From the design standpoint, the aggregate within the intermixed layer is ineffective.

These various mechanisms explain, at least in a qualitative sense, the improved performance or rutting resistance of aggregate layers reinforced with fabric. The contribution of each of the aforementioned mechanisms is difficult to quantify because of the extreme complexity of the AFS system. Kinney (9) has shown through his scale-model studies the confining effect of fabric, and Lai and Robnett (4) have reported on the change in stress state at the subgrade that appears to be due primarily to the membrane effect. Thompson (15), with theoretical studies, has shown the effect of confinement on the moduli of the aggregate and the resultant structural behavior of the AFS system.

The degree of benefit offered to the AFS system for its service life by a fabric depends to a large extent on the mechanical and durability (chemical stability) properties of the particular fabric used. Other factors such as subgrade strength, loading environment, and aggregate properties also have an important influence on the behavior and performance (rutting resistance) of the AFS system.

It should also be acknowledged that the relative contribution of the various mechanisms most likely will be different for railroad ballast-subballast systems or permanent, surfaced highway pavement structures than for the high deformation access or haul road type application.

AFS SYSTEM PERFORMANCE

General

Numerous laboratory and field studies are reported in the literature (2,3,8-10,12,16-24) that serve to illustrate the general behavior of fabric-reinforced aggregate layers over soft subgrade soil subjected to repeated surface loading. At the recent International Conference on Geotechnics (25), at least five papers were presented that showed positive structural benefits from the use of fabrics in conjunction with aggregate support layers over soft soil. Normally, the performance of these AS and AFS systems is measured in terms of surface rutting or rut depth. For specific job applications, the tolerable rut depth must be established. For highway vehicles, rut depths as great as 6-8 in might be tolerable, but for off-highway vehicles, even more rut depth might be acceptable. Obviously, if the vehicle becomes immobilized because of contact of the undercarriage with the rutted pavement surface, this is undesirable. Similarly, the deformation that occurs under each wheel load will influence the power required to move the vehicle forward. Hammitt (26) has suggested that an elastic deformation of 1.5 in or less is acceptable in terms of the tractive resistance and commensurate power requirements of the vehicle. Hammitt (26) and others (5,8) also suggest that a 3-in rut depth be used as a design criteria for these unsurfaced roads. Obviously, for higher-type pavements that have asphaltic concrete surfaces, deformations of these magnitudes could not be tolerated.

Figure 3 (24) illustrates the performance of a full-scale road that contains test sections of aggregate (control) and fabric-reinforced aggregate over a soft subgrade [California bearing ratio (CBR) ≈ 1]. Kinney and Barenberg (19) have reported the results of a laboratory repetitive loading study that used small-scale two-dimensional testing apparatus wherein a low modulus and a high modulus fabric were used in conjunction with crushed stone over a soft soil layer. Results of a large-scale pit test wherein AS and AFS (nonwoven fabric) were tested under repetitive loading (Figure 2) have been presented by Lai and Robnett (4). Typical results are shown in Figure 4 (4).

Thus, it is obvious that fabric-reinforced aggregate layers can provide superior performance compared with a similar system that does not contain fabric. The performance of such systems is, however, influenced by a number of factors.

Factors of Influence

The following factors seem to have major influence on the performance of AFS and AS systems.

Soil Properties

Fabric is most often used in conjunction with aggregate roads where soil conditions are poor. AFS systems have been built on soil conditions where the CBR is as low as 0.5 or less. Obviously, for these very low strength conditions, repeated vehicular loadings cannot be placed on the soil without excessive rut development and vehicle mobility problems; rather, a pavement structure of some sort such as aggregate or aggregate-fabric must be placed between the soft soil and the applied wheel loads.

The typical approach used by many (5,8,14) to establish the tolerable level of imposed subgrade stress has as a basis the theory of plasticity and classical work done by Rodin (27) and Whitman and Hoeg (28). Basically, this approach stipulates that

Figure 3. Rut depth as a function of vehicle passes.

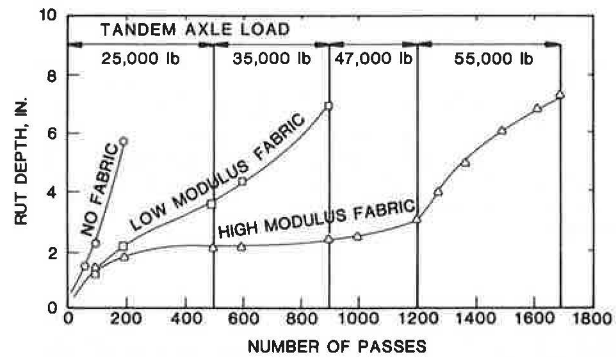
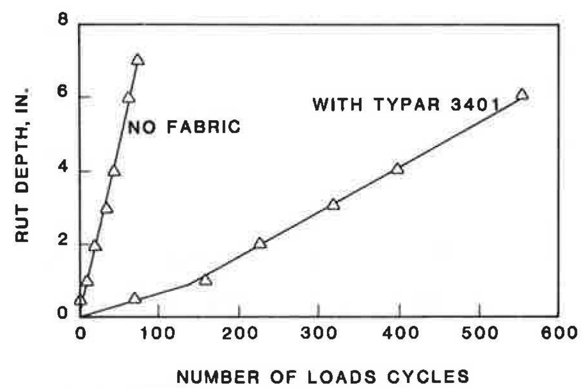


Figure 4. Rut depth versus number of load application plots for 8-ft test pit results.



the onset of plastic deformation occurs in frictionless soils ($\phi = 0$) under loading when

$$q_e = \pi c \quad (1)$$

where q_e is the elastic bearing capacity of soil and c is the undrained shear strength of the frictionless soil.

Complete bearing failure occurs when

$$q_{ult} = (\pi + 2) c \quad (2)$$

where q_{ult} is the ultimate or general bearing capacity of the soil.

Skempton (29) modified this equation to take into account geometric loading and depth influence factors and found for a square footing,

$$q_{ult} = (6.17)c.$$

Rodin (27), in his paper dealing with clay fills, states that the ultimate bearing capacity for a static circular or square footing resting on the surface of the clay is

$$q_{ult} = (6.2)c.$$

Thus, to develop rutting in the subgrade, the stress under repeated loading should fall somewhere in the range of πc to 6.2c. For a given amount of rutting, it is reasonable to assume that, as the number of applied loads increases, the imposed subgrade stress must decrease in this range and tend toward the πc value.

A major assumption associated with the foregoing discussion is that a unique and constant relation exists between shear strength and the rutting or permanent deformation a soil undergoes during repetitive application of stress. The rut development in the subgrade soil is influenced not only by the inherent nature of the soil but also by the duration or rate of loading. The soil can be considered as a viscoelastic material, which means that its permanent deformation is not only controlled by the magnitude of applied stress but also by the duration of stress application. As the duration increases, the amount of permanent deformation per load or rut rate should be expected to increase.

Figure 5. Effect of fabric modulus on initial rate of rut formation of AFS systems: 3-ft pit results.

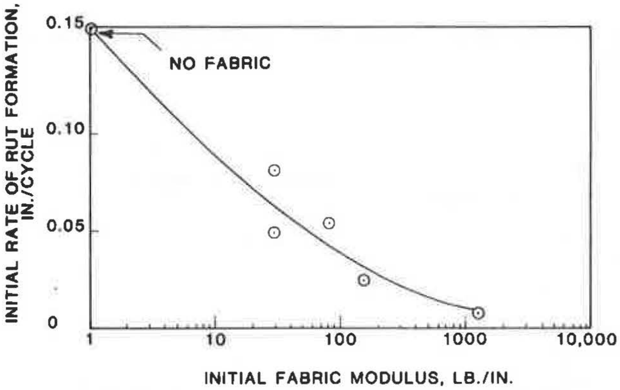


Figure 6. Effect of fabric modulus on number of load applications to cause 2-in or 4-in rut in AFS system: 3-ft pit results.

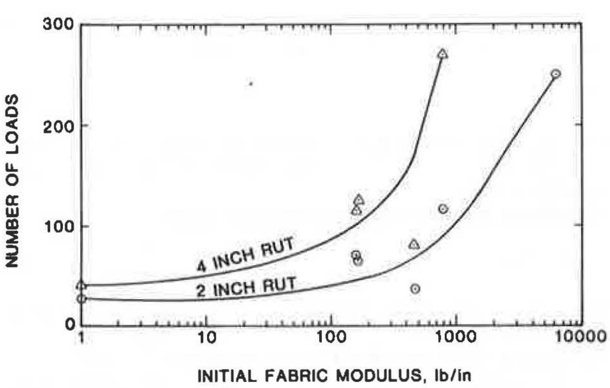
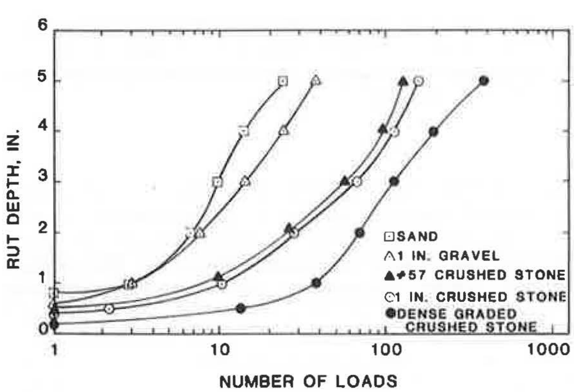


Figure 7. Effect of aggregate type on rutting of AFS systems: 3-ft pit results.



Fabric Properties

Critical and optimum properties and characteristics of fabric for use in roadways have not been firmly established. Bell and others (1) suggest that tensile strength, modulus, friction-adhesion, creep, bond strength, fatigue, failure elongation, and burst strength are important mechanical properties. Lavin and others (2), Robnett and others (3), and Lai and Robnett (4) have shown the importance of fabric modulus on the performance (rutting for a given number of loads and rutting rate) of AFS systems. Typical results from Lai and Robnett (4) are shown in Figures 5 (4) and 6 (4). Giroud and Noiray (5) recognize the significance of fabric modulus and failure elongation in development of their thickness design approach. In their approach, for a given geometry, the effect due to membrane support directly relates to the fabric stress and hence the fabric modulus. Barenberg (30) shows the reduction in design thicknesses of aggregate layer for AFS systems that contain a high modulus woven fabric compared with the systems that contain a lower modulus nonwoven fabric.

We are not aware of any data published wherein other fabric properties are related to potential performance.

Aggregate

A fairly good understanding currently exists as to the factors that influence the repeated load behavior of various aggregates in typical flexible pavement applications. Extensive research has been conducted in the areas of elastic or resilient behavior, the permanent deformation characteristics, and shear strength of various aggregates (15,31-37). However, none of these various studies has specifically addressed the behavior of various types of aggregate in the system wherein fabric is used as a reinforcement.

In a study being conducted in the School of Civil Engineering at the Georgia Institute of Technology, the effect of various types of aggregate on the performance of AFS systems has been studied. The details of one of the testing methods being used have been reported elsewhere (38). The following is a brief summary of the test program:

1. A 3-ft diameter test pit (similar to Figure 2, except smaller) with 15-in thick, soft subgrade, and aggregate layer thicknesses that range from 5 to 13 in;
2. Subgrade soil prepared to vane shear ≈ 4 psi and CBR ≈ 0.9;
3. Fabric placed between soil and aggregate;
4. Repeated loading applied on 6-in diameter plate, plate contact pressure = 70 psi, repetition rate = 20/min, and pulse duration = 0.2 s; and
5. During loading, vertical movement of loading plate is monitored.

Five different types of aggregate (sand, rounded gravel, and uniform and dense-graded crushed granite) were tested. Figure 7 depicts typical performance of the various AFS systems. Note from these data that a substantially different response of AFS systems might be expected with different aggregates. Differences in response can also occur as a result of gradation and density changes.

Loading Conditions

The loading conditions that can affect the performance of AFS systems include magnitude of loading, contact pressure, wheel configuration, duration or rate of loading, number of coverages, and degree of channelization of wheel loadings. The effects of duration of loading on the performance of AFS systems have been discussed previously.

The magnitude of the vertical pressure exerted on top of the subgrade depends on the magnitude of surface loading, tire pressure and contact area, and the thickness and load-spreading ability of the aggregate. Because the crushed stone has virtually no tensile strength, load spreading is controlled primarily by shear resistance and the ϕ of the aggregate (39), which in turn can be beneficially influenced by the reinforcement or confinement provided by a fabric.

ANALYSIS TECHNIQUES FOR AFS SYSTEMS

The capability for analysis of stresses, strains, and deflections in typical AFS systems is needed if a more rational approach to the understanding of the response characteristics and factors of influence for AFS systems is made. For many years, structural analysis of flexible pavements was accomplished by use of the Boussinesq and Burmister elastic theories. These approaches have definite shortcomings due to the inability to represent material response properly. In recent years considerable effort has been expended in developing refined theoretical models for flexible pavement analysis [e.g., viscoelastic methods (VESYS), a shear layer method that allows the aggregate to resist shear but not flexural stresses (39)] and various finite element techniques that can accommodate the typical nonlinear stress-strain response of subgrade soil, aggregate, and crushed stone. Extrapolation of even these refined techniques to analysis of AFS systems provides crude approximations, at best.

The AFS system has interesting and complex features such as the following:

1. Normally undergoes large plastic deformations,
2. Has a thin fabric component with high tensile modulus but very low flexural modulus,
3. Can exhibit slip at the aggregate-fabric or the fabric-soil interface,
4. Can exhibit membrane action, and
5. Can, because of the fabric, have an abnormal confining pressure on the aggregate as deformation progresses and causes the aggregate to become stiffer.

Because of these features, most conventional finite-element techniques, although capable of incorporating nonlinear stress-strain response, cannot accurately model the true AFS system. Raad (11), for example, has attempted to model the AFS system by prestressing a row of elements to enhance the membrane effect and assigning a Mohr-Coulomb approximation for the slippage characteristics of the fabric-soil and fabric-aggregate interfaces. Raad and Figueroa (40) have reported on a comprehensive analysis method for AS systems without fabric. With these approaches the resilient response and the stress and strain state within a low deformation AFS or AS system can be calculated.

Giroud and Noiray (5), Barenberg (30), and Bender and Barenberg (8) use the concept of the Boussinesq theory to estimate the initial state of stresses in an AFS system; they then correct (reduce) the vertical subgrade stress for the fabric-induced stress as rutting occurs. Kinney's fabric tension model (9) is an approach developed to calculate the magnitude of contribution of the fabric tension to the wheel-load-induced stresses in the subgrade of the AFS system.

None of these techniques allows for explicit calculation of the permanent deformation that develops under repetitive traffic loading; rather, typically an algorithm or model such as the one developed by Duncan and Chang (41) and later modified by Barksdale (42) could be used to calculate plastic strains in the system. Giroud and Noiray (5) took data developed by Webster and Watkins (23) and Webster and Alford (24) and empirically derived an expression that relates aggregate thickness, number of loads, load magnitude, soil strength, and rutting for an AS system that contains no fabric.

Thus, based on current literature, no theoretical technique appears to be available for precise analysis of the response of an AFS system subjected to repetitive loading.

It would be desirable for an analytical technique to be developed for the AFS system that would

1. Handle large displacements,
2. Handle cyclic or repetitive loading,
3. Incorporate nonlinear material behavior including failure,
4. Incorporate a flexible fabric membrane that has tensile modulus but no bending resistance,
5. Allow for slippage on both interfaces of the fabric, and
6. Determine accumulated permanent deformation.

DESIGN OF AFS SYSTEMS

Methods for calculating with a reasonable degree of confidence the aggregate thickness requirements for AFS systems are needed in order to efficiently and economically use fabrics. Such design methods can have two distinctly different approaches. One basic approach might be called a theoretical-mechanistic approach and the other would be empirical in nature. The theoretical-mechanistic approach is one wherein theories for calculating the state-of-stress imposed on the system are coupled with the mechanistic response of the system components. In this approach, it is essential that the theory used be capable of accommodating a complete definition of the mechanistic response of the AFS system to imposed loading for the large deformations involved. The complexity of the AFS system, as suggested by previous discussion, makes it difficult to model the system perfectly and thus calculate the load-deformation response of the system.

Even though a theoretical-mechanistic approach has the distinct advantage of being capable of considering a broad range in component material properties (especially fabric properties), such an approach does not appear to be at a state where it can be incorporated in a thickness-design method. Such an approach, which incorporates certain simplifying assumptions, could be used, however, for parameter studies and sensitivity analyses of the AFS system.

Based on the foregoing, some combination of available theory, fundamental behavior of soil, aggregate, and fabric, experimental and observational data, field experience, and engineering judgment probably offers the most reasonable basis for a thickness-design method. In fact, a combination of these forms the basis for most current design methods.

We are aware of five design methods (43-47) for specific commercial fabric product lines and two more general design procedures, one by the U.S. Forest Service (14), which has had some limited field verification, and one by Giroud and Noiray (5), which is based on simplified theory and limited experimental results published by others. In concept, the procedure by Giroud and Noiray (5) can potentially be used for a broad range of fabrics in that the procedure does require design input concerning fabric modulus and percentage failure elongation; however, at the time it was presented the procedure did not mention any field validation.

The U.S. Forest Service method (14), although implying broad applicability, probably is limited to use with certain nonwoven fabrics because it is based on work by Bender and Barenberg (8).

The DuPont method (43) is based on thickness design concepts developed by the U.S. Army Corps of Engineers (48) and actual full-scale field tests and other performance observations. The Celanese methods (44,45) are based on small-scale laboratory testing (8,9), fundamental material behavior (Rodin's work) (27), Boussinesq theory of stress distribution, and Kinney's fabric tension model (9) for the procedure described elsewhere (45). The Bidim method (46) has as its basis large-scale experiments that use a constructed bentonite subgrade and actual full-scale traffic loadings. The basis of the Imperial Chemical Industries (ICI) method (47) is unknown to us.

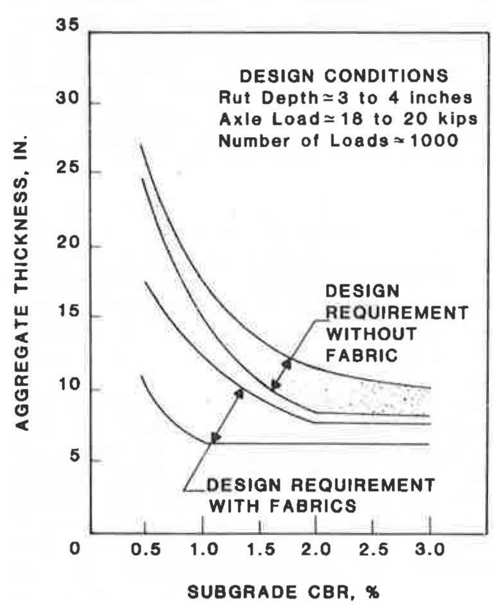
Note that most of these methods do not allow for variations in performance or rut depth. The failure criteria assumed are often unclear. Also, only the DuPont, ICI, Giroud and Noiray, and, to a limited extent, U.S. Forest Service methods allow for traffic density as a design input.

Space limitations do not allow for additional critique and discussion of these design procedures. None are ideal since none appear to explicitly consider all important design inputs. Figure 8 (5,43-46) summarizes the general range in thickness requirements for aggregate-surfaced roads with and without the inclusion of fabric found from these design methods [except the method of ICI (47)] for the design conditions shown.

SUMMARY AND CONCLUSIONS

This paper has briefly reviewed the general performance characteristics of AFS systems, the mechanisms that contribute to the improved performance due to fabric, various factors that affect the performance of AFS systems, and methods of analyzing and designing AFS systems.

Figure 8. Typical aggregate thickness requirements for haul roads with and without fabric.



Based on the discussions presented, the following conclusions appear warranted.

1. Based on available information, the use of an interlayer of fabric in an aggregate-surfaced road can lead to better performance or, alternately, the fabric will allow reductions (25-40 percent) in aggregate layer thickness (compared with the AS system that contains no fabric) for the same level of performance.
2. Higher modulus fabrics appear to be of greater benefit to performance than lower modulus fabrics. Fabrics can provide reinforcement to the AFS system even before substantial rutting has occurred.
3. Numerous mechanisms responsible for the benefits have been postulated; however, the exact contribution of all these mechanisms has not been quantified.
4. Numerous factors, including component material (soil, fabric, and aggregate) and loading conditions, have major influence over the response of AFS systems. Recognition of these factors and integration of them into design and decisionmaking processes will lead to more predictable performance of the AFS system.
5. The behavior of the AFS system under repetitive loading is complex. Currently, no analytical models are available that correctly model this complex structure. Consequently, calculated structural response (stresses, strains, deflections) are often at best only estimates. Substantial work is yet to be accomplished in terms of analytical model development for the AFS system.
6. At least seven design procedures are available for determining the thickness requirements of AFS systems. Five of these are for specific commercial fabrics, however. These methods are primarily empirical in nature and in general do not consider all important design parameters. There is no general design procedure available that can accommodate a variety of fabrics with widely differing inherent properties and a variety of other design input values. There is need to develop a general, fundamentally sound, design method that can allow the potential fabric users to make valid economic decisions in selecting fabrics and design thicknesses for various job requirements.

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REFERENCES

1. J.R. Bell and others. Test Methods and Use Criteria for Filter Fabrics. Federal Highway Administration, Rept. FHWA-RD-80-021, 1980.
2. J.G. Lavin, C.D. Murray, L.E. Murch, Q.L. Robnett, and J.S. Lai. Prospects of Spunbonded Fabrics in Civil Engineering. Proc., Non-Woven Fabrics Conference, Univ. of Manchester Institute of Science and Technology, June 24-26, 1980.
3. Q.L. Robnett, J.S. Lai, and others. Use of Geotextiles in Road Construction: Laboratory Study. In First Canadian Symposium on Geotextiles, Calgary, Alberta, Canada, Sept. 1980, pp. 113-124.
4. J.S. Lai and Q.L. Robnett. Use of Geotextiles in Road Construction. Proc., Third Conference--Road Engineering Association of Asia and Australia, Taipei, Taiwan, April 20-24, 1981.

5. J.P. Giroud and L. Noiray. Geotextile-Reinforced Unpaved Road Design. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT9, Sept. 1981, pp. 1233-1254.
6. R.M. Koerner and J.P. Welsh. Construction and Geotechnical Engineering Using Synthetic Fabric. Wiley, New York, 1980.
7. P.R. Rankilor. Membranes in Ground Engineering. Wiley, New York, 1981.
8. D.A. Bender and E.J. Barenberg. Design and Behavior of Soil-Fabric-Aggregate Systems. TRB, Transportation Research Record 671, 1978, pp. 64-75.
9. T.C. Kinney. Fabric Induced Changes in High Deformation Soil-Fabric-Aggregate Systems. Univ. of Illinois, Urbana-Champaign, Ph.D. thesis, 1979.
10. V.A. Barvashov and others. Deformations of Soil Foundation Reinforced with Prestressed Synthetic Fabrics. Proc., International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, France, April 20-22, 1977, pp. 67-69.
11. L. Raad. Reinforcement of Transportation Support Systems Through Fabric Prestressing. TRB, Transportation Research Record 755, 1980, pp. 49-51.
12. M.R. Thompson. Mirafi 140 Fabric in Conventional Railway Track Support Systems. Department of Civil Engineering, Univ. of Illinois, Urbana-Champaign, Res. Rept., 1976.
13. J.R. Bell, D.R. Greenways, and W. Vischer. Construction and Analysis of a Fabric Reinforced Low Embankment. Proc., International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, France, April 20-22, 1977, pp. 71-76.
14. J. Steward, R. Williamson, and J. Mohney. Guidelines for Use of Fabrics in Construction and Maintenance of Low Volume Roads. Forest Service, U.S. Department of Agriculture, June 1977.
15. J.E. Gray. Characteristics of Graded Base Course Aggregates Determined by Triaxial Tests. National Crushed Stone Asssociation, Washington, DC, Engineering Bull. No. 12, July 1962.
16. H.L. Jessberger. Load-Bearing Behavior of a Gravel Subbase-Nonwoven Fabric-Soft Subgrade System. Proc., International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, France, April 20-22, 1977, pp. 9-24.
17. P.M. Jarrett, R.A. Lee, and D.V.B. Ridell. The Use of Fabrics in Road Pavements Constructed on Peat. Proc., International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, France, April 20-22, 1977, pp. 19-22.
18. A. McGown and K.Z. Andrawes. The Influence of Nonwoven Fabric Inclusions on the Stress Strain Behavior of a Soil Mass. Proc., International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, France, April 20-22, 1977, pp. 161-166.
19. T.C. Kinney and E.J. Barenberg. Soil Movement in Geotextile Reinforced Roads. In The Use of Geotextiles for Soil Improvement, Session 64, ASCE Convention, Portland, OR, April 1980.
20. P.M. Jarrett. Large Scale Model Tests on Gravel-Fabric-Peat Systems. First Canadian Symposium on Geotextiles, Calgary, Alberta, Canada, Sept. 23, 1980, pp. 101-112.
21. T.A. Haliburton and J. Fowler. Design and Construction of Fabric-Reinforced Roads and Embankments on Soft Foundations. First Canadian Symposium on Geotextiles, Calgary, Alberta, Canada, Sept. 23, 1980, pp. 137-148.
22. J.R. Kerr and D.L. Townsend. Experiences with Geotextiles in Soft Subgrade Clays. First Canadian Symposium on Geotextiles, Calgary, Alberta, Canada, Sept. 23, 1980, pp. 215-224.
23. S.L. Webster and J.E. Watkins. Investigation of Construction Concepts for Tactical Bridge Approach Roads Across Soft Ground. U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS, TRS-77-1, Feb. 1977.
24. S.L. Webster and S.J. Alford. Investigation of Construction Concepts for Pavements Across Soft Ground. U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS, TRS-78-6, July 1978.
25. Proc., International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, France, Vols. 1 and 2, April 20-22, 1977.
26. G.M. Hammitt. Thickness Requirements for Unsurfaced Roads and Airfields, Bare Base Support, Project 3782-65. Soils and Pavements Laboratory, U.S. Army Engineers, Waterways Experiment Station, Vicksburg, MS, TR2-70-5, July 1970.
27. S. Rodin. Ability of a Clay Fill to Support Construction Plant. Journal of Terramechanics, Vol. 12, No. 4, 1965, pp. 51-68.
28. R.V. Whitman and K. Hoeg; Massachusetts Institute of Technology. Development of Plastic Zone Beneath a Footing. U.S. Army Engineers, Waterways Experiment Station, Vicksburg, MS, 1966.
29. A.W. Skempton. The Bearing Capacity of Clays. Proc., British Building Research Congress, Vol. 1, 1951, pp. 180-189.
30. E.J. Barenberg. Design Procedures for Soil-Fabric-Aggregate Systems with Mirafi 500X Fabric. Civil Engineering Studies, Department of Civil Engineering, Univ. of Illinois, Urbana-Champaign, Transportation Engineering Series No. 30, Oct. 1980.
31. R.D. Barksdale. Repeated Load Test Evaluation of Base Course Materials. School of Civil Engineering, Georgia Institute of Technology, Atlanta.
32. R.D. Barksdale. Laboratory Evaluation of Rutting in Base Course Materials. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1972, pp. 161-174.
33. R.G. Hicks and C.L. Monismith. Factors Influencing the Resilient Behavior of Granular Materials. HRB, Highway Research Record 345, 1971, pp. 15-31.
34. S.F. Brown and A.F.L. Hyde. Significance of Cyclic Confining Stress in Repeated-Load Triaxial Testing of Granular Material. TRB, Transportation Research Record 537, 1975, pp. 49-58.
35. J.J. Allen. The Effects of Non-Constant Lateral Pressures on the Resilient Response of Granular Materials. Univ. of Illinois at Urbana-Champaign, Ph.D. thesis, 1973.
36. R.M. Knutson, M.R. Thompson, T. Mullin, and S.D. Tayabji. Material Evaluation Study. Ballast and Foundation Materials Research Program. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA/OR&D-77-02, Jan. 1977. NTIS: PB 264215.
37. E.Y. Huang and D.G. Lohmeier. Comparison of Soil-Aggregate Mixture Strength by Two Methods. HRB, Highway Research Record 52, 1964, pp. 1-9.
38. Q.L. Robnett and J.S. Lai. Use of Geotextiles to Extend Aggregate Resources. ASTM Symposium on Extending Aggregate Resources, Dec. 1980.
39. E.J. Barenberg. A Shear Layer Concept for Flexible Pavements. Department of Civil Engineering, Univ. of Illinois, Urbana-Champaign, Dec. 1967.
40. L. Raad and J.L. Figueroa. Load Response of

Transportation Support Systems. Transportation Engineering Journal, ASCE, TE1, Jan. 1980, pp. 111-128.
41. J.M. Duncan and C.Y. Chang. Nonlinear Analysis of Stress and Strain in Soils. Journal, Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM5, Sept. 1970, pp. 1629-1654.
42. R.D. Barksdale. A Nonlinear Theory for Predicting the Performance of Flexible Highway Pavements. HRB, Highway Research Record 337, 1970, pp. 22-39.
43. A Method for Constructing Aggregate Bases Using TYPAR Spunbonded Polypropylene. DuPont Company, Wilmington, DE, E-21317-1, Oct. 1978.
44. Constructing Access Roads with Mirafi 140 Fabric. PM-6, Celanese Fibers Marketing Company, New York, Sept. 1975.
45. Mirafi 500X Fabric, Design Guidelines: Haul Roads and Area Stabilization. MTB-1, Celanese Fibers Marketing Company, New York, Jan. 1979.
46. Bidim Engineering Fabric, Installation Guide for Access Roads, Storage Yards, Parking Lots, Logging Yards, Truck Terminals, and Board Roads. Monsanto Textiles Company, St. Louis, MO, 1979.
47. Designing with Terram, A Summary of Techniques and Physical Data Used in the Design of Terram/Soil Structures. Imperial Chemical Industries, Ltd., Pontypool, Gwent, Great Britain, Oct. 1977.
48. Development of CBR Flexible Pavement Design Method for Airfields Symposium. Transactions, ASCE, Paper No. 2406, Jan. 1949, pp. 453-589.

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Case for Removing Bridge or Culvert Rails on Low-Volume Rural Roads

BOB L. SMITH

Concrete or masonry bridge rails, parapets, or hubguards (if more than 4 in higher than the roadway surface) on narrow bridges and culverts on low-volume (ADT < 400) rural (LVR) roads are dangerous roadside obstacles. Based on the current state of knowledge it is suggested that, in many instances, striking the end of a rigid bridge-culvert rail is more hazardous to the motorist than traversing the adjacent stream bed or drainage area when rails have been removed. The case for rail removal is supported by the effective widening of the roadway due to rail removal, convenience to farmers in moving wide, low farm equipment, and benefit/cost ratios. The benefits are estimated reductions in annual accident costs and were calculated by using current published data on estimated collision frequencies, accident severity indices, and accident costs. The costs are the estimated costs of rail removal. There is a need for roadside hazard research aimed specifically at the problem of quantifying the hazard of vehicles that strike bridge-culvert rails versus the hazard of vehicles running off the road after rail removal.

The concrete or masonry bridge rails, parapets, or hubguards (if more than 4 in higher than the roadway surface) on narrow bridges and culverts are dangerous roadside obstacles or hazards.

A bridge rail is a longitudinal barrier whose primary function is to prevent an errant vehicle from going over the side of the bridge structure (1). It is apparent, in driving on low-volume (ADT < 400) rural (LVR) roads, that in many instances it would be far better for the vehicle to go over the side of the structure than to strike the bridge rail, especially the end of the rail.

Informal discussions with roadside hazard research engineers at Texas Transportation Institute and Southwest Research Institute of "When is it better, on LVR roads, to strike the rail rather than traverse the ditch next to the culvert or bridge?" resulted in the following consensus: It is almost always better to take to the ditch than hit the bridge rail--unless the ditch is very deep, steep, or the culvert or bridge has a large drop off to its bottom. In other words, the best safety strategy is to remove the bridge rails on narrow LVR structures.

The validity of this consensus is supported by the widely accepted priority of actions or strategies with regard to existing roadside obstacles (hazards) (2; 3, p. 340; 4):

1. Remove the obstacle,
2. Relocate the obstacle to a point where it is less likely to be struck,
3. Use breakaway devices to reduce the severity,
4. Use impact attenuation devices to reduce severity, or
5. Protect the driver through redirection of the errant vehicle (use of guardrails or roadside barriers).

A roadside barrier is a longitudinal barrier used to shield hazards located within an established minimum width clear zone (1).

Strategy 1, removal of bridge rails, is supported by the American Association of State Highway and Transportation Officials (AASHTO) (1, pp. 111, 3, 5, 15).

> ...Current criteria suggest that bridge rails should be installed on all bridge structures; however, the view is now held by some highway engineers that these criteria are too restrictive and in some cases have resulted in the unnecessary use of bridge rails. A possible example of this would be their use on a short structure that spans a shallow stream or drainage area on a low-volume rural roadway. Many such structures do not have an approach roadside barrier to shield the bridge rail end. It is likely that the exposed end of the rigid bridge rail is more hazardous to the motorist than would be the stream or drainage area. Judgment must therefore be used to determine if the overall hazard of the bridge rail and the approach roadside barrier necessary to shield the bridge rail end is less hazardous than the roadside condition being shielded. Warrants for barriers to shield culverts can be established from the criteria in Section III-A....
>
> ...It has been said that a traffic barrier is like life insurance--it is good to have as long as it is not needed. Although this is an over-

> statement, it cannot be overemphasized that a traffic barrier is itself a hazard. Every effort should be made in the design stage to eliminate the need for traffic barriers. Existing highways should be upgraded when feasible to eliminate hazardous conditions that require barrier protection. A traffic barrier should be installed discriminately and only when it is unfeasible to remove the hazardous condition...
> ...Typically the cost-effective procedure can be used to evaluate three options: (1) remove or reduce the hazard so that it no longer needs to be shielded, (2) install a barrier, or (3) leave the hazard unshielded. The third option would normally be cost-effective only on low volume and/or low speed facilities, when the probability of accidents is low.
> ...A clear, unobstructed flat roadside is highly desirable. When these conditions cannot be met, criteria to establish barrier need for shielding roadside objects are necessary. Roadside obstacles are classified as nontraversible hazards and fixed objects. These highway hazards account for over 30 percent of all highway fatalities each year and their removal should be the first alternative considered. If it is not feasible to remove or relocate a hazard, then a barrier may be necessary. However, a barrier should be installed only if it is clear that the barrier offers the least hazard potential....

In the AASHTO report (1) there are also several footnotes, tables, and special comments in recommended procedures that indicate that the fixed object should be removed or relocated, if practical, so that a barrier is unnecessary.

Strategy 2, relocate the obstacle, is usually not cost effective because of limited right-of-way, relatively high costs, and limited effectiveness on LVR roads (4). Strategies 3 and 4 are not applicable. Strategy 5, installation of guardrails, is not generally cost effective for any ranges of LVR volumes (4).

Thus, it appears there are two reasonable strategies to use on existing rails on narrow bridges or culverts on LVR roads:

1. Remove them.
2. Leave them as they now exist.

It would, of course, be very helpful if warrants were available for bridge rail removal. In the absence of such warrants the following is offered as guidelines for considering removal of rails.

GUIDELINES FOR REMOVAL OF BRIDGE RAILS

One of the primary practical reasons for removing the rails is for convenience to farmers in moving wide farm equipment (combines and discs, in particular) from one location to another. Most combines can be readily raised vertically 24 in above the roadway. Equipment, more than 24 ft wide, should be expected to be transported on trucks whose widths will generally be considerably less than 24 ft; thus, the clearance heights will be no problem. It follows, then, that bridges narrower than 24 ft that have rails over 24 in high on roads used for movement of wide farm equipment are likely candidates for rail removal. For safety, if the rails are removed they should be removed, preferably, to the height of the roadway surface or should extend no higher than 4 in above the surface.

Roadside Safety Considerations

Bridge and culvert rails are dangerous roadside obstacles (hazards). Where feasible, such roadside hazards should be removed since it is likely that the end of the rigid bridge rail is more hazardous to the motorist than the stream bed or drainage area. Judgment must be used in determining if the hazard of the bridge rail end is less than the hazard of the stream bed or drainage area. It appears likely that, for bridges or culverts 6 ft or less in depth, (i.e., the height of roadway surface above the stream bed), the bridge rail end is probably the greater hazard, and for depths greater than 9 ft the bridge rail end may be the lesser hazard.

For many narrow bridges and culverts it is obvious that the ditch at the culvert or bridge is no more hazardous than the mile after mile of unprotected roadside ditch. In these cases, the best strategy, from a safety standpoint, is to remove the bridge rail.

If the rail is removed, the roadway, in effect, is widened at that location. Figure 1 shows a minimum distance of about 0.5 ft from the centerline of outside wheel to inside of the rail for the safe traversing of a structure.

Figure 2 shows that, after rail removal, in an emergency the centerline of outside wheel could travel down the outside edge of the cut-off rail.

The effective widening, on one side of the roadway, is approximately w+0.5, where w is the width of rail in feet. For a 6-in rail width the widening of the roadway is 1 ft and for a rail width of 12 in the roadway widening is 1.5 ft. Where both rails are removed, the total roadway will be widened 2-3 ft.

In the case of a 16-ft-wide bridge-culvert, this provides an 18- to 19-ft clear roadway after rail removal, an increase in roadway width of 13-19 percent. This additional roadway width must certainly contribute to increased safety at the bridge site. A Minnesota study (6) reports on numerous studies that have documented that lane widths of 11 or 12 ft are significantly safer than 9- or 10-ft lanes. The same study shows a decrease in the accident rate of about 15 percent when pavement widths for rural highways are increased from 18 to 20 ft. Surely, then, there must be a significant increase in safety at a site where narrow bridge widths, from 16 to 22 ft, are increased by 2 or 3 ft. This should be especially significant when the width of an existing bridge is narrower than the approaching roadway.

Figures 3 and 4 show that the width in which a vehicle can impact the end of the bridge rail is about twice the width of the vehicle. The AASHTO report (1) uses an effective width of the vehicle of 92 in to represent an automobile in a partial skid. This shows that there is a 12-15 ft width in which a vehicle can impact the end of the rail. Any impact with the bridge rail end will probably result in a severe accident with a high probability of a fatality or injury accident occurring (1). The severity index (2) is estimated at 9 and estimated cost per accident is $160 000.

Assume now that the rail has been removed. If a vehicle encroaches as shown in Figures 3 or 4, it appears likely that:

1. The vehicle's outside wheel may stay on the top of the cutoff rail (Figure 3), in which case the vehicle will incur little or no damage and the probability of an injury or fatality occurring is very low.
2. The outside wheel may drop over the cutoff edge, which will probably cause vehicle damage and perhaps injuries to occupants. From a Texas Transportation Institute (TTI) report (5, p. 25), this might be compared with striking a culvert headwall

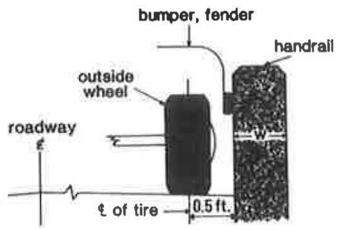


Figure 1. Effective roadway width before rail removal.

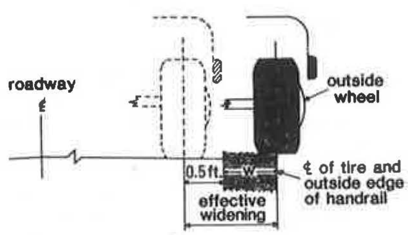


Figure 2. Effective widening of roadway on one side after rail removal.

Figure 3. Impact on outside edge of vehicle.

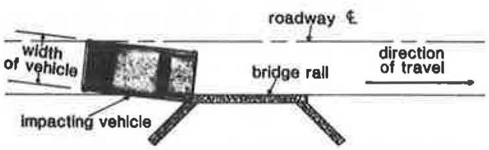
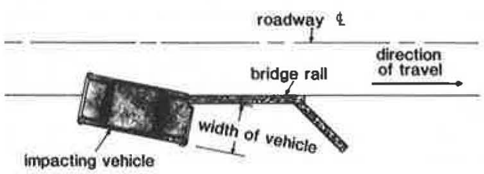


Figure 4. Impact on inside edge of vehicle



with a severity index of 7.9 with an estimated cost per accident of $120 000.

3. The vehicle may vault over the edge and land in the drainage ditch. If the ditch is low and relatively smooth, the severity index is estimated to lie between 3 and 5 and costs per accident between $6000 and $17 000, respectively (5, p. 25).

By using material in the TTI report (5, p. 25), the estimated annual accident costs for the above examples are given in Table 1 for average daily traffic (ADT) of 50, 100, and 200 vehicles.

For narrow bridges one would also expect some additional accident costs based on the probability of striking the bridge rail on the left. This additional cost is not included in Table 1. Note that the annual accident costs in Table 1 are for one side of a structure. The annual costs for rails on both sides of the road would be expected to be about twice the costs in Table 1.

The accident costs in Table 1 suggest that the removal of a rail on one side of a bridge will result in the reductions in annual accident costs as shown in the table below.

ADT	Accident Cost Reduction ($/year) Range	Average
50	0-88	44∿50
100	0-192	96∿100
200	0-400	200∿200

Table 1. Annual accident costs for various severity indexes and ADTs.

Severity Index	Cost per Accident ($)	Annual Accident Costs[a] ($) ADT = 50 cf^b = 0.000 55	ADT = 100 cf^b = 0.0012	ADT = 200 cf^b = 0.0025
3	6 000	3	7	15
5	17 000	9	20	43
7.9	120 000	66	144	300
9	160 000	88	192	400

Notes: Assumption is that bridge rail is 10 ft long x 1 ft wide, located at roadway edge.
cf = collision frequency (accidents per year).
[a]Annual accident costs = cost per accident x cf (accidents/year).
[b]From TTI report (2, Figure 5.1.17).

Table 2. Present worth of future accident cost reductions.

ADT	Approximate Avg Annual Cost Reduction ($)	Present Worth of Future Accident Cost Reductions for n years ($) n = 1 Year	n = 2 Years	n = 5 Years	n = 10 Years
50	50	45	87	190	305
100	100	90	174	380	610
200	200	180	348	760	1220

Note: Present worth = average annual accident cost reduction x $^{i}_{n}$PWF where PWF = present worth factor at interest rate i and n = number of years' accident reductions; i = 10 percent; n = 1, 2, 5, 10 years; and $^{10}_{1}$PWF = 0.9, $^{10}_{2}$PWF = 1.74, $^{10}_{5}$PWF = 3.8, $^{10}_{10}$PWF = 6.1.

Table 3. Benefit/cost ratios.

ADT	Cost of 1 Rail Removal ($)	B/C Ratios n = 1 Year	n = 2 Years	n = 5 Years	n = 10 Years
50	50	0.90	1.74	3.80	6.10
	100	0.45	0.87	1.90	3.05
	500	0.09	0.17	0.38	0.61
100	50	1.80	3.48	7.6	12.2
	100	0.90	1.74	3.8	6.1
	500	0.18	0.35	0.76	1.22
200	50	3.60	6.96	15.20	24.40
	100	1.80	3.48	7.60	12.20
	500	0.36	0.70	1.52	2.44

Table 2 gives the present worth of various future accident cost reductions for ADTs from 50, 100, and 200 vehicles.

The benefit/cost (B/C) ratios for various ADTs, cost per rail removal, and number of years of accident reductions are given in Table 3.

Benefits are assumed to be the present worth of future accident cost reductions for a selected ADT and number of years of accident reductions (n).

Costs are the cost of removing one rail.

For example, the B/C ratio for ADT = 100, cost of one rail removal = $50, and n = 2 years is

B/C = $174 (from Table 3)/$50 (cost to remove one rail) = $3.48

B/C ratios greater than 1.0 show that the benefits received are greater than the costs incurred.

It is apparent from Table 3 that significant economic benefits are gained from rail removal, especially when the costs per rail removal are in the $50-$100 range.

The benefits may be even greater if the outside rails are removed from the bridges or culverts on horizontal curves. Glennon (4), the Minnesota study (6), AASHTO (1), and TTI (5, p. 25) clearly show the need for additional roadway widths on the outside of horizontal curves.

One of the vexing problems with the removal of rails from existing narrow bridges or culverts is that of liability of the local government unit. Since most of the structures in question were built many years ago under then-prevailing width and roadside barrier standards, it appears that leaving the structures as they exist will not result in liability for the governmental unit.

On the other hand, their removal may increase safety. It seems likely that if rails are removed and an accident occurs there may be some lawsuits in which it is claimed that striking the old rail was safer than the new design (i.e., rail removed). Claims that the old rail was safer and proof of this are probably more difficult to deal with than proof that rail removal is safer.

RESEARCH NEEDED

There is a need for some roadside hazard research aimed specifically at this problem. County or local road engineers need warrants for bridge and culvert rail removal or a set of guidelines for quantifying the hazard of vehicles striking various bridge and culvert rails versus the hazard of vehicles running off the road into the adjacent ditch, stream bed, or drainage area.

It would be most helpful if a set of, say, severity indexes were developed for specific roadside hazards on LVR roads considering typical speeds, roadway cross-sections, alignment, and roadside hazards.

ACKNOWLEDGMENT

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REFERENCES

1. A Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, DC, 1977.
2. Roadside Safety Design. Texas Transportation Institute, Texas A&M Univ., College Station, 1970.
3. Dynamic Design for Safety. Institute of Traffic Engineers, Arlington, VA, Oct. 1975, p. 340.
4. J.C. Glennon. Design and Traffic Control Guidelines for Low-Volume Rural Roads. NCHRP, Rept. 214, 1979, 41 pp.
5. Texas Transportation Institute. A Supplement to a Guide for Selecting, Designing, and Locating Traffic Barriers. U.S. Department of Transportation; Texas Transportation Institute, Texas A&M Univ., College Station, 1980.
6. J.E. Leisch and T.R. Newman. Study of Width Standards for State Aid Streets and Highways, Volume II: Research Report and Appendices. Local Road Research Board, Minnesota Department of Transportation, St. Paul, 1979.

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Cost Responsibility for Low-Volume Roads in Virginia

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Cost responsibility is a research tool for determining the amount highway user groups should contribute to the financing of highways. Several cost-responsibility studies have been conducted at the national and state levels; however, most have omitted from analysis the cost responsibility for low-volume roads. This study presents the method and calculations of cost responsibility for Virginia's 43 000 miles of low-volume roads. Costs were divided for allocation purposes into three categories—occasioned, demand-driven, and common costs. Costs in the categories are divided among four vehicle classes by various methods. Data and methods for three major cost areas are described in detail: site preparation and geometry, pavement construction, and pavement repair and resurfacing. The results of the study show that 75 percent of the costs on low-volume roads is the responsibility of cars and light trucks. The remaining 25 percent is the responsibility of heavy trucks. The study also shows that on low-volume roads the per mile cost responsibility for each vehicle class is more than twice that of high-volume roads.

Cost responsibility has emerged as a central issue in nearly every recent discussion of highway financing. The term cost responsibility has come to mean an analysis of the extent to which highway user groups contribute an equitable share of the costs of financing highways. At the federal level, Congress mandated the completion of a four-year cost-responsibility study by 1982; in the states, at least 20 cost-responsibility studies have been undertaken in the past few years.

Interest in cost responsibility has been spurred by the impact of several recent phenomena on the highway financing system. Highways in the United States are unique among publicly financed facilities in that they have been financed largely by those who use the roads. At the national level, about 70 percent of highway funding has come from user payments (1). The dedication or earmarking of user payments for highway financing has been held to be a major factor in the comparatively high level of development of the highway system (2). In recent years, however, funds from user payments have not met expectations for the continuing development of the system. The Arab oil embargo, mandated increases in fleet fuel efficiency, restrictions on the supply of motor fuel, and general economic malaise have contributed to the revenue shortfall.

The failure of established user tax sources to produce the expected amount of revenues gives impetus for a change in the level or structure of highway taxes. From a political perspective, it is easier to pass tax increases if the burden is distributed fairly. In a social sense, the charges for highway use can suboptimize the resource allocation and consumption patterns for highways. This can be accomplished by establishing an equitable and efficient taxing burden (1,3). An equity based cost-responsibility study may, therefore, provide the basis for a politically feasible and socially rational change in taxing.

Cost-responsibility studies under way at this time have encountered numerous difficulties. The conceptual sticking points of allocating costs of joint-use facilities and long-run versus short-run marginal cost pricing plague the studies. In addition, limited data on highway use and the costs that use generates make the task difficult. Another problem, the cost responsibility for low-volume roads, is the focus of this paper.

LOW-VOLUME ROAD ENIGMA

Although cost-responsibility studies have been touted as important tools for highway financing, low-volume roads have been neglected. In light of the recent activity surrounding cost responsibility and the magnitude of expenditures on low-volume roads, the lack of research in the area is an enigma. Several reasons for not applying cost responsibility to low-volume roads can be identified.

First, cost-responsibility studies are usually sponsored by the federal or a state government. Most low-volume roads are not on the federal system and only four states have direct responsibility for a significant portion of their low-volume roads. Since the study sponsors are most concerned with analyzing the road systems they administer, low-volume roads are not included in the analysis.

Second, states have generally preempted the levy of user taxes by local government. Localities, which typically control low-volume roadways, are therefore unable to use a study to establish a system of equitable user payments. Low-volume roads are generally supported from general fund (nonuser) sources.

Third, data on low-volume roads have not been routinely collected in a manner consistent with the data for higher-volume roads. Usage data and data on the cost of maintenance activities are two examples of data that are necessary but may be lacking. Furthermore, data that are available may be collected differently in each administering jurisdiction. Because the data are expensive to obtain and difficult to manipulate in the same way as data on high-volume roads, the task of determining cost responsibility for low-volume roads can be cumbersome and difficult.

Low-volume roads have not, however, been excluded entirely from cost-related analysis. Two methods of analyzing low-volume road costs were used in the 1961 federal study--relative use and earnings credit. These procedures were designed to split the costs of low-volume roads into user and nonuser portions. The rationale underlying these procedures was that access benefits accrued to nonusers by the provision of the roads. These benefits were, in turn, used as justification for property tax (nonuser) support of low-volume roads.

Although the 1969 federal study clung to the earnings-credit method, little theoretical support exists for the peculiar treatment of access benefit (3,4). Access benefit is accrued as a roadway is used--the greater the use, the greater the benefit. Of course, some types of use derive greater benefits than others, and therein lies the general problem with benefit analysis. The access benefit argument must stand or fall with the decision on whether cost-occasioned or benefit received is to be used as a mechanism for setting the tax burden. If a cost-occasioned approach is executed, no exception should be made for low-volume roads.

Low-volume roads have a range of costs that are incurred to accommodate the use of the roads by vehicles of various types. The revenues produced through the use of those vehicles are not sufficient to recover the expenditures on the roadways. Thus, the charges to recover the costs of low-volume roads must be generated from nonuse or lump sum charges. This does not mean that a nonuser or property tax base is appropriate. Other tax sources, such as registration fees, are available for low-volume road support. The decision to allocate a portion of low-volume road costs to nonusers on the basis of access benefit is therefore arbitrary.

The traditional affiliation between low-volume-road expenditures and nonuser tax support was never established in Virginia. Since 1932, the relatively low-volume secondary road system has existed within the state highway system. Funding for the entire system has been generated primarily through user payments [e.g., motor fuel taxes (59 percent), registration fees (15 percent), and sales tax on motor vehicles (15 percent)]. Thus, a cost-responsibility study of Virginia's highways offered the opportunity to include a cost-occasioned analysis of secondary roads without violating the financing tradition established in the state.

SETTING FOR THE STUDY

When the 1980 session of the Virginia General Assembly mandated a study of the fair apportionment and allocation of highway expenditures among motor vehicles of various sizes and weights, the conditions were ripe for a study of cost responsibility on low-volume roads. As mentioned, the tax base for all highway systems was user-oriented and the state administered the low-volume, secondary road system. Moreover, the importance of the secondary system relative to the other systems was impossible to dismiss.

Secondary roads make up 43 000 miles of Virginia's 67 000-mile system. In addition, 67 percent of the state's secondary roads are paved, in contrast to the national average of 32 percent for the other 49 states. Secondary road expenditures are also a major consideration: approximately 40 percent of the state's maintenance funds and 16 percent of the state's construction funds are expended on secondary roads. These factors made the inclusion of secondary roads a necessity in an analysis of all highway expenditures.

As Table 1 indicates, traffic patterns differ significantly between Virginia's low- and higher-volume roads. As might be expected, traffic on secondary roads shows a higher concentration of passenger cars and pickup trucks. In addition, truck mileage on secondary roads is more skewed toward single-unit trucks than combination vehicles. A difference in expenditure patterns and cost allocations on the systems is expected from the difference in the traffic streams. Therefore, a detailed analysis of the expenditures on secondary roads was needed.

One caveat is necessary here--Virginia's secondary roads do not encompass all low-volume roads in the Commonwealth, and all secondary roads are not exclusively low volume. The state's urban system, which contains at least 7046 miles of low-volume roads, is not included in this presentation, although the state-supported costs of this system were included in the study mandated by the General Assembly (5,6). In addition, some secondary roads in urbanized counties accommodate high-volume traffic. For example, the two highest-volume secondary projects in the study's sample of construction projects had a mean average daily traffic of more than 18 000 cars. Nonetheless, the vast majority of the secondary system is low volume in nature.

Table 1. Travel by each vehicle class on primary and secondary roads.

		Proportion of Travel (%)	
Vehicle Class	Description	Primary Roads	Secondary Roads
1	Passenger cars and light trucks	90.4	93.2
2	Two-axle, six-tire trucks	3.9	4.6
3	Three-axle, single-unit trucks	1.4	1.5
4	Tractor-trailer combination trucks	4.3	0.7

FRAMEWORK FOR COST RESPONSIBILITY

An analysis of cost responsibility determines whether users are contributing an equitable share of the costs of operating the highway system. The cost-occasioned approach defines the costs incurred on behalf of a user to be the costs for which that user is responsible. For example, ferries that transport only automobiles should be supported by the revenues generated by automobiles.

A central problem for cost responsibility arises because the highway system is built to accommodate a variety of vehicles. Different vehicles have a wide range of requirements for pavement width and strength and for the amount of roadway required. The approach followed in this study was to divide highway costs into three categories:

1. Occasioned costs--Costs incurred as a result of some characteristic of the vehicle stream (e.g., weight or size),
2. Demand-driven costs--Costs attributed on the basis of the relative demand exercised by components of the vehicle stream, and
3. Common costs--Costs that are not causally linked to vehicle characteristics or demand and are attributed on the basis of overall highway system use.

IMPLEMENTING THE STUDY

Several key decisions were guided by the mandate provided by the General Assembly. Since the study was to determine the equitable apportionment of user tax burden for Virginia highway users, the state's highway system, construction and maintenance practices, and vehicle use pattern formed the basis for analysis. Therefore, an extensive effort was mounted to develop attribution procedures consistent with the design, construction, and maintenance practices of the Virginia Department of Highways and Transportation (VDHT).

The mandate also provided guidance for the definition of costs to be considered in the study. Cost responsibility can be construed to include all costs generated by the highway system--public, private, and opportunity costs. However, the study mandate asked for a cost-responsibility analysis as a highway financing tool. Hence, costs were defined as the total expenditures on the Virginia highway system.

In defining costs, care had to be exercised to ensure that actual or proposed expenditures fully captured the cost to the public of providing a highway system. If some expenditures, particularly for maintenance, were being deferred, then present costs would have been underestimated and passed to future taxpayers.

Detailed analysis of the possibility of highway deterioration indicated that highway disinvestment was not occurring in significant amounts for the purposes of the study. Relevant maintenance replacement expenditures per lane mile, adjusted for inflation, showed real growth over the past 10 years. Other data regarding the nature of highway construction showed that much current construction consists of major reconstruction, resurfacing, and rehabilitation of existing highway facilities. Moreover, state maintenance engineers consistently judged that no appreciable, premature, structural deterioration was occurring on the state's highways. For these reasons, expenditures were judged to be the relevant measure of highway costs.

Finally, a decision on classifying vehicles had to be made in order to implement the study. The legislative mandate called for a study of cost apportionment among vehicles of various sizes and weights. In theory, a separate cost-responsibility estimate could be calculated for each individual who uses the highway system. Since calculation of millions of individual equations is not possible, cost-responsibility analysis requires a method for classifying users in some meaningful fashion.

Classification was achieved by grouping vehicles into a manageable number of categories based on the following:

1. Characteristics directly associated with how costs are occasioned,
2. The way in which vehicles are defined by law and are taxed, and
3. The way in which traffic and registration data are collected.

Based on these three criteria, four vehicle classes were selected to provide a basis for subsequent analysis.

Class 1--All passenger cars, pickup trucks, panel trucks, and motorcycles;
Class 2--All two-axle, six-tire trucks;
Class 3--All three-axle, single-unit trucks; and
Class 4--Three-, four-, and five-axle tractor-trailer combinations.

COST-ALLOCATION ANALYSIS

Low-volume roads are typically designed by using different procedures than those applied to Interstate and primary highways. In addition, reliable data on expenditures, structural conditions, travel patterns, and vehicle weights may be difficult to acquire. These and other problems generally require that the cost allocation for low-volume roads be addressed separately during cost-responsibility studies.

Site preparation, pavement construction, and pavement maintenance are among the most frequently discussed elements of cost allocation. They represent the highest expenditure elements of most highway systems and include the majority of costs occasioned by particular vehicle characteristics. This section discusses the procedures used to allocate these costs for Virginia's secondary roads. The data and findings apply only to the study's base period, FY 1980.

Roadway Construction Data

Site preparation and pavement construction are the principal costs involved in roadway construction. They include the costs most likely to vary with the size and weight characteristics of the traffic stream. In order to allocate these costs equitably, it was necessary to examine empirically the degree to which vehicle characteristics govern the cost of secondary road construction. A sample of 61 roadway construction projects provided the means for this examination.

Project Sample

In order to ensure that the analysis would not be unduly biased by any single construction project, all projects completed during a fiscal year were selected for the sample. The 61 projects selected ensured that actual costs incurred and materials used would condition the cost allocation.

All projects included in the sample were examined to ensure that they were representative of Virginia's secondary road construction program. The examination showed an appropriate geographical distribution and also showed diversity in project cost, size, and in the nature of construction. As a final check on the representativeness of the sample, the state highway department's construction engineer, in concert with construction division personnel, certified that the project sample represented a typical year.

VDHT maintains an automated file on each construction project that is presented for bid, is under way, or has been completed. These files form the basis of the department's system for estimating project costs. Data kept on the 61 secondary projects included materials costs and quantities for all items included on contract bids. The project files provided the means for determining how pavement and site preparation costs vary with varying traffic.

Because the 61 projects completed during FY 1980 had been initiated over several years, it was necessary to standardize prices for like activities and material quantities. Failure to standardize prices would have caused projects initiated more recently, which showed higher prices caused by inflation, to weigh disproportionately in the overall sample. Item prices were standardized to mid-FY 1980 levels.

State Force Construction

In Virginia, some secondary construction projects are conducted entirely by state highway department personnel. Such projects are statutorily limited to $200 000 and typically involve adjustments to roadway geometry, some resurfacing, or safety improvements. Although these projects represent a small portion of secondary construction, their costs had to be accounted for. However, data on materials quantities and prices are not maintained as accurately as data on contracted projects. It was therefore necessary to compensate for state force construction by adjusting the project sample.

To handle this problem, the 61 secondary construction projects were examined by the state's engineer for secondary construction to see if any projects were directly comparable to the types of projects conducted by state forces. The cost of 12 secondary projects identified as comparable were weighted to represent the cost of state force construction.

Project Clustering

The projects in the sample were grouped to ensure that projects that had similar characteristics would be analyzed together. The grouping of projects also allowed a reduction in the number of projects to be analyzed.

Most cost-responsibility studies reviewed as part of the study literature search simply group projects by administrative or functional classification. However, these classifications often overlap in significant design and traffic features that are used as the basis of cost allocation. In Virginia this is true even for the secondary road system. Although most secondary roads are low volume, there is considerable variation in this regard. Secondary projects included in the sample ranged from the typical two-lane, undivided roadway to a small number of four-lane, divided highways.

Table 2. Clusters of secondary construction projects.

Subgroup Characteristics	No. of Projects	Mean Weighted ADT
2 lanes, less than 12 ft lanes	10	79.3
	24	295.0
	7	708.6
	8	1 982.5
2 lanes, 12 ft lanes	3	623.5
	7	4 263.0
4 lanes, 12 ft lanes	2	18 263.0

The roadway characteristics judged most relevant for cost allocation were the number of lanes, lane width, and traffic. Projects were therefore grouped into clusters based on these characteristics. The process of grouping projects into clusters consisted of two steps.

In step 1 the number of lanes and lane width for each project was used to divide the secondary projects into three subgroups. Projects were divided into subgroups of: 2-lane, less than 12-ft lane projects; 2-lane, 12-ft lane projects; and more than 2-lane, 12-ft lane projects.

In step 2, within each subgroup of projects, clusters were formed by computing the mean and standard deviation of the weighted average daily traffic total, which is used in designing secondary pavement depth. Beginning at the mean, cluster boundaries were formed by moving up (or down) one standard deviation at a time. By this procedure, all secondary projects were enclosed in clusters equal to one standard deviation in length.

Seven clusters of secondary construction projects were identified by this process. Table 2 shows the number of projects, the number of lanes and lane width, and mean weighted average daily traffic (ADT) for each cluster. The clustering procedure allowed a reduction in workload and minimized the effect of aggregation bias, which generally exists if projects are not grouped homogeneously.

The project clusters were used in the allocation of the costs of site preparation and pavement construction.

Site Preparation and Roadway Geometry

Site preparation includes all activities directly related to the construction of a road, except the laying of pavements. In general, the activities are mobilizing the construction crew and equipment, clearing and grubbing, excavating, grading, installing drainage facilities, and providing improvements such as signs, signals, and vegetation. Together these activities amounted to 53.9 percent of secondary construction expenditures in the study year.

Site preparation requirements and costs vary with the size of vehicles that the roadway is designed to carry. For example, wider vehicles require wider lanes and shoulders. The costs of excavation, drainage structures, and other materials are therefore increased. Heavier vehicles require thicker pavements and generate higher excavation costs associated with preparing deeper trenches for pavement.

To determine the proportion of costs occasioned by large, heavy vehicles, an incremental technique was applied. Mixed-traffic design standards currently used by VDHT were examined to identify which aspects of roadway design could be reduced if the roadway were used only by small, light vehicles (class 1). With safety and speed considerations

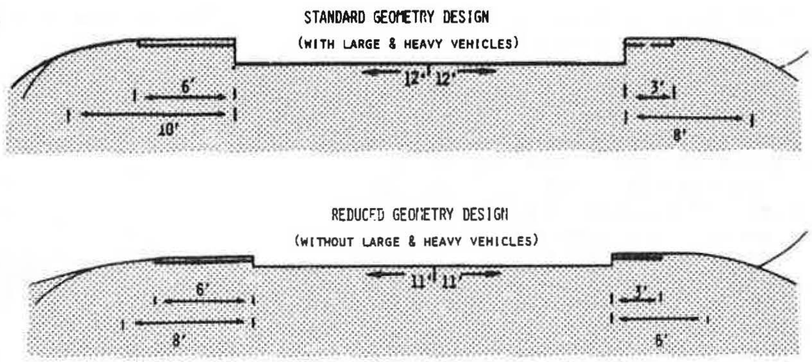


Figure 1. Example of geometry reduction.

Table 3. Sample cost reduction from project redesign.

Site Preparation	Design Cost ($) Standard	Design Cost ($) Reduced	Cost Difference ($)
Mobilization	351 429	190 889	39 552
Excavation	2 043 665	1 787 778	255 887
Drainage	292 222	288 706	3 516
Traffic and roadside improvements	785 101	782 994	2 107
Total	3 351 429	3 050 367	301 062

held inviolate, three size-related reductions were possible for most clusters:

1. Lane width could be reduced by 1 ft, from 12 to 11 ft;
2. Cut shoulders could be reduced by 2 ft, from 8 to 6 ft; and
3. Fill shoulders could also be reduced 2 ft, from 10 to 8 ft.

As is discussed in the next section, when heavy vehicles were removed from the traffic stream, thinner pavements were also possible. Therefore, the redesigning of site preparation requirements for a thinner pavement allowed a reduction in trench depth. Because the degree of trench depth reduction depended on the depth of the original pavement, the amount of reduction was computed for each cluster. The roadway cross section shown in Figure 1 illustrates the reductions possible for a two-lane, undivided roadway.

Assumptions about cost reductions were tested empirically. VDHT design engineers were asked to redesign actual projects to determine the cost reductions associated with removing large, heavy vehicles from the traffic stream. This procedure ensured that cost reductions were based on actual design practice rather than on theoretical estimates.

A project for each cluster was selected for redesign in order that differences in geometric designs (i.e., 11-ft lanes versus 12-ft lanes) would be accounted for. Preference in selection from a cluster was given to projects that included all major cost elements of site preparation and roadway construction and that helped produce an even geographic distribution of projects across the state.

The difference between the site preparation and roadway geometry costs for the standard design and the reduced design was then used as an estimate of the truck-occasioned increment of site-preparation costs. Table 3 illustrates the results of the reduction for one project, which shows a 9 percent reduction from standard to reduced design.

The degree of reduction possible for each cluster was determined by applying the proportional reduction generated from the project redesign to the site preparation and roadway geometry cost for the entire cluster.

Costs associated with the truck-occasioned increment were assigned to a vehicle class on the basis of their relative use of roadways in the cluster. Costs associated with the reduced design were assumed to be a function of the general demand for the basic roadway facility. The cost of the reduced design was therefore categorized as a demand-driven cost and charged to all vehicles on the basis of relative use. Total cost responsibility for each cluster was first summed and then weighted to equal total FY 1980 expenditures for secondary site preparation and roadway geometry. The table below gives the results of that allocation.

Vehicle Class	Secondary Road Site Preparation and Roadway Geometry Allocation: Cost Responsibility ($)	Percent
1	33 963 446	90.9
2	2 473 315	6.6
3	576 048	1.5
4	362 280	1.0
Total	37 375 089	

Pavement Construction

Pavement construction expenditures accounted for 24.0 percent of roadway construction costs in the study year. Pavement construction included pavements that were laid in construction and reconstruction projects during FY 1980. Other pavement work, such as rehabilitation and replacement, was included in maintenance costs.

Some pavement costs are occasioned by vehicles because they demand wider lanes and thicker pavements. Based on the preceding analysis, it was evident that 1 ft of each 12 ft-lane is required solely for large vehicles. Costs associated with this width increment are truck occasioned.

Pavement depth for the entire lane width is occasioned by factors related to axle weights and the repetitions of axle weights. Pavement cost allocation must therefore be sensitive to both the axle weights and volume of traffic on the roadways.

Pavement Design

Pavement engineering design criteria were used to determine the relation between axle weight and traffic volume on the one hand and pavement depth on the other. The design criteria were originally developed in the American Association of State Highway Officials (AASHO) road tests conducted in Ottawa, Illinois, and were modified in Virginia to serve as the basis for pavement design.

For most roads VDHT uses the 18 000-lb equivalent single axle load (ESAL) measures developed in the AASHO tests to determine required pavement strength. In practice, estimates of daily ESALs for the 10th year (regarded by VDHT as the design year) are used to compute an index of the necessary pavement thickness [referred to as thickness units (T.I.)].

In secondary roads, the design process is somewhat different. Secondary roadway pavements are designed from a nomograph that uses a weighted average daily traffic total. Although ESALs are not directly used in the design, empirically derived assumptions about vehicle weights are included in the nomograph calibration.

The basic objective of pavement cost allocation is to separate a given thickness of pavement into two components: (a) a component that is directly related to the expected weights of vehicles that use the road, and (b) a component that is principally the result of the strength and bonding requirements necessary to preserve the pavement through weathering cycles. The first component is weight-related and allocated to the vehicles that create demand for the pavement because of their weight (Figure 2). The second component is more appropriately considered a demand-driven common cost because it is principally related to weathering (5,6).

Pavement Cost Allocation

The minimum pavement method was used as the primary pavement allocation method for both high- and low-volume roads. It is also being proposed for use in the cost-allocation study being conducted by the Federal Highway Administration and has been endorsed in concept by the American Consulting Engineers Council.

The minimum pavement method begins by determining the amount of pavement to be laid if weight were so low as to be an inconsequential factor in pavement design. VDHT pavement engineers concluded that, in Virginia, the minimum pavement equals 3.6 T.I., the practical equivalent of 6 in of crushed stone base covered by a sealant coat. Pavement thickness required above 3.6 T.I. must be concluded to be related to the axle weights of vehicles in the traffic stream.

Existence of the minimum pavement is best conceived as being related to the demand for the basic roadway facility. Accordingly, costs associated with the minimum pavement can be allocated by a measure of relative roadway use. In this study, the cost of the minimum pavement for each cluster was allocated by each vehicle class's proportion of ADT for the cluster.

Because all pavement above the 3.6 level is weight-related, pavement above the minimum was allocated by the proportion of ESALs contributed by each vehicle class. The handling of the weight-related portion of pavement in this manner distributes equitably the inherent economy of scale in pavement construction.

Application of the minimum pavement method to secondary pavements is more complicated than application of it to Interstate and primary highway pavements. As previously indicated, a nomograph rather than an ESAL-driven equation is used to determine required pavement depth. To avoid a discrepancy between the amount of pavement determined by the ESAL-driven and secondary road procedure, the nomograph was used to compute T.I. ESALs equivalent to the total weighted average traffic for each vehicle type were then used to allocate the pavement. Table 4 shows a sample calculation that uses this procedure.

Figure 2. Pavement cost-allocation illustration.

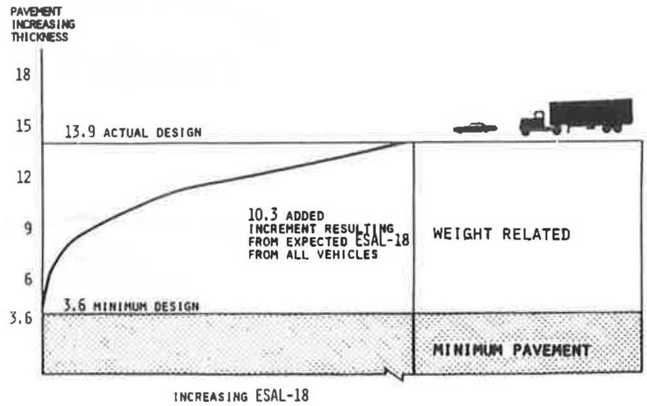


Table 4. Sample allocation of secondary pavement construction costs.

			Cost ($)			
Vehicle Class	ADT (%)	ESALs (%)	Minimum Pavement	Weight-Related Pavement	Total Class Attribution	Percentage of Cost
1	91.9	1.3	512 753	10 960	523 859	37.4
2	5.3	38.7	30 687	326 287	357 972	25.6
3	1.2	20.7	6 695	174 526	179 757	12.8
4	1.4	39.3	7 811	331 346	339 478	24.2
Total			557 947	843 119	1 401 066	

This procedure was used to allocate pavement construction costs for the seven secondary clusters. One cluster contained projects that have pavement thickness indices below the 3.6 minimum. Pavement costs for this cluster were allocated on a proportional traffic basis. All other clusters' pavement costs were allocated by the minimum pavement method. The table below summarizes the results of that allocation.

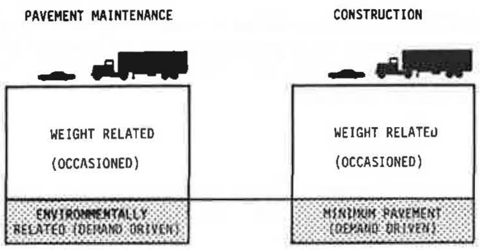
Vehicle Class	Secondary Road Pavement Construction Allocation Cost Responsibility ($)	Percent
1	9 486 473	56.8
2	3 559 709	21.3
3	1 968 351	11.8
4	1 659 588	10.1
Total	16 710 121	

Pavement Repair and Replacement

Pavement maintenance refers to an assortment of activities designed to inhibit or reverse the effects of pavement deterioration. The activities range from seal coating, skin patching, and pothole filling to resurfacing existing roadways. Pavement maintenance had the single highest expenditure of all secondary maintenance activities and accounted for 44.4 percent of total maintenance costs in FY 1980.

A principal concern in allocating pavement maintenance costs is distinguishing the amount of pavement deterioration caused by axle weights, which is occasioned by vehicle classes, from the amount caused by environmental factors, which is unrelated to vehicle use. Although the AASHO road tests establish the direct relationship between the number of ESALs and pavement deterioration, the results are

Figure 3. Pavement maintenance-allocation illustration.



not directly applicable to the problem of pavement maintenance. The tests lasted only two years, an insufficient period to simulate normal weathering cycles. Moreover, because little routine maintenance of the pavement surface was performed during the tests, pavement deterioration was artificially accelerated.

In recognition of the gaps in technical knowledge regarding pavement damage over time, the Federal Highway Administration has contracted with consultants to produce estimates of the proportion of pavement deterioration that results from weight and the proportion that results from environmental conditions. Even when the studies are completed, they are likely to be subject to much additional review. In lieu of empirically confirmed results, estimates regarding weight and environmental deterioration must be developed judgmentally.

This study used an alternative approach. For this study, the problem was characterized as drawing a line through a range of potentially reasonable estimates of weight-related versus environmentally related deterioration (Figure 3). The range of potential estimates is shown as the shaded area and can be labeled the zone of uncertainty. As Figure 3 illustrates, a decision was made to draw the line through the zone of uncertainty on the same basis as the division between weight-related and minimum pavement components in the secondary pavement construction allocation.

Beside providing results that were compatible with those derived from estimates used in other states, the use of an estimate related to construction allowed the study results to be sensitive to highway system differences. For secondary roads, the cluster-based pavement allocation showed that the 3.6 T.I. minimum pavement represented 46.9 percent of pavement costs; 54.1 percent of pavement costs were weight-related. This same proportion was used to split pavement maintenance costs. The environmentally related portion was allocated as a demand-driven common cost, and the weight-related portion was allocated by proportional ESALs on the secondary system. The result of this procedure was to allocate less secondary pavement maintenance expenditures on the basis of weight than were allocated on the Interstate and primary systems (Table 5).

Table 6. Secondary road cost allocation.

Item	Cost Allocation by Vehicle Class ($000 000s) 1	2	3	4
Construction				
Pavement	9.5	3.6	2.0	1.7
Site preparation and geometry	24.8	1.8	0.4	0.3
Engineering and right-of-way	14.3	0.7	0.2	0.1
Bridges	5.4	1.1	0.6	0.4
Total	54.0	7.2	3.2	2.5
Maintenance				
Pavement	19.7	12.1	7.9	5.1
Other	52.3	2.6	0.9	0.4
Total	72.0	14.7	8.8	5.5
Total	126.0	21.9	12.0	8.0

STUDY RESULTS

Thus far, the presentation of results of the cost-responsibility study has been limited to three expenditure areas. Other areas, such as bridge construction and rehabilitation, and other maintenance activities were also considered in the study. A summary of the allocation of all costs associated with secondary roads is given in Table 6. The procedures for arriving at these allocations are discussed in detail elsewhere (5,6).

The data in Table 6 show that more than half of the total cost responsibility of each truck class comes from pavement maintenance. The size of that expenditure item and its high proportion of occasioned costs explains this. Interesting, too, is the proportion of costs borne for bridges--nearly 30 percent of the bridge construction and rehabilitation costs were allocated to trucks. The effect of gross weight on bridge design is evident in this allocation.

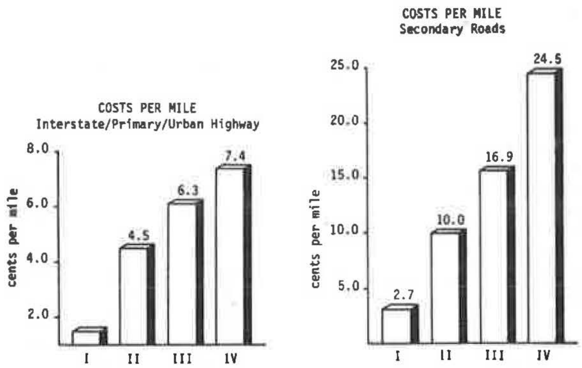
Compared with other cost-responsibility studies, the proportion of costs attributed to the individual truck classes seems incongruous. The proportions are 13.0 percent, 7.2 percent, and 4.8 percent for classes 2-4, respectively. This finding can be explained by the amount of use each of these classes contributes on secondary roads (Table 1). Secondary roads are frequently traveled by delivery trucks, dump trucks, and buses, which are included in classes 2 and 3. Class 4 vehicles travel the secondary roads much less frequently. Therefore, even though the ESALs per pass is greater for class 4 vehicles, the paucity of their presence on secondary roads minimizes their overall cost responsibility.

One further illustration lends comparative perspective on the cost responsibility for low-volume roads. The cost-per-mile responsibility for secondary roads can be compared with the cost-per-mile responsibility for Virginia's Interstate, primary, and urban roads (Figure 4). Both of the responsibilities were calculated following the methods outlined in this paper.

Table 5. Secondary road pavement maintenance allocation.

Vehicle Class	Vehicle Miles Traveled (%)	Cost of Environmental Portion ($)	ESALs (%)	Cost of Weight-Related Portion ($)	Cost Responsibility ($)	Percentage of Cost
1	93.2	19 191 745	2.1	524 572	19 716 317	43.9
2	4.6	945 376	45.9	11 142 296	12 087 672	26.9
3	1.5	313 066	31.4	7 623 292	7 936 359	17.7
4	0.7	146 235	20.6	4 995 576	5 141 811	11.5
Total		20 596 422		24 285 736	44 882 159	

Figure 4. Costs per mile traveled.



The cost-per-mile responsibility is overwhelmingly greater for secondary roads. Even with a consistent methodology, the responsibilities are at least doubled for each class on the secondary roads. Also, the cost-per-mile produces the expected relationship among the truck classes (i.e., combinations have greater responsibility than single-unit trucks). This, of course, stems from removing the impact of the miles traveled on secondary roads.

In sum, determination of the cost responsibility for secondary roads was an important part of the overall study of cost responsibility in Virginia. The findings on secondary roads showed a different distribution of cost responsibility than on the other systems, as had been expected. The combination of secondary road allocations with the other road system allocations led to the conclusion that class 2 and 3 vehicles were underpaying. Inclusion of secondary road expenditures in the state's cost-responsibility analysis was therefore a key factor in influencing study results.

ACKNOWLEDGMENT

This paper reflects our positions and opinions and should not be construed to represent the position of the Joint Legislative Audit and Review Commission of the Virginia General Assembly.

REFERENCES

1. E.J. Mosbaek. Highway Finance: Assessing User and Non-User Shares. Jack Faucett Associates, Washington, DC, March 1981.
2. J.A. Dunn, Jr. The Importance of Being Earmarked: Transport Policy and Highway Finance in Great Britain and the United States. Comparative Studies in Sociology and History, Vol. 2, Jan. 1978.
3. D. Lee. Alternative Criteria for Highway Cost Allocation. U.S. Department of Transportation, March 1980.
4. D. Lee. Recent Advances in Highway Cost Allocation Analysis. TRB, Transportation Research Record 791, 1981, pp. 1-5.
5. Joint Legislative Audit and Review Commission of the Virginia General Assembly. Methodology for a Vehicle Cost Responsibility Study. Senate Document No. 12 of 1981 Session, Richmond, Jan. 1981.
6. Joint Legislative Audit and Review Commission of the Virginia General Assembly. Vehicle Class Cost Responsibility in Virginia. Senate Document No. 13 of 1982 Session, Richmond, Nov. 1981.

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Effect of Unit-Train Grain Shipments on Rural Nebraska Roads

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The unit-train concept has altered the rural pricing structure for grains and consequently encouraged longer-distance truck transportation in larger-sized lots by producers and rural elevators over the 1975-1980 period. Annual data on grain production, livestock consumption, and storage capacity were obtained from Nebraska Agricultural Statistics. Primary data on truck receipts were collected by interview with the managers of 86 unit-train shippers across the state. A computer model was developed to calculate the total ton miles of producer transport of grains within the elevator's trade area for each district as well as ton miles of interelevator transfer. Nebraska producers in 1980 transported 71 percent more ton miles delivering grain to commercial elevators than in 1975. Combined with the growth in interelevator grain transfers by truck, the annual ton miles of rural truck transport of grains in 1980 was nearly double the 1975 level. The investment required to maintain and upgrade the rural road system is not independent of changes in other sectors of the total U.S. transportation system. The increased use of unit-trains has precipitated an increase in the ton miles of grain hauled over low-volume roads as well as an increase in the weight per axle and a subsequent increase in stress on rural roads and ridges.

From 1975 through 1980, Nebraska's annual production of grains and oilseeds averaged more than 22 million metric tons. More than 71 percent of this, or approximately 16.5 million metric tons, moved over rural roads annually via farm vehicles to commercial elevators. The growth in total volume of rural grain traffic in recent years has placed increased demand on the rural road system. Nebraska grain production increased by 7 percent annually between 1975 and 1980, more than twice the growth rate of U.S. production. With no distinguishable trend in Nebraska's feed requirements during the 1975-1980 period, increased production resulted in an average annual increase of nearly 1.5 million metric tons of grain to be marketed commercially.

Historically, bid prices to farmers have differed only marginally between competitive elevators in a given area. Consequently, producers had little incentive to seek a market beyond their nearest elevator. However, in 1975 the unit-train concept was initiated in Nebraska and offered fares significantly lower than previous single-car rates. For most elevators, adjusting to the unit-train shipment meant making a sizable investment in fixed facilities sufficient to load the 25-, 50-, and 75-car trains in 24 h.

Many shippers needed to expand their trade area to assemble the bushel volume required to support a unit-train shipping program. This was accomplished by passing a portion of the rate advantage on to producers as higher bid prices. With this incentive, producers from outlying areas found it profitable to transport grain further to unit-train facilities. Some elevators that chose not to become unit-train shippers found their grain merchandizing advantage eroded and, in turn, transferred grain by truck to the unit shippers.

The result has been an increase in rural truck transportation of grains into fewer elevators that then ship directly to the export market and bypass the traditional terminals. As producers moved grain greater distances, it became profitable to increase truck size. The net effect has been to increase the ton miles hauled over low-volume roads as well as to increase the weight per axle with a subsequent increase in stress on rural roads and bridges.

OBJECTIVES

The purpose of this study was to estimate the magnitude of such changes for major grain-producing regions of the state over the 1975-1980 period. Specific objectives were to (a) describe the changes between 1975 and 1980 in variables that influence the rural transportation system such as the truck transportation costs, the number of single-car and unit-train elevators, and the annual volume of marketed Nebraska grain; (b) compare the producer ton miles of grain transportation by crop reporting district for 1975 and 1980; and (c) estimate the increase in ton miles of interelevator grain transfer by truck by crop reporting district from 1975 to 1980.

Methodology

The total ton miles of grain transportation required to serve the originating elevator depends on several variables. The quantity of nonfeed grain per square mile and the number and size of elevators within the crop reporting district are important considerations. The amount and distance of interelevator grain transfers by truck identify additional transportation necessary to position the grain for shipment out of the area. Secondary data were obtained for grain production, livestock numbers, and feed requirements as well as elevator size and numbers. Primary data were collected on elevator bid prices, interelevator grain transfers, and trucking costs.

A computer program was developed to calculate the difference in ton miles of rural truck transportation between 1975 and 1980. Equation 1 is the identity equation used to compare the two years of transportation needs in district K.

$$\Delta M_K = M_{K_{80}} - M_{K_{75}} \qquad (1)$$

where M_K is ton miles of rural truck transport in district K.

Estimating Rural Truck Transport in 1975

The total ton miles of rural truck transport in 1975 ($M_{K_{75}}$, Equation 1) was attributed to producer transport since grain transfers from country elevators to terminals would probably not occur over rural roads and the country-terminal traffic was not included in either the 1975 or 1980 calculation. Equation 2 computes the ton miles of producer transport in 1975.

$$M_{K_{75}} = \sum_{J=1}^{J} \sum_{R=1}^{R} [(T_{J,K,R} - T_{J,K,R-1})\, 1.207 D_K R_M] \qquad (2)$$

where

$T_{J,K,R} = (B_{J,K} / \sum_{J=1}^{J} B_{J,K})\ (A_K)$, the square mileage trade area of elevator J in district K with a radius of R in 1975;
A_K = total square mileage area of district K;
$B_{J,K}$ = licensed storage capacity of elevator J in district K;
D_K = density (metric tons per square mile) of marketed grain; and
$R_M = \sqrt{R^2 - R + 0.5}$, the radius to the midpoint of the trade area delineated by R and R - 1.

According to $B_{J,K}$, the total square mileage of the crop-reporting district was allocated among the elevators in proportion to their licensed grain storage capacity, thereby determining the trade areas for individual elevators. It was assumed that elevators that possess a large percentage of the district's total licensed storage capacity served a proportionately large trade area in 1975.

A circular configuration was imposed on each individual trade area even though it is recognized that this implies an equal amount of overlapped and nontraded area. Although other classical location models have assumed hexagonal or other polygonal patterns (1), the circular form allows trade areas to be of unequal size, which was crucial to the present study. Once an individual firm's trade area was determined, the elevator was assumed to be located at the central origin of a rectangular grid road system that extends over the entire area. Beginning with the outer rim portion of the trade area ($T_{J,K,R} - T_{J,K,R-1}$) the ton miles generated from each 1-mile-wide concentric area were summed for all radii R inwardly toward the epicenter. An average correction factor of 1.207 was calculated by using the Pythagorean theorem to convert direct radius distances into road distances to account for marketed grain originating off the main X or Y axis of the grid network. Marketed grain is defined as the annual production of all grains and oilseeds in a given district minus the annual livestock feed requirements in that district. The ton miles of transport were then summed for all firms J in district K.

Estimating Rural Truck Transport in 1980

By 1980, licensed storage capacity was not a relevant criteria for estimating elevator trade areas for the unit-train shippers. The volume of grain handled by single-car shippers was calculated by using estimated annual turnover rates on storage capacity. Because local livestock feed requirements were already deducted in determining marketable grain, the turnover rates of single-car elevators did not include the volume of grain handled by elevators to satisfy local feeders. Based on discussions with industry sources, the turnover rate of 1.0 times the total licensed storage capacity was applied against grains marketed through the single-car shipper into the interregional commerce.

The annual volume of direct producer deliveries of grain to unit shippers in 1980 was estimated to consist of the total marketed grain for that district minus the estimated grain receipts of single-car shippers. This remaining grain was distributed between the unit-train elevators in proportion to their car-loading capacity.

Equation 3 estimates the total ton miles of rural truck transport of grain in 1980. Its three major components are estimates of producer transport to single-car elevators, producer transport to unit-train elevators, and interelevator truck transport.

$$M_{K_{80}} = \sum_{N=1}^{N} \sum_{R=1}^{R} [(T_{N,K,R} - T_{N,K,R-1})\, 1.207\, D_K R_M]$$

(producer transport to single-car elevators)

$$+ \sum_{U=1}^{U} \sum_{R=1}^{R} [(T_{U,K,R} - T_{U,K,R-1})\, 1.207\, D_K R_M]$$

(producer transport to unit-train elevators)

$$+ 1.9375\, T_{U,K,R}\, D_K \qquad (3)$$

(interelevator transfer)

where

$T_{N,K,R} = B_{N,K}/D_K$, the square mileage trade area of single-car elevator N in district K with radius R in 1980;

$T_{U,K,R} = (L_{U,K}/\sum_{U=1}^{U} L_{U,K})\,(A_K - \sum_{N=1}^{N} T_{N,K,R})$, the square mileage trade area of unit-train elevator U in district K with radius R; and

$L_{U,K}$ = load-out capacity of unit-train elevator U in district K, specified as the number of hopper cars capable of being spotted on the elevator's siding.

The total ton miles of producer transport in 1980 within a given elevator's trade area was calculated in a similar manner as the 1975 computations. The amount of interelevator transfer was estimated as a percentage of total unit-elevator receipts, based on primary data collected from the operators of all 66 unit-train elevators in 1980.

RESULTS

The total quantity of marketed grain in 1980 was nearly 2.5 million metric tons or 19 percent larger than in 1975 according to Table 1. This growth was accomplished in spite of the fact that livestock feed requirements in 1980 were significantly above the 1975-1980 trend and a widespread drought reduced 1980 production significantly below the trend. Districts such as the Southeast and East experienced growth in livestock feed requirements that nearly offset increased grain production. Consequently, their volume of 1980 marketed grain was only 6 and 8 percent, respectively, larger than in 1975. In contrast, grain marketed over rural roads in the Central district increased by 25 percent, in the Northeast by 27 percent, in the Southeast by 31 percent, and in the South by 44 percent over the same period. Even if the average length of haul did not change from 1975 to 1980, one could expect ton miles of rural producer transport of Nebraska grains to increase approximately 19 percent due only to the greater volume of grain being marketed.

The total number of receiving elevators has remained relatively constant with less than 1 percent decline over the five years. However, the real change (as Figure 1 illustrates) has been in the upgrading of single-car elevators into unit-train facilities along with a small increase in newly constructed unit elevators. The number of single-car elevators declined by nearly 11 percent due to up-

Table 1. Changes in quantity of Nebraska's marketed grain, 1975-1980.

	Grain Production		Feed Requirements		Commodity Surplus	
Crop Reporting District	Absolute Increase (000s metric ton)	Increase (%)	Absolute Increase (000s metric ton)	Increase (%)	Absolute Increase (000s metric ton)	Increase (%)
Northeast	483	18	100	8	383	27
Central	333	15	-72	-12	405	25
East	547	11	231	24	316	8
Southwest	475	27	24	8	451	31
South	810	39	28	10	782	44
Southeast	307	10	166	27	141	6
State total[a]	2955	17	477	12	2478	19

[a]State total does not include Northwest and North districts.

Figure 1. Changes in Nebraska's grain elevators: 1975-1980.

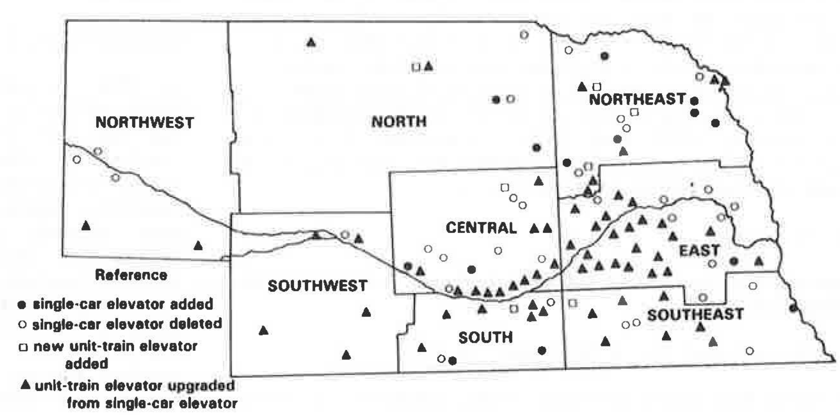

Table 2. Change in ton miles of rural Nebraska truck transportation of grains, 1975-1980.

Region	Producer to Elevator: Nonunit Shipper (000s ton mile)	Unit Shipper (000s ton mile)	Increase: Ton Miles (000s)	Increase: Percent	Interelevator Nonunit to Unit Transfer (000s ton mile)	Total Increase: Ton Miles (000s)	Total Increase: Percent
Northeast	-5 121	+16 214	+11 092	97	+2 940	+14 033	123
Central	-4 141	+12 882	+8 471	83	+3 215	+11 957	113
East	-10 244	+12 337	+2 093	9	+5 393	+7 486	32
Southwest	+223	+20 280	+20 503	165	+2 727	+23 230	189
South	-6 538	+14 229	+7 691	61	+3 878	+11 569	93
Southeast	-682	+8 797	+8 115	71	+2 424	+10 538	92
State total[a]	-26 503	+84 738	+58 235	71	+20 576	+70 812	96

[a] State total does not include Northwest and North Regions; assumes 1.0 turnovers per year on licensed storage capacity for nonunit shippers not including grain for local livestock feeding.

grading as well as closures. The number of rural train-load shippers increased by 66, not including the 19 terminal facilities.

The pricing structure has changed as well. Weekly cash bid prices collected over the last two years from six privately owned single-car and unit-train shippers in central Nebraska were compared. It was found that the prices offered to producers by unit-train shippers were approximately 8 cents/bushel higher than at nearby single-car elevators. Given this differential bid price between elevators, the extent to which the trade areas of unit-train elevators would likely expand is determined by the producer's cost of rural transport. Payne, Baumel, and Moser (2) calculated variable costs of trucking by using a 16-ft, 325-bushel grain box at approximately 0.06 cents/bushel per mile (1975 dollars), not including dead-haul labor costs. The pricing of identical items (2) by using 1980 prices revealed a variable cost per bushel per mile of about 0.16 cents for the 325-bushel load. This would indicate that it could be economically rational for a producer to increase a haul by more than 6 miles for a 1 cent differential in bid price. In addition, more producers are now using 18-ft, 425-bushel grain boxes for which the variable costs are estimated to be only 0.13 cents/bushel per mile.

Of additional importance is the seasonal difference in the demand for rural transport. Primary research of producer marketing practices in Nebraska in 1977 revealed that more than 72 percent of producer deliveries of wheat to local elevators occurred in June and July. Likewise, 50 percent of the 1977 corn and 75 percent of the milo harvest was delivered to commercial centers for sale or storage during the October-November period (3). Not only does the demand on the system appear extremely concentrated but maintenance of rural road conditions during the wet snowy late fall conditions requires a different investment than during the dry wheat harvest of early July. The high concentration of corn, sorghum, and soybean production in the Central, South, Southeast, and Northeast crop-reporting districts indicates that nearly half of all rural grain transport in those areas will occur in the late fall.

The combined impact of increases in marketed grain and the greater differential in bid prices relative to the costs of transport has meant more bushels that travel farther over rural roads. According to Table 2, the total ton miles transported by producers for all major grain districts was 71 percent greater in 1980 than in 1975. However, the increase in demand on rural roads was by no means consistent between districts. In the East, where the marketed grain increased only marginally in 1980 and a considerable number of single-car elevators were upgraded to unit-train capacity, total ton miles increased by 9 percent in five years. That growth occurred as deliveries to unit-train facilities more than offset an overall decrease in ton mileage incurred for delivery to nonunit elevators. On the high side, the Southwest district experienced a 165 percent increase in producer ton miles, primarily because of increases in marketed grain and a modest growth in unit-train facilities.

In addition to increased producer transportation, Table 2 indicates an increase in interelevator truck transfer from single-car to nearby unit-train elevators. Based on primary data collected from the 66 unit-train elevators in 1981, more than 6 percent of their total volume was received by truck from single-car shippers located an average of 31 miles away. Consequently, Nebraska's road system supported nearly 17 million additional ton miles of truck transportation due to interelevator transfers, from single-car to unit-train facilities.

In summary, Table 2 indicates that total ton miles of truck transport of grains over non-Interstate rural roads has nearly doubled over the 1975-1980 period. Nearly three-quarters of this increase has been due to changes in producer transportation patterns.

SUMMARY AND CONCLUSION

The investment required to maintain and upgrade the rural road system is not independent of change in other sectors of the total U.S. transportation industry. The introduction of the unit-train concept for on-rail transportation in the Great Plains has altered the rural pricing structure for grains and consequently encouraged longer distant truck transport of grains by producers in major production regions.

This study has indicated that the 19 percent growth rate over 1975-1980 in Nebraska's marketed grain has combined with longer distant hauls to increase total ton miles of rural truck transport at a rapid pace. It was found that, in 1980, Nebraska producers transported 71 percent more ton miles in delivering grain to commercial elevators than in 1975. Combined with the growth in interelevator grain transfers by truck, the annual ton miles of rural truck transport of grains in 1980 was nearly double the 1975 level.

Recognition of the interdependencies between transportation modes and the economies of unit-train transport will assist in anticipation of the demands placed on rural roads. A better understanding of the amount and type of demands on the rural road network will assist in making long-run public investments.

ACKNOWLEDGMENT

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REFERENCES

1. B.C. French. The Analysis of Productive Efficiency in Agricultural Marketing: Models, Methods, and Progress. A Survey of Agricultural Economics Literature, Vol. 1 (L. Martin, ed.), Univ. of Minnesota Press, Minneapolis, 1977.
2. D. Linsenmeyer. Analyses of Nebraska Wheat Flows. Department of Agricultural Economics, Univ. of Nebraska, Lincoln, 1981.
3. W.F. Payne, C.P. Baumel, and D. Moser. Estimating Truck Transport Costs for Grain and Fertilizer. College of Agriculture, Univ. of Missouri, Columbia, Res. Bull. 1027, June 1978.

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Development of a Computerized Technique to Identify Effective Forest Roadway Networks

DAVID C. SHUNK AND ROBERT D. LAYTON

Forest transportation planning is a complex task that involves many decisions. This paper presents an algorithm and computer program that will assist in effective transportation planning and decisionmaking in the national forests. This identification of an efficient arterial, collector, and local roadway network is a primary component in the transportation planning process. An earlier study by Kehr and Layton identified the primary factors and important decision criteria used to evaluate forest arterial and collector networks. This study employs these decision criteria in the development of a computerized comprehensive analytical framework, PLANET1 and PLANET2, to identify and evaluate forest arterial and collector networks. Two main computerized network algorithms have been used in transportation network evaluation: the shortest path algorithm and the minimum spanning tree algorithm. The shortest path algorithm provides the most direct route to each point, without direct consideration of construction costs. The minimum spanning tree algorithm emphasizes the least-cost connective network and ignores the travel times and operating costs. The computerized technique presented in this report combines the advantages of the shortest path algorithm with those of the minimum spanning tree algorithm to determine a more efficient roadway network than is provided by either approach used individually. Examples of the use of these new algorithms, PLANET1 and PLANET2, are presented and discussed.

As defined by the Forest Service Manual, the objective of transportation planning is (1) "to ensure that plans for the development and operation of the forest development transportation system are made, and that they are consistent with land-use planning policies and procedures, and will effectively achieve resource management objectives".

The identification of an efficient arterial, collector, and local roadway network is a primary component of the transportation planning process. A previous study by Kehr and Layton (2) identified the primary factors and important decision criteria used to evaluate forest arterial and collector networks. The study employs these decision criteria in the developing of a comprehensive analytical framework to identify and evaluate forest arterial and collector networks. The method developed in the report by Kehr and Layton, however, is a manual method that takes a great deal of time to use.

Two computerized network algorithms used extensively in forest transportation network analysis and evaluation are the shortest path algorithm and the minimum spanning tree algorithm. The shortest path algorithm provides the most direct route to each point, measured by time, distance, or cost, usually operating cost. The minimum spanning tree algorithm provides the least-cost connective network, which is usually measured by the length of links or link costs, typically construction and maintenance costs.

The method developed by Kehr and Layton (2) recommends the use of a method that employs both the shortest path and the minimum spanning tree algorithms. However, no computerized technique is presented in that study. A primary analysis technique for national forest planning is the timber transport model (TIMBRI), a computerized method to find the least-cost timber haul routes (3). This technique relies heavily on the shortest path algorithm together with a mixed integer linear programming routine. However, that technique focuses on identifying the most efficient network for timber haul alone. The PLANET1 and PLANET2 algorithms combine the advantages of a shortest path analysis, which minimizes time or operating cost, and the minimum spanning tree analysis, which minimizes construction costs and, if desired, maintenance costs, to determine an efficient roadway network.

SCOPE

This paper presents a computerized method to identify effective forest roadway networks. The decision criteria used to evaluate forest road networks are divided into four major groups: physical, analytical, quantitative, and qualitative. The important decision criteria in each group are given below in rank order.

1. Physical criteria--connection to regional mills and markets, connectivity with surrounding road networks, types of vehicles and users present, extent of access to forest area, interface conflicts and delays, and access to adjacent lands;
2. Analytical criteria--total cost for timber haul, timber traffic volume, recreational traffic volume, least-cost connective network, construction cost, operating cost, maintenance cost, safety cost, level of service, and capacity;
3. Quantitative criteria--size of area served, speed of travel, and road design standards; and
4. Qualitative criteria--compatibility with environment, comfort and convenience, and safety.

The computerized methods described in this report incorporate some of the physical, analytical, and quantitative decision criteria, but not the qualitative decision criteria. The specific criteria taken into account, either directly or indirectly, by these methods are as follows:

1. Speed of travel,
2. Least-cost connective network,
3. Operating cost,
4. Maintenance cost,
5. Construction cost,
6. Safety cost,
7. Timber traffic volume,
8. Recreational traffic volume, and
9. Road design standards.

Forest transportation planning is a complex task that involves many decisions. The PLANET1 and PLANET2 programs assist the decisionmaker in making effective transportation planning decisions. This report presents PLANET1 algorithm and its development and briefly addresses the capabilities of PLANET2.

MODEL DEVELOPMENT

The goal of PLANET1 is to provide a computerized technique to find a roadway network that combines the advantages of the shortest path tree (more direct routes) and the minimum spanning tree (lower construction costs).

The shortest path algorithm can be formulated as a linear programming problem as follows:

$$\text{Minimize } Z' = \sum_{I=1}^{\text{Number of centroids}} \text{Centroid I travel time} \quad (1)$$

Such that,

Flow into each node = flow out of the node,
Flow at centroids = 1, and
Flow out at origin = - number of centroids.

where centroids are nodes that have some traffic-generating activity present, referred to as activity nodes in this paper.

This can be written for the network in Figure 1 as

$$\text{Minimize } Z' = 6\phi_{12} + 4\phi_{13} + 6\phi_{21} + 1\phi_{23} + 5\phi_{24} + 4\phi_{31} + 1\phi_{32} + 7\phi_{34} + 5\phi_{42} + 7\phi_{43}$$

Such that,

$$Y_1 + \phi_{21} + \phi_{31} = \phi_{12} + \phi_{13}$$
$$Y_2 + \phi_{12} + \phi_{32} + \phi_{42} = \phi_{21} + \phi_{23} + \phi_{24}$$
$$Y_3 + \phi_{13} + \phi_{23} + \phi_{43} = \phi_{31} + \phi_{32} + \phi_{34}$$
$$Y_4 + \phi_{24} + \phi_{34} = \phi_{42} + \phi_{43}$$
$$Y_1 = -3$$
$$Y_3 = 1$$
$$Y_4 = 1$$

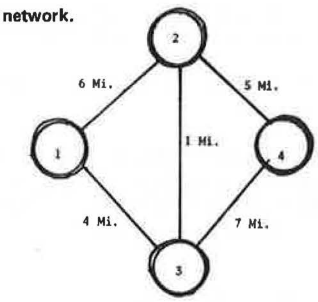
where

All $\phi \geq 0$,
ϕ_{xy} = flow from x to y, and
Y_I = flow added at node x.

The minimum spanning tree algorithm can be formulated as a zero-one mixed integer problem as follows,

$$\text{Minimize } Z'' = \sum_{I=1}^{\text{Number of Links}} \begin{bmatrix} Z_I = 1 \text{ if link} \\ \text{is used} \\ Z_I = 0 \text{ if not} \end{bmatrix} \begin{bmatrix} \text{Link I} \\ \text{cost} \end{bmatrix} \quad (2)$$

Figure 1. Example network.



Such that,

Flow into each node = flow out of the node,
Flow in at centroid = 1,
Flow out at origin = - number of centroids, and
Flow x→y + flow x←y $\leq$ (a large number) Z_I.

This can be written for the problem in Figure 1 as

$$\text{Minimize } Z'' = 6Z_1 + 4Z_2 + 1Z_3 + 5Z_4 + 7Z_5$$

Such that,

$$Y_1 + \phi_{21} + \phi_{31} = \phi_{12} + \phi_{13}$$
$$Y_2 + \phi_{12} + \phi_{32} + \phi_{42} = \phi_{21} + \phi_{23} + \phi_{24}$$
$$Y_3 + \phi_{13} + \phi_{23} + \phi_{43} = \phi_{31} + \phi_{32} + \phi_{34}$$
$$Y_4 + \phi_{24} + \phi_{34} = \phi_{42} + \phi_{43}$$
$$Y_1 = -3$$
$$Y_2 = 1$$
$$Y_3 = 1$$
$$Y_4 = 1$$
$$\phi_{12} + \phi_{21} \leq MZ_1$$
$$\phi_{13} + \phi_{31} \leq MZ_2$$
$$\phi_{23} + \phi_{32} \leq MZ_3$$
$$\phi_{24} + \phi_{42} \leq MZ_4$$
$$\phi_{34} + \phi_{43} \leq MZ_5$$

where

All $\phi_{xy} \geq 0$,
All Z_I = 0 or 1,
ϕ_{xy} = flow from x to y,
Y_x = flow added at node x, and
M = extremely large number.

The shortest path algorithm and the minimum spanning tree algorithm, as illustrated above, can be combined by using a trade-off factor. In both cases the mileage between the nodes is considered to be proportional to the travel cost for the shortest path algorithm and the construction cost for the minimum spanning tree. Since the mileage is used to represent both the travel and construction costs, it is necessary to add a trade-off factor, which is a ratio of the construction cost to the travel cost, when comparing the two. The trade-off factor assumes a uniform demand at each centroid and can be written as

$$\text{Trade-off factor} = [\text{Construction cost (\$/mile)}] / [\text{Uniform demand (vehicle)} \times \text{travel cost (\$/vehicle mile)}] \quad (3)$$

The ideal PLANET1 objective function can be formulated as a zero-one mixed-integer program as follows:

$$\text{Minimize: } Z''' = Z' + \text{trade-off factor } (Z'') \quad (4)$$

Such that,

Flow into each node = flow out of the node,
Flow in at centroids = 1,

Flow in at the origin = - number of centroids, and
Flow $x \rightarrow y$ + flow $x \leftarrow y \leq$ (a large number) Z_I.

This can be written for the network in Figure 1 as,

$$\text{Minimize } Z''' = (6\phi_{12} + 4\phi_{13} + 6\phi_{21} + 1\phi_{23} + 5\phi_{24} + 4\phi_{31} + 1\phi_{32} + 7\phi_{34} + 5\phi_{42} + 7\phi_{43}) + \text{trade-off factor } (6Z_1 + 4Z_2 + 1Z_3 + 5Z_4 + 7Z_5)$$

Such that,

$$Y_1 + \phi_{21} + \phi_{31} = \phi_{12} + \phi_{13}$$
$$Y_2 + \phi_{12} + \phi_{32} + \phi_{42} = \phi_{21} + \phi_{23} + \phi_{24}$$
$$Y_3 + \phi_{13} + \phi_{23} + \phi_{43} = \phi_{31} + \phi_{32} + \phi_{34}$$
$$Y_4 + \phi_{24} + \phi_{34} = \phi_{42} + \phi_{43}$$
$$Y_1 = -3$$
$$Y_2 = 1$$
$$Y_3 = 1$$
$$Y_4 = 1$$
$$\phi_{12} + \phi_{21} \leq MZ_1$$
$$\phi_{13} + \phi_{31} \leq MZ_2$$
$$\phi_{23} + \phi_{32} \leq MZ_3$$
$$\phi_{24} + \phi_{42} \leq MZ_4$$
$$\phi_{34} + \phi_{43} \leq MZ_5$$

where

All $\phi_{xy} \geq 0$,
All Z_I = 0 or 1,
ϕ_{xy} = flow from x to y,
Y_x = flow added at node x, and
M = extremely large number.

This problem can be solved by using an integer programming technique. However, for large networks this would require excessive computational time to reach the optimum solution. The PLANET1 assumptions allow it to find a good solution, not necessarily the optimum, in a very short time.

The PLANET1 algorithm starts with the shortest path tree for the network. A tree, as used here, is a connected graph that contains no loops. It analyzes every unused link to determine if it should be added to the tree. This will be referred to as the candidate link. If the candidate link is added, one of the links already in the tree must be removed. The two decision criteria used are the savings in travel cost to the centroids (DTIME) created by using the link to be added and the increase in construction cost (DCOST) created by using the candidate link instead of another link. The unused link is added to the tree if

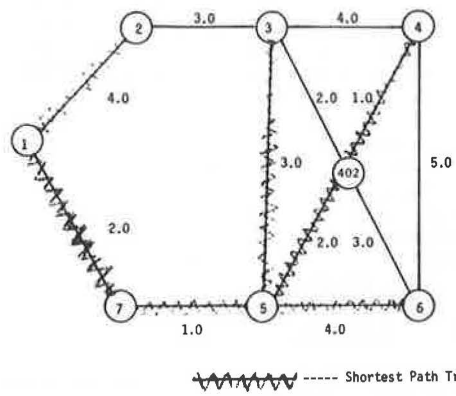
$$\text{DTIME} > \text{Trade-off factor} \times \text{DCOST} \tag{5}$$

Otherwise, it is not added and the next unused link is analyzed. As pairs of links are added and removed, PLANET1 moves closer to the optimum solution. Through this process the PLANET1 algorithm attempts to imitate the process that occurs in an integer programming approach. The PLANET1 algorithm does not guarantee the optimum solution, but it gives a good solution in a very short computation time and comes much closer to the optimum network than does the shortest path algorithm or the minimum spanning tree algorithm.

PLANET1 Algorithm

An algorithm is defined as a systematic set of mathematical steps for solving a problem. The PLANET1 algorithm combines the advantages of the shortest path tree and the minimum spanning tree in determining the most efficient network possible. The steps in the PLANET1 algorithm are as follows:

Figure 2. Example network for PLANET1.



Step 1. Find the shortest path tree from the primary activity centers by using the shortest path algorithm.

Step 2. Consider adding each link in the link file, in turn. This link will be referred to as the candidate link. Start with the first link.

Step 3. If the candidate link is already part of the tree, go to step 10.

Step 4. Consider removing the next link down the tree. Start with the predecessor link of the origin node of the candidate link. For example, in Figure 2 if link 3-2 were the candidate link, link 3-5 would be considered for removal, then link 5-7, etc.

Step 5. If removing this link creates a disconnected graph, go to step 10. A disconnected graph, as used here, is a network where all desired nodes cannot be reached from the origin node.

Step 6. Calculate the increase in link cost (DCOST) created by using the candidate link instead of the link being removed.

Step 7. Calculate the savings in travel times (DTIME) to the centroids created by using the candidate link instead of the link being removed.

Step 8. If Equation 5 is true, add the candidate link to the tree, remove the old link, and go back to step 2. Otherwise go on to step 9.

Step 9. If the destination node of the link to be removed is not the origin, go back to step 4. If it is the origin, go on to step 10.

Step 10. If the candidate link is not the last link in the link file, go back to step 2. If it is the last link, go on to step 11.

Step 11. If no changes have been made on this pass through the link file, terminate the algorithm. Otherwise, go back to step 2 and start through the link file again.

Example Illustrating the PLANET1 Algorithm

The following is an example that illustrates the use of the PLANET1 algorithm. Table 1 gives the steps involved in PLANET1 for the example network. Figure 3 shows the network and the shortest path tree, and Table 2 gives the initial link file to be used in this example. Figures 4 and 5 and Tables 3 and 4 show the steps in the algorithm in the first iteration as links are added and deleted in the network.

The KOUNT array used in this example indicates if a link is in the tree (KOUNT = 1) or not (KOUNT = 0). The column labeled TRADE? indicates whether the candidate link is traded for the link being considered for removal (LINK OUT). Notice that the trade-off factor is equal to 2.0. The number next to the link arrows in Figures 3-8 indicates the number of the link in that direction. The two numbers

Table 1. PLANET1 graphical example, iterations 1 and 2.

Candidate Link	Link Out	DTIME	Factor	DCOST	Trade	Remarks
Iteration 1						
4	3	-5.0	2.0	-1.0	No	Use Figure 3, Table 2
5	7	-1.0	2.0	0.0	No	Use Figure 3, Table 2
5	14	-22.0	2.0	2.0	No	Use Figure 3, Table 2
5	19	-22.0	2.0	1.0	No	Use Figure 3, Table 2
6	7	-4.0	2.0	1.0	No	Use Figure 3, Table 2
8	7	-1.0	2.0	-1.0	Yes	Use Figure 4, Table 3
9	11	-5.0	2.0	3.0	No	Use Figure 4, Table 3
10	11	-6.0	2.0	4.0	No	Use Figure 4, Table 3
10	23	-14.0	2.0	3.0	No	Use Figure 4, Table 3
12	-		No possible removals			
16	17	-4.0	2.0	1.0	No	Use Figure 4, Table 3
18	17	-1.0	2.0	-1.0	Yes	Use Figure 5, Table 4
Iteration 2						
4	3	-6.0	2.0	-1.0	No	Use Figure 5, Table 4
5	8	0.0	2.0	1.0	No	Use Figure 5, Table 4
5	23	-8.0	2.0	1.0	No	Use Figure 5, Table 4
5	14	-16.0	2.0	2.0	No	Use Figure 5, Table 4
5	19	-16.0	2.0	1.0	No	Use Figure 5, Table 4
6	8	-3.0	2.0	2.0	No	Use Figure 5, Table 4
7	8	1.0	2.0	1.0	No	Use Figure 5, Table 4
7	23	-5.0	2.0	1.0	No	Use Figure 5, Table 4
9	11	-5.0	2.0	3.0	No	Use Figure 5, Table 4
10	11	-7.0	2.0	4.0	No	Use Figure 5, Table 4
12	-		No possible removals			
13	-		No possible removals			
16	18	-3.0	2.0	2.0	No	Use Figure 5, Table 4
17	18	1.0	2.0	1.0	No	Use Figure 5, Table 4
17	23	-9.0	2.0	2.0	No	Use Figure 5, Table 4

Note: Since no changes were made in iteration 2, the network in Figure 11 and Table 5 is the solution.

Figure 3. Initial network for PLANET1 graphical example.

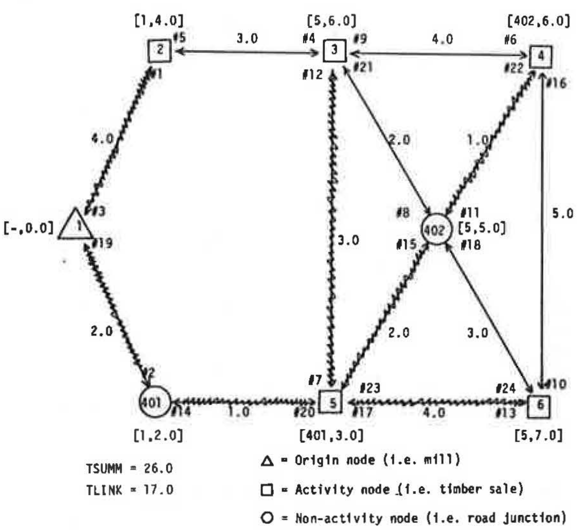


Table 2. Initial link file for PLANET1 graphical example.

Link No. (X)	Origin Node [NODE(X,1)]	Destination Node [NODE(X,2)]	Link Criteria [CRIT(X,1)]	KOUNT (X)
1	1	2	4.0	1
2	1	401	2.0	1
3	2	1	4.0	1
4	2	3	3.0	0
5	3	2	3.0	0
6	3	4	4.0	0
7	3	5	3.0	1
8	3	402	2.0	0
9	4	3	4.0	0
10	4	6	5.0	0
11	4	402	1.0	1
12	5	3	3.0	1
13	5	6	4.0	1
14	5	401	1.0	1
15	5	402	2.0	1
16	6	4	5.0	0
17	6	5	4.0	1
18	6	402	3.0	0
19	401	1	2.0	1
20	401	5	1.0	1
21	402	3	2.0	0
22	402	4	1.0	1
23	402	5	2.0	1
24	402	6	3.0	0

in brackets next to each node are the predecessor node number and the travel time from that node to the origin, [predecessor node number, travel time].

Figures 5-8, then, show the results for this network from PLANET1, BUILDER, MINTREE, and MINSPAN, respectively. BUILDER builds shortest path trees; MINTREE builds a network that contains the least-cost connective network from the origin activity node to all activity nodes; MINSPAN builds the least-cost connective network from the origin activity node to all nodes. The range for TSUMM, the sum of the link criteria from the origin to each activity node, is from 26.0 (BUILDER) to 34.0 (MINSPAN). Notice that TSUMM for PLANET1 (= 28.0) is close to the minimum. The range for TLINK, the total link criteria for the link in the tree, is from 14.0 (MINSPAN) to 17.0 (BUILDER). Notice that TLINK for PLANET1 (= 15.0) is close to the minimum.

MODEL APPLICATION

Figure 9 shows an example forest network. The lines that connect the three node ones (i.e.; 1, 1', and 1") represent the main arterial roadway. PLANET1 is used to find the best roadway network to connect the forest activities with the main arterial. All nodes on the arterial are indexed as node 1. Note that the network is not drawn to scale.

Figures 10, 11, and 12 show the results obtained from the shortest path tree algorithm (program BUILDER), the minimum spanning tree algorithm (program MINSPAN), and the minimum spanning tree algorithm (program MINTREE), respectively. MINTREE builds the minimum spanning tree from activity node to activity node without concern for nodes that rep-

Figure 4. PLANET1 graphical example: entering links 8 and 21.

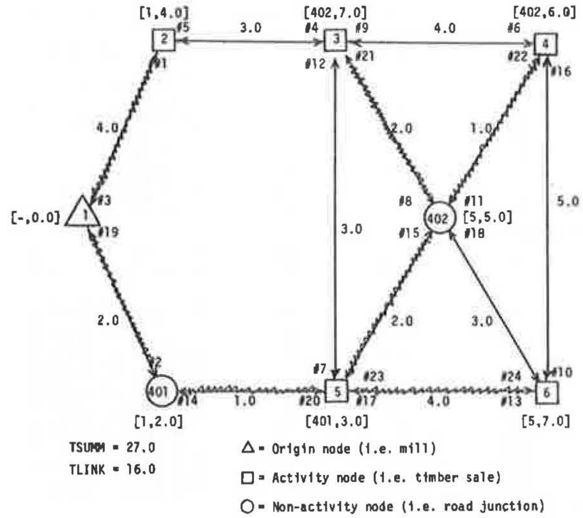


Figure 5. PLANET1 graphical example: entering links 18 and 24.

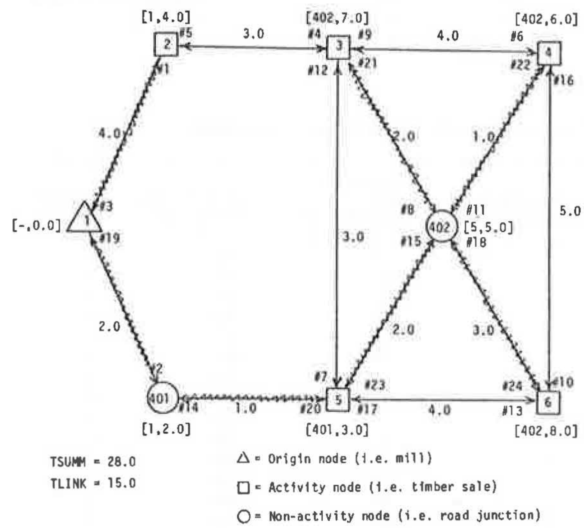


Table 3. Link file for PLANET1 graphical example: entering links 8 and 21 (shown as Figure 4).

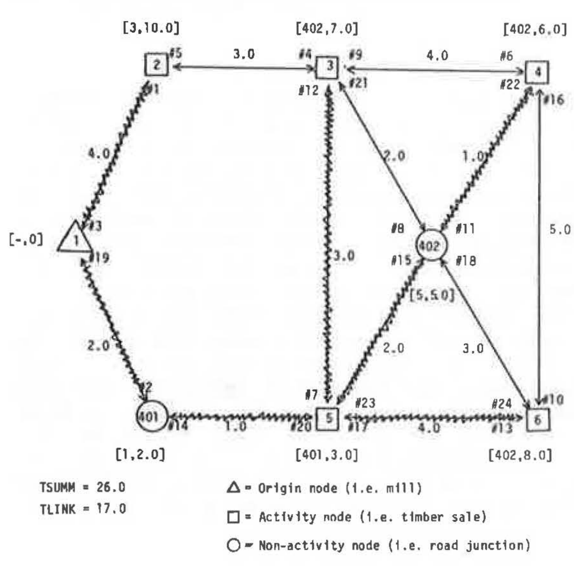
Link No. (X)	Origin Node [NODE(X,1)]	Destination Node [NODE(X,2)]	Link Criteria [CRIT(X,1)]	KOUNT (X)
1	1	2	4.0	1
2	1	401	2.0	1
3	2	1	4.0	1
4	2	3	3.0	0
5	3	2	3.0	0
6	3	4	4.0	0
7	3	5	3.0	0
8	3	402	2.0	1
9	4	3	4.0	0
10	4	6	5.0	0
11	4	402	1.0	1
12	5	3	3.0	0
13	5	6	4.0	1
14	5	401	1.0	1
15	5	402	2.0	1
16	6	4	5.0	0
17	6	5	4.0	1
18	6	402	3.0	0
19	401	1	2.0	1
20	401	5	1.0	1
21	402	3	2.0	1
22	402	4	1.0	1
23	402	5	2.0	1
24	402	6	3.0	0

resent network intersections and nodes where road class changes. Figure 13 shows the results obtained from PLANET1 with a trade-off factor of 5.0. The TSUMM variable is accumulated travel time from the origin to each of the activity nodes. This represents the total travel cost for the network. The variable TLINK is the sum of the link measure used in the network tree, this represents the total construction cost required to provide this network. Table 5 gives the results obtained from the four programs. The travel times to each activity node from node 1 are listed for the four programs. Table 6 indicates how the accumulated times to activity nodes (TSUMM) and accumulated link costs (TLINK) vary. As the trade-off factor varies from 0 to 9999 for PLANET1, the tree identified changes in character. With a trade-off factor of 0, the PLANET1 network is the shortest path tree. With a high trade-off factor, the PLANET1 network tends toward the minimum spanning tree.

Table 4. Link file for PLANET1 graphical example: entering links 18 and 24 (shown as Figure 5).

Link No. (X)	Origin Node [NODE(X,1)]	Destination Node [NODE(X, 2)]	Link Criteria [CRIT(X,1)]	KOUNT (X)
1	1	2	4.0	1
2	1	401	2.0	1
3	2	1	4.0	1
4	2	3	3.0	0
5	3	2	3.0	0
6	3	4	4.0	0
7	3	5	3.0	0
8	3	402	2.0	1
9	4	3	4.0	0
10	4	6	5.0	0
11	4	402	1.0	1
12	5	3	3.0	0
13	5	6	4.0	0
14	5	401	1.0	1
15	5	402	2.0	1
16	6	4	5.0	0
17	6	5	4.0	0
18	6	402	3.0	1
19	401	1	2.0	1
20	401	5	1.0	1
21	402	3	2.0	1
22	402	4	1.0	1
23	402	5	2.0	1
24	402	6	3.0	1

Figure 6. Shortest path tree (BUILDER) for example network.



PLANET1 can be used for roadway systems where information on the construction, maintenance, and operating costs is not available. Program PLANET2 is an extension of the concepts of PLANET1 and was developed to analyze roadway systems where the cost information is available. Some of the additional features of PLANET2 are as follows:

1. The actual construction, maintenance, and operating costs can be used.
2. Different road classes such as arterial, collector, and local are allowed.
3. The actual demands for the individual activities can be used.

Figure 14 shows an example network obtained from PLANET2. In Figure 8 circles are used to represent activities. The area of the circle reflects the relative magnitude of the activity level.

CONCLUSIONS

PLANET1 and PLANET2 systematically combine the advantages of the shortest path algorithm and the min-

Figure 7. Minimum spanning tree (MINTREE) for example network.

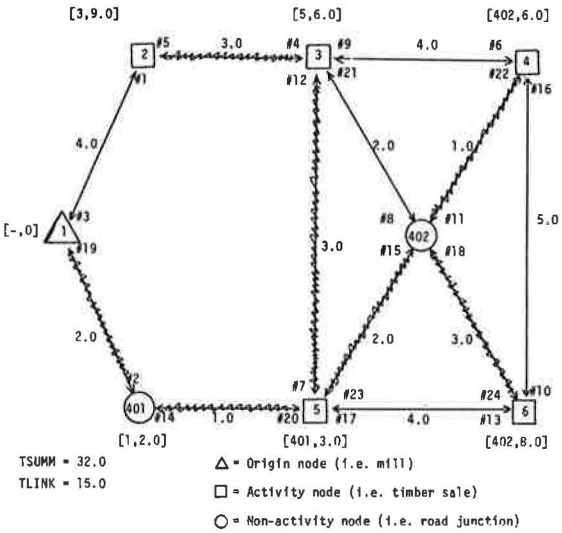


Figure 8. Minimum spanning tree (MINSPAN) for example network.

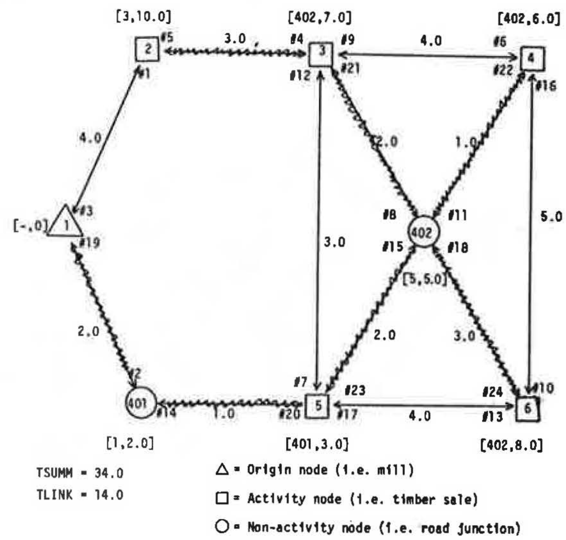


Figure 9. Network for sample forest.

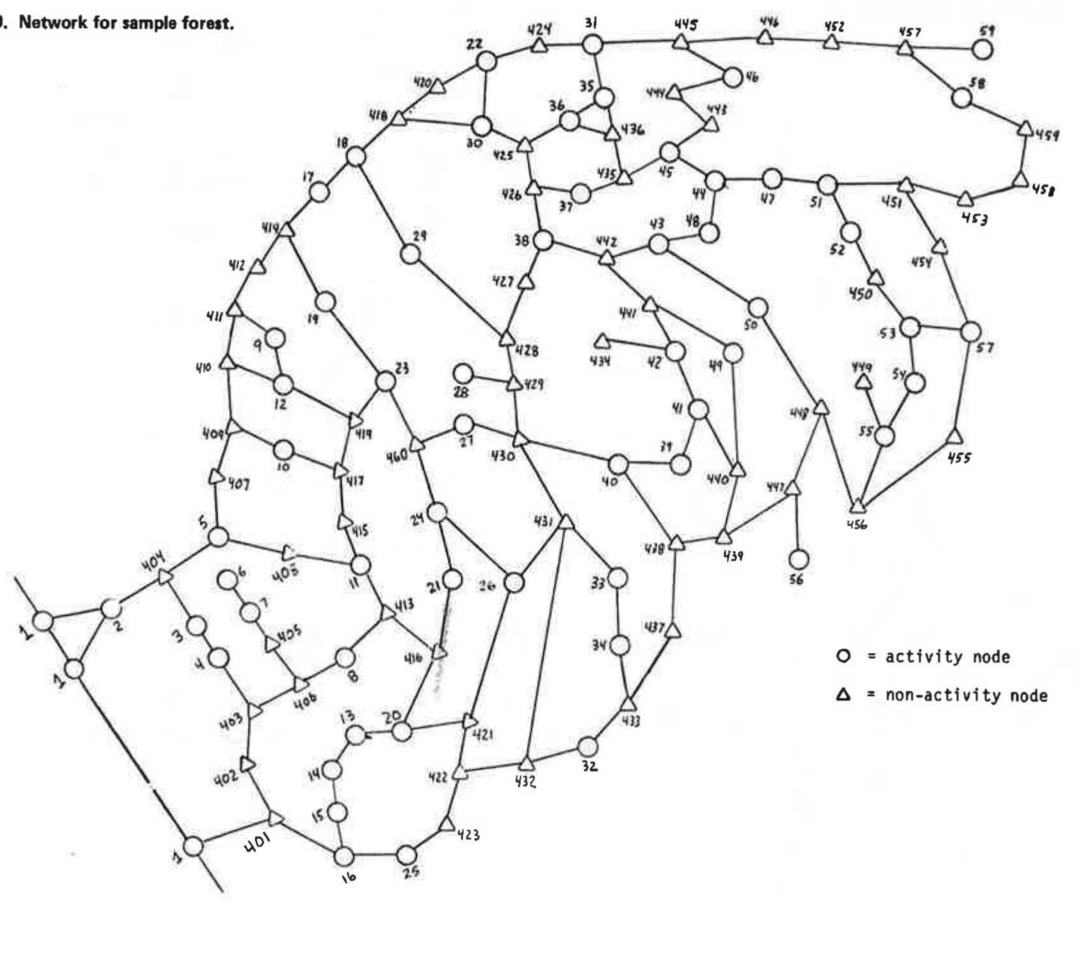


Figure 10. BUILDER results for sample forest network.

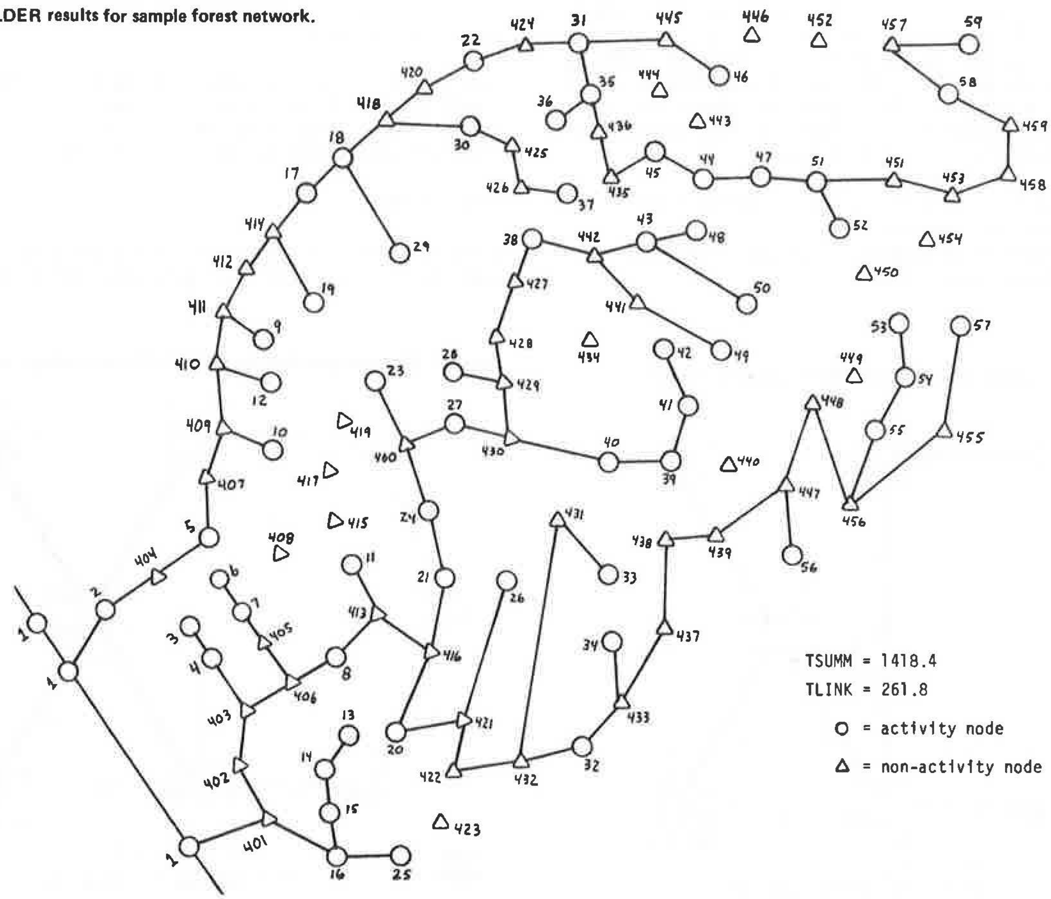


Figure 11. MINSPAN results for sample forest network.

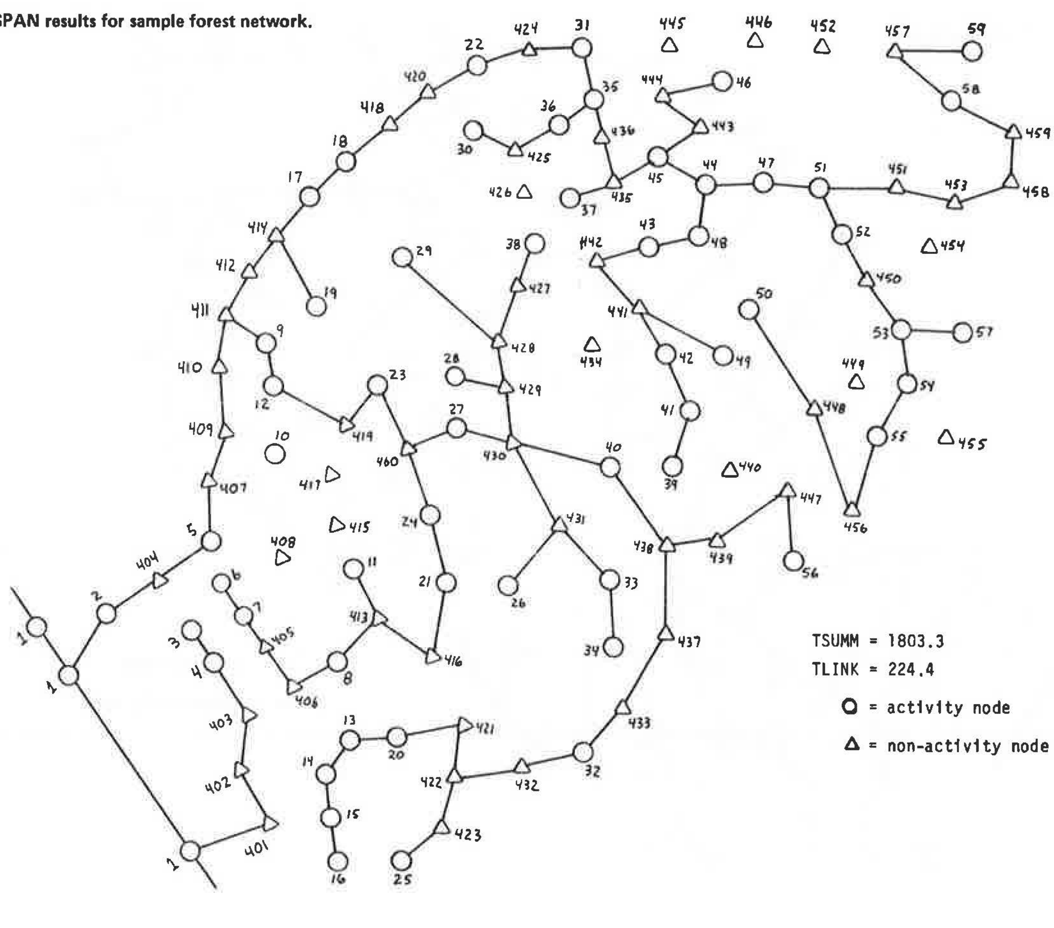


Figure 12. MINTREE results for sample forest network.

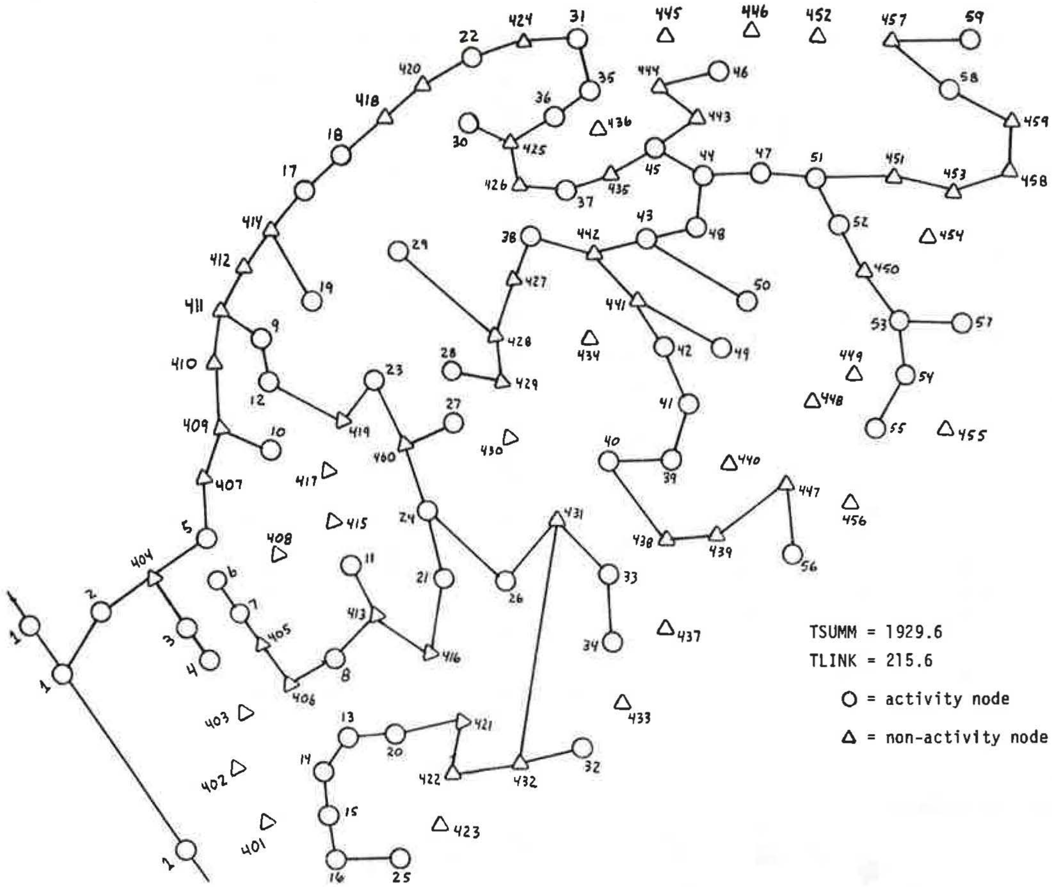


Figure 13. PLANET1 results for sample forest network.

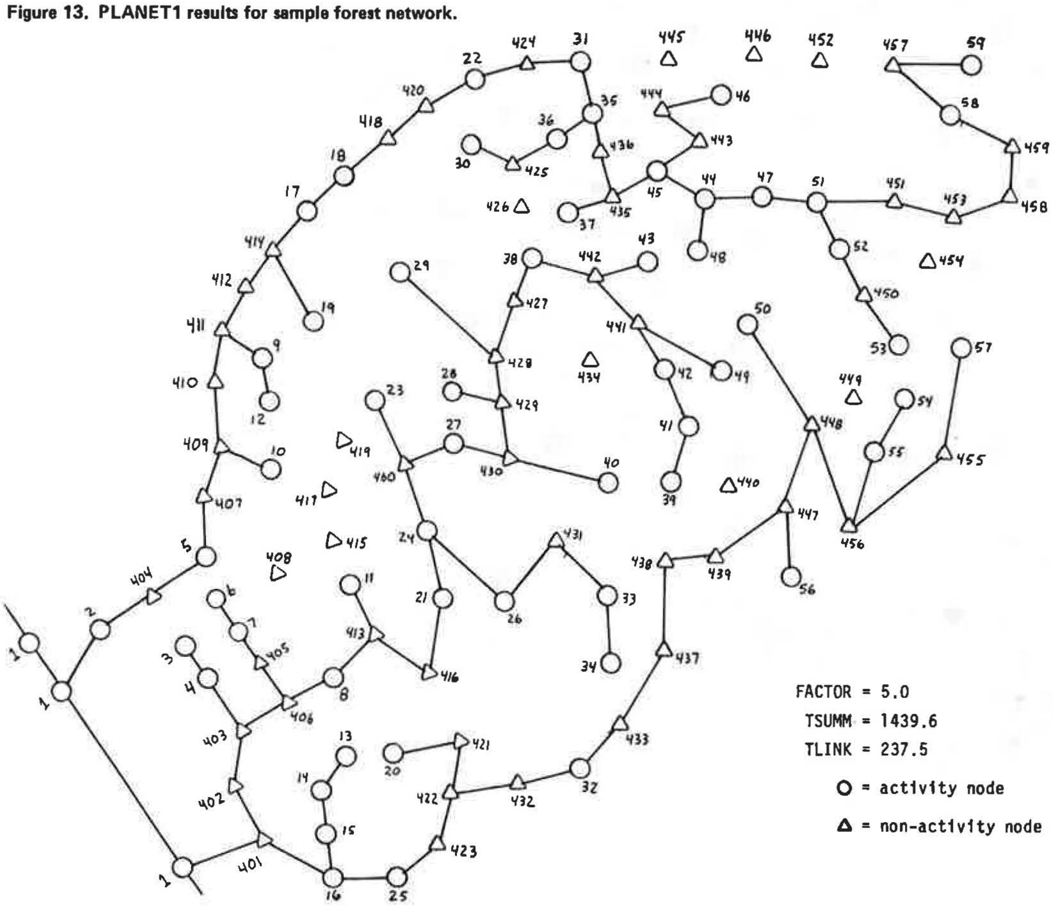


Table 5. Comparison of travel times with each activity node from node 1, highway arterial.

Node	MINSPAN	MINTREE	BUILDER	PLANET1	Node	MINSPAN	MINTREE	BUILDER	PLANET1
1	0.0	0.0	0.0	0.0	32	35.9	28.8	19.5	19.6
2	4.0	4.0	4.0	4.0	33	29.5	27.4	24.1	24.6
3	9.4	10.5	9.4	9.4	34	30.6	28.5	24.5	25.7
4	9.4	10.5	9.4	9.4	35	25.7	25.7	25.7	25.7
5	9.9	9.9	9.9	9.9	36	27.1	27.1	27.1	27.1
6	33.6	33.6	11.0	11.0	37	30.6	32.5	28.2	30.6
7	33.6	33.6	11.0	11.0	38	28.5	42.3	26.1	26.1
8	30.1	30.1	10.9	10.9	39	36.5	42.2	28.9	31.2
9	16.1	16.1	16.1	16.1	40	28.1	45.4	25.7	25.7
10	15.1	15.1	15.1	15.1	41	36.0	41.7	29.4	30.7
11	31.0	14.8	14.8	14.8	42	35.5	41.2	29.9	30.2
12	17.6	17.6	17.3	17.6	43	33.5	39.2	29.2	29.2
13	41.2	32.5	15.9	15.9	44	30.8	36.5	30.8	30.8
14	41.2	32.5	15.9	15.9	45	29.5	35.2	29.5	29.5
15	43.5	34.8	13.6	13.6	46	34.0	39.7	32.8	34.0
16	46.4	37.9	10.5	10.5	47	32.4	38.1	32.4	32.4
17	17.9	17.9	17.9	17.9	48	31.8	37.5	30.9	31.8
18	19.2	19.2	19.2	19.2	49	36.5	42.2	31.2	31.2
19	18.8	18.8	18.8	18.8	50	45.4	42.5	32.5	33.0
20	38.7	30.0	16.7	18.0	51	33.7	39.4	33.7	33.7
21	25.6	25.6	15.4	15.4	52	34.7	40.7	34.7	34.7
22	21.1	21.1	21.1	21.1	53	36.9	42.6	36.6	36.9
23	21.1	21.1	19.9	19.9	54	38.6	44.3	34.9	35.0
24	21.9	21.9	19.1	19.1	55	40.7	46.4	32.8	32.9
25	41.7	41.2	13.8	13.8	56	71.9	89.2	65.7	65.8
26	27.8	24.3	21.0	21.5	57	38.8	44.5	34.9	35.0
27	23.2	23.2	20.8	20.8	58	44.9	50.6	44.9	44.9
28	30.3	46.9	27.9	27.9	59	72.0	77.7	72.0	72.0
29	30.4	46.8	25.4	28.0	TSUMM	1803.2	1929.6	1418.4	1439.6
30	29.7	29.7	24.6	29.7	TLINK	224.4	215.6	361.8	237.5
31	23.4	23.4	23.4	23.4					

Figure 14. PLANET2 results for sample forest network.

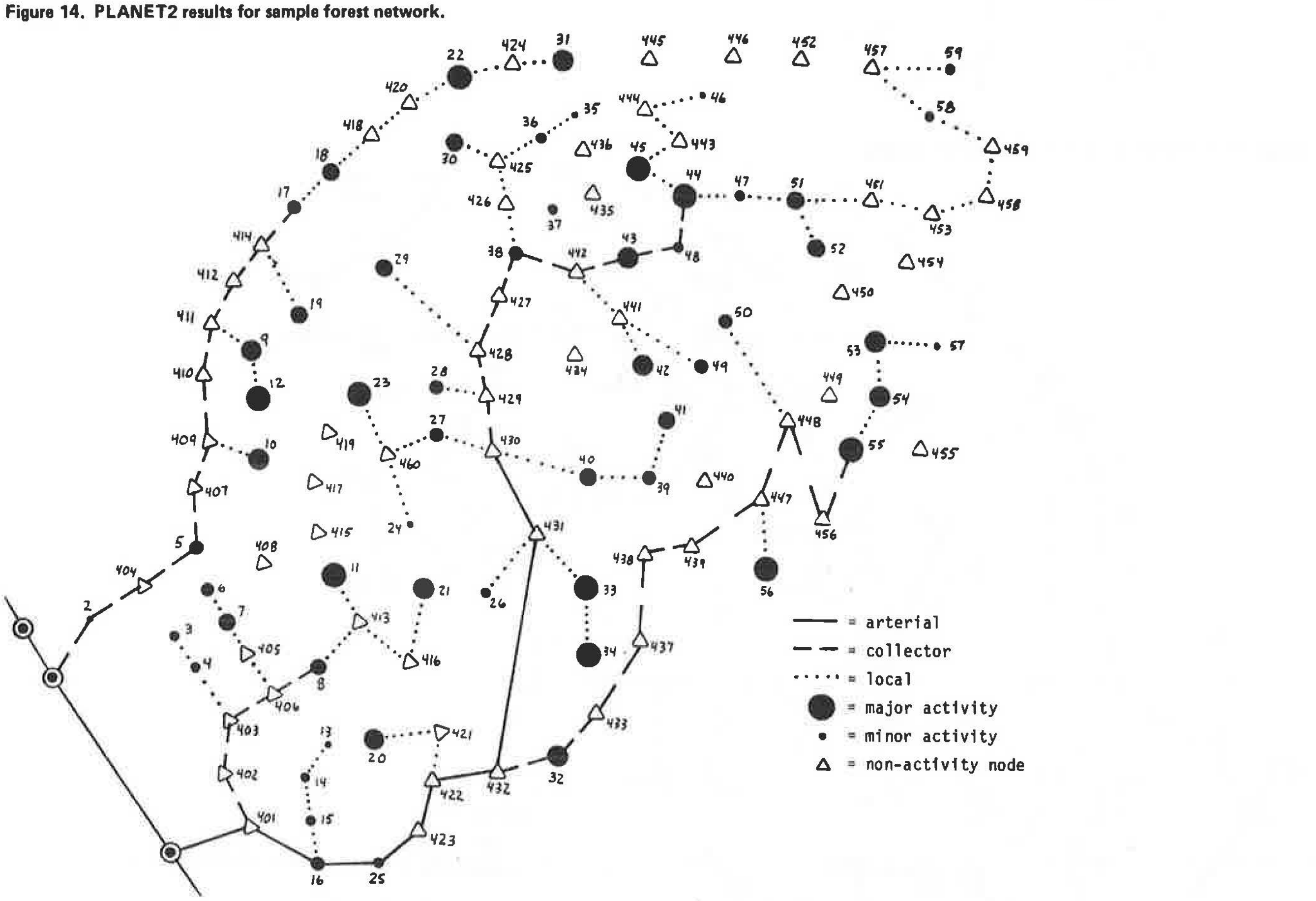


Table 6. Total network time (TSUMM) and total network distance (TLINK) versus trade-off factor.

Program	Factor	TSUMM	TLINK
BUILDER		1418.4	261.8
PLANET1	0.0	1418.4	259.5
PLANET1	0.2	1418.7	257.3
PLANET1	0.6	1421.0	249.5
PLANET1	1.0	1425.4	245.4
PLANET1	3.0	1432.1	241.7
PLANET1	5.0	1439.6	237.5
PLANET1	10.0	1481.0	228.9
PLANET1	15.0	1489.9	228.3
PLANET1	100.0	1558.2	226.0
PLANET1	999.9	1803.2	224.4
MINSPAN		1803.2	224.4
MINTREE		1929.9	215.6

imum spanning tree algorithm to determine the best network they can. PLANET1 and PLANET2 do this with very reasonable computation times. The computational times on Oregon State University's CDC Cyber 73 computer, for the network in Figure 9 were approximately 2.4 s for PLANF1 (PLANF1 generates the input file for PLANET1) and 1.4 s for PLANET1. For a similar network that has three links of different classes between each pair of nodes, the computation times for PLANET2 were approximately 10 s for PLANF2 (PLANF2 generates the input file for PLANET2) and 4 s for PLANET2.

PLANET1 should be used when the information about the network is limited. PLANET2 should be used if (a) the actual construction, maintenance, and operating costs are available; (b) different roadway classes are to be used; or (c) the demands for the individual activities are used. Some additions to PLANET1 and PLANET2 that may be possible are to

1. Divide the traffic into different vehicle classes,
2. Determine which links should be closed and which should be open at a lower class when they are not in the tree, and
3. Take roadway capacities into consideration.

These are some of the additions that should be considered in the future development of PLANET1 and PLANET2.

These two programs make it possible for the analyst or decisionmaker to analyze and evaluate the trade-offs in construction and maintenance cost as convenience is increased, that is as travel time or operating costs are reduced. Since many activities with varying objectives must be served by a forest road network, the transportation planning task is complex. Computerized techniques that indicate the trade-offs between networks identified according to differing criteria assist the decisionmaker in identifying the appropriate roadway arterial, collector, and local systems.

REFERENCES

1. Forest Service Manual. U.S. Forest Service, U.S. Department of Agriculture, 1976.
2. R. Kehr, Jr., and R.D. Layton. A Procedure to Identify Forest Arterial and Collector Road Networks. Oregon State Univ., Corvallis, Transportation Res. Rept. 79-1, Jan. 1979.
3. B. Barnes and E.C. Sullivan. Timber Transport Model, Version 2.0. Univ. of California, Berkeley, Institute for Transportation Studies Res. Rept. UCB-ITS-RR-79-18, Sept. 1980.

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Analyzing Transportation Networks for Rural Development

EDWARD C. SULLIVAN

This paper describes a new version of the Timber Transport Model, which is a comprehensive route analysis and network optimization computer program developed to support land management planning in rural forest areas. The technique is generally applicable to transportation economic analysis in any rural setting and involves the transportation of resources or agricultural commodities in a many-to-few shipping pattern. The overall capabilities and problem size limits of the program are described. Program features are illustrated through a simple example. The technique is compared with the classical transshipment problem, with which it has certain features in common. The mathematical formulations used in the program are also presented.

This paper describes the Timber Transport Model, a comprehensive network analysis computer program created to support national forest transportation and lane management planning. A previous version of this program has existed for a number of years and has been used in the selection of capital investments, maintenance levels, and, in some cases, network rehabilitation priorities following slides, floods, and other transportation emergencies (1,2). The current version contains several operational simplifications and enhancements, in many cases suggested by users throughout the country.

This technique was developed under sponsorship of the U.S. Forest Service, and consequently contains features intended to facilitate analysis of timber haul. However, it is suited to a wide range of rural transport planning situations--in particular, the analysis of penetration road networks in developing regions.

The problems to which the Timber Transport Model is suited have the following characteristics:

1. Network investment and management decisions are based primarily on service to resource-based commerce, such as agriculture, mining, or (as in the national forests) logging;
2. Transport needs are predominantly many-to-few in character, such as in farm-to-market or forest-to-mill transport;
3. Commercial transport needs are multicommodity in nature in that different market locations may exist for different goods;
4. Transportation planning, although attempting to serve numerous objectives and users, is dominated by considerations of economic and financial feasibility and market advantage; and
5. Engineering economic analysis considers the

total benefits and costs associated with the transportation system; that is, the analysis extends to benefits and costs in construction, maintenance, and user operations (user costs and benefits are usually the largest components).

Classical network analysis involves the systematic exploration of alternative routes and tallying their costs, traffic, and other consequences. Network analysis is used in planning through its application to both land use and transportation network alternatives. The Timber Transport Model provides a convenient computerized way to perform classical network analysis in fairly large real-world networks. In addition, it provides the option of network optimization, based on either cost minimization or present worth maximization. As desired, network optimization can be performed for a static situation or for multiple time periods with discounting.

Like its predecessor, the new version of the Timber Transport Model is specifically designed to place the capabilities of computerized network analysis, traffic estimation, and mathematical programming into the hands of field personnel that have relatively little specialized training in planning methods, quantitative analysis, or economics. Intended users are staff engineers and planners at field offices located throughout the country.

Our purpose here is twofold: to introduce the capabilities of the Timber Transport Model, and to show how transportation planning can be included in rural development planning. This is accomplished in two sections, the first of which describes the program, and the second of which documents an example application. The paper presents a brief discussion of implementation and transferability considerations and concludes with the mathematical program formulations used in optimization.

PROGRAM DESCRIPTION

The Timber Transport Model contains three principal capabilities:

1. Route analysis for network alternatives, which includes finding best and several next-best routes in a network on the basis of minimum operating cost, minimum travel time, or minimum travel distance;
2. Traffic assignment, which determines link traffic volumes associated with any selected set of travel routes; and
3. Network optimization, which determines the most economic combination of routes and link changes to achieve either minimum cost or maximum present worth (revenues minus transport costs).

Table 1 contains a summary of the particular features within each of the three principal model capabilities. Size limitations of problems that can be solved are listed in the table below.

Item	Must Not Exceed
Links	1000
Nodes	1400
Origins	100
Destinations	15
Best and next-best routes generated per origin	10
Time periods for discounting	7
Capital investment projects, integer 0-1 variables	99
Link volume capacities	50

The Timber Transport Model incorporates routines for data input and verification, network editing, minimum path and next-best-path estimation, traffic assignment, and mixed-integer mathematical programming.

EXAMPLE PROBLEMS

Three simple examples are used to illustrate several features of the Timber Transport Model. The first demonstrates the route-finding capability of the program, which is a necessary first pass in order to provide input to other steps. The second example demonstrates a traffic assignment. The third shows what is involved in finding the least-cost combination of capital investment alternatives for a rural transportation network to be used in a 20-year program of logging. These examples are simplified versions of the types of problems to which the model is applied in national forests throughout the United States. Although problems of this simplicity would never be solved by computer, because manual solutions are preferable for such simple problems, they are used here because representative real-world examples are too lengthy for useful illustration.

Figure 1 illustrates the existing road network in the hypothetical forest region that is the subject of our example. Each road section is crudely described in terms of its grade, quality of alignment, length, and surface type. This information is the basis for a network representation of the system, contained in Figure 2. Link geometry, condition, and surface type are used to define a link class. Unit costs ($/km) and average running speeds for different vehicles and load conditions are then associated with each class and provide a convenient method to establish costs and travel times for all network links without having to code this information separately for each link. Since many different

Table 1. Summary of model features.

Capability	Feature	Capability	Feature
Route analysis	Finds shortest routes based on travel cost, distance, or travel time Finds shortest and up to nine next shortest routes Permits selected commodities to be shipped to a specified subset of destinations Permits shortest routes to include intermediate stops for repackaging or processing Permits direct input of user-added routes	Optimization	Selects network investment projects from up to 99 alternatives that span up to seven time periods Projects are selected either to minimize the sum of capital and operating costs or to maximize product revenues minus capital and operating costs Model can select the optimal time periods in which to ship commodities Constraints may be imposed on shipment quantities, link traffic, and total capital investment in any time period Origin productivities may vary among time periods
Assignment	Accommodates both all-or-nothing and multipath traffic assignment Output includes traffic in vehicles and volumes of commodities Traffic output is separate for different time periods		

Figure 1. Example road system.

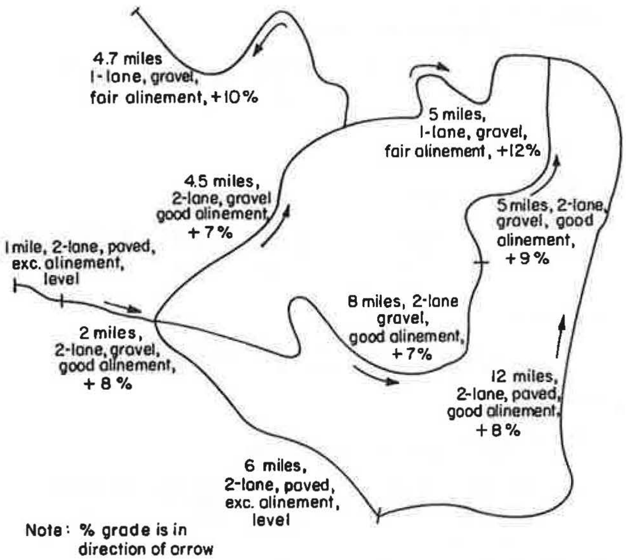
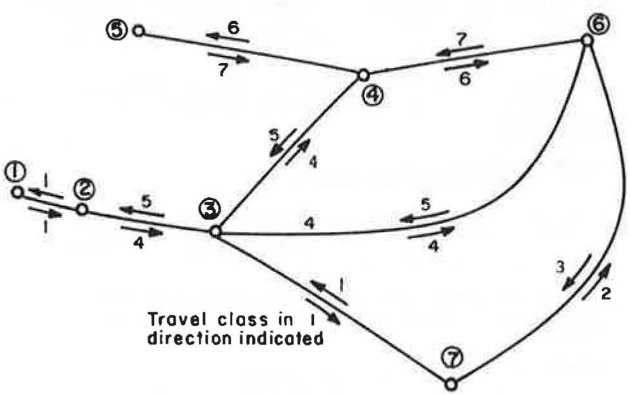


Figure 2. Example road system with node numbers and link travel classes assigned.



links commonly have similar physical characteristics, and therefore fall into the same class, this method reduces the labor involved in network coding substantially.

Travel speeds and vehicle operating costs associated with the different link classes may be obtained from a variety of published sources. Most current users of the model use U.S. Forest Service-compiled cost data for logging trucks (3). The agency periodically publishes unit costs for logging trucks that correspond to a system of 320 standard link classes that are considered to be appropriate for conditions typical of the U.S. national forests (4). For developing countries and applications other than logging, it would be appropriate to use cost data from other sources, such as data gathered by the British Road Research Laboratory in Kenya and more recently by the World Bank and the Brazilian government in Brazil (5,6).

Route Analysis

In order to perform route analysis the origin nodes, destination nodes, unit costs, travel times, estimates of the quantities to be shipped from each origin node, and the average truckload must be specified. One must also specify how routes are to be selected, for example, the three best routes computed on the basis of minimum travel cost.

Figure 3 shows the model output in the case of the example; where nodes 4, 5, and 6 are origin nodes that ship, respectively, 6, 12, and 5 million board ft annually. (The average load is 5000 board ft/truck.) Node 1 is specified as the destination. In this run, the model evaluates the three shortest routes on the basis of cost, followed by the three shortest routes on the basis of travel time. Although these are identical in this case, they are not always so.

Traffic Assignment

Figure 4 shows the result of using the model to perform an all-or-nothing traffic assignment by using the least-cost routes determined previously. To produce this result, it is necessary to specify, for each origin, the proportion of its shipment that uses each route. Through such specifications, a tally of the traffic consequences of, for example, routing half of a shipment one way, half another is simple.

At this stage, it would be possible to introduce any other routes for evaluation and use in traffic assignment. This is accomplished by listing the nodes through which a route reaches a destination from its origin. This is not illustrated in the example.

Optimization

Figure 5 illustrates how the model is used to solve network optimization problems. The first step involves creation of a network that contains links that are identified as investment projects; that is, portions of the network that can be used for traffic only if an associated capital cost is incurred. Projects are specified by means of network updates. The revised network, which contains projects, is then subjected to route analysis by following the method illustrated in Figure 3. The result is a collection of alternative travel routes, some of which use projects, others not. At this stage, it is essential to confirm that the available routes contain all significant combinations of implementing and not implementing the projects under consideration.

Once route analysis is complete for the network that contains the candidate investment projects, economic optimization can begin. The result of using the model to compute the overall least-cost network and routing pattern for the specified quantity of timber appears in Figure 5. In this example the optimization problem is defined for three time periods. For each period, maximum and minimum limits are imposed on the total amount to be shipped.

To illustrate a form of sensitivity analysis, Figure 5 shows how the origin volumes and capital costs of projects can be changed at optimization time. The model determines which combination of projects, routes, and shipments from the various origins produces the overall least-cost solution. In this simple example, the result is intuitive--to defer the most-expensive shipments as long as possible, subject to satisfaction of the minimum constraints of shipping 3 million board ft in each of the early time periods. The result is that investment project 1 is economically justified in that the associated savings in user cost exceed the size of the investment.

DISCUSSION OF RESULTS

This approach to network analysis is significantly

Figure 3. Output for route-finding example.

```
PLEASE GIVE OUTPUT TO   ED SULLIVAN
*** OUTPUT FROM ASCII FORTRAN TIMBER MODEL ***
YOUR INPUT CRITERIA FILE NAME IS   CRIT1.
YOUR INPUT LINK FILE NAME IS   LINKS.
YOUR UPDATED LINK FILE NAME IS
YOUR OUTPUT TRAFFIC FILE NAME IS   TRAF1.
YOUR MESSAGE FILE NAME IS   MESS1.
YOUR DOCUMENT FILE NAME IS   DOC1.
@XQT ITS*BBTIMBER.TIMBRIIX
TRANSPORTATION TIMBER MODEL PROGRAM

DATA  RUN

NMIL =  1, NMKT =  1, NSAL =   3
NPATHS =  2, TRUCK LOADING = 5.000

MILLS-
MILL DESCRIPTION                      NODE MIN-VOL  MAX-VOL
                                S001    1  NONE  UNLIMITED

MARKETS-

                                S001    1


MARGINAL LOG HAUL COSTS -
              FOR KTYP =         1       2       3       4       5       6       7
 UP TO DISTANCE=  100.000-     .139    .428    .236    .469    .264    .600    .
...328
 FROM  100.000 TO  200.000-    .139    .428    .236    .469    .264    .600    .
...328
 BEYOND DISTANCE=  200.000-    .139    .428    .236    .469    .264    .600    .
...328


MARGINAL LUMBER HAUL COSTS -
              FOR KTYP =         1       2       3       4       5       6       7
 UP TO DISTANCE=  100.000-     .139    .428    .236    .469    .264    .600    .
...328
 FROM  100.000 TO  200.000-    .139    .428    .236    .469    .264    .600    .
...328
 BEYOND DISTANCE=  200.000-    .139    .428    .236    .469    .264    .600    .
...328

NEW LINK CLASS SPEEDS UP TO CLASS   7

55.00 27.00 48.00 27.00 42.00 17.00 20.00

PATH CRITERIA - COST TIME


NO UPDATES


SALES-
SALE  SALE DESCRIPTION        ROAD MILE  SALE  VOL-    TRUCK NO.OF  B  ALLOWED MILL
NODE                               POST  CODE   UME    LOAD TRUCKS     DESTINATIONS
  6                                      T001  5000.    5.   1000      S001
  5                                      T002 12000.    5.   2400      S001
  4                                      T003  6000.    5.   1200      S001

*************

GENERATED ROUTE NO.    1
            1ST SHORTEST PATH   BASED ON COST

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE          TOT COST  DISTANCE       TIME
     6         1       1             3.495    12.500       25.377
PATH/    6-    4-    3-    2-    1-

*************

GENERATED ROUTE NO.    2
            2ND SHORTEST PATH   BASED ON COST

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE          TOT COST  DISTANCE       TIME
     6         1       1             4.099    16.000       22.520
PATH/    6-    3-    2-    1-

$$$$$$$$$$$$$
*************

GENERATED ROUTE NO.    3
            1ST SHORTEST PATH   BASED ON COST

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE          TOT COST  DISTANCE       TIME
     5         1       1             3.397    12.200       24.477
PATH/    5-    4-    3-    2-    1-

*************

GENERATED ROUTE NO.    4
            2ND SHORTEST PATH   BASED ON COST

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE          TOT COST  DISTANCE       TIME
     5         1       1             8.641    25.700       54.267
PATH/    5-    4-    6-    3-    2-    1-

$$$$$$$$$$$$$
*************
```

Figure 3. Continued.

```
GENERATED ROUTE NO.    5
            1ST SHORTEST PATH   BASED ON COST

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE          TOT COST  DISTANCE       TIME
     4         1       1             1.855     7.500       10.377
PATH/    4-    3-    2-    1-

*************

GENERATED ROUTE NO.    6
            2ND SHORTEST PATH   BASED ON COST

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE          TOT COST  DISTANCE       TIME
     4         1       1             7.099    21.000       40.167
PATH/    4-    6-    3-    2-    1-

$$$$$$$$$$$$$

LEAST COST PATH TOTAL UNDISCOUNTED COST FOR VOLUME
      23000.00 FROM    3 SALE NODES = $               .00

*************

GENERATED ROUTE NO.    7
            1ST SHORTEST PATH   BASED ON TIME

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE            TIME    DISTANCE       COST
     6         1       1            22.520    16.000        4.099
PATH/    6-    3-    2-    1-

**WARNING** IDENTICAL TOTAL COST AND SALE ORIGIN AS PREVIOUSLY
GENERATED PATH.   IT WAS NOT WRITTEN ONTO YOUR TRAFFIC FILE
*************

GENERATED ROUTE NO.    8
            2ND SHORTEST PATH   BASED ON TIME

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE            TIME    DISTANCE       COST
     6         1       1            25.377    12.500        3.495
PATH/    6-    4-    3-    2-    1-

**WARNING** IDENTICAL TOTAL COST AND SALE ORIGIN AS PREVIOUSLY
GENERATED PATH.   IT WAS NOT WRITTEN ONTO YOUR TRAFFIC FILE
$$$$$$$$$$$$$
*************

GENERATED ROUTE NO.    9
            1ST SHORTEST PATH   BASED ON TIME

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE            TIME    DISTANCE       COST
     5         1       1            24.477    12.200        3.397
PATH/    5-    4-    3-    2-    1-

**WARNING** IDENTICAL TOTAL COST AND SALE ORIGIN AS PREVIOUSLY
GENERATED PATH.   IT WAS NOT WRITTEN ONTO YOUR TRAFFIC FILE
*************

GENERATED ROUTE NO.    10
            2ND SHORTEST PATH   BASED ON TIME

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE            TIME    DISTANCE       COST
     5         1       1            54.267    25.700        8.641
PATH/    5-    4-    6-    3-    2-    1-

**WARNING** IDENTICAL TOTAL COST AND SALE ORIGIN AS PREVIOUSLY
GENERATED PATH.   IT WAS NOT WRITTEN ONTO YOUR TRAFFIC FILE
$$$$$$$$$$$$$
*************

GENERATED ROUTE NO.    11
            1ST SHORTEST PATH   BASED ON TIME

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE            TIME    DISTANCE       COST
     4         1       1            10.377     7.500        1.855
PATH/    4-    3-    2-    1-

**WARNING** IDENTICAL TOTAL COST AND SALE ORIGIN AS PREVIOUSLY
GENERATED PATH.   IT WAS NOT WRITTEN ONTO YOUR TRAFFIC FILE
*************

GENERATED ROUTE NO.    12
            2ND SHORTEST PATH   BASED ON TIME

TIMBER SALE  MILL  APPRAISAL       MINIMUM   ASSOCIATED  ASSOCIATED
    NODE     NODE    NODE            TIME    DISTANCE       COST
     4         1       1            40.167    21.000        7.099
PATH/    4-    6-    3-    2-    1-

**WARNING** IDENTICAL TOTAL COST AND SALE ORIGIN AS PREVIOUSLY
GENERATED PATH.   IT WAS NOT WRITTEN ONTO YOUR TRAFFIC FILE
$$$$$$$$$$$$$

LEAST TIME PATH TOTAL UNDISCOUNTED COST FOR VOLUME
      23000.00 FROM    3 SALE NODES = $               .00

THE FOLLOWING ROUTES (IDENTIFIED BY ROUTE NUMBER)
WERE NOT ADDED TO YOUR TRAFFIC FILE BECAUSE THEY WERE
EITHER DUPLICATE PATHS OR PATHS THAT USED PROJECT LINKS
WITHOUT COMMON TIME PERIOD AVAILABILITY
    7    8    9   10   11   12
```

Figure 4. Output for traffic-assignment example.

```
PLEASE GIVE OUTPUT TO    ED SULLIVAN
*** OUTPUT FROM ASCII FORTRAN TIMBER MODEL ***
YOUR INPUT CRITERIA FILE NAME IS   CRIT2.
YOUR INPUT LINK FILE NAME IS   LINKS.
YOUR INPUT TRAFFIC FILE NAME IS   TRAF1.
YOUR MESSAGE FILE NAME IS   MESS2.
YOUR DOCUMENT FILE NAME IS   DOC2.
@XQT ITS*BBTIMBER.TIMBRIIX
TRANSPORTATION TIMBER MODEL PROGRAM

PICK  RUN

SALES-
SALE  SALE DESCRIPTION     ROAD MILE  SALE VOL-    TRUCK NO.OF  B  ALLOWED MILL
NODE                            POST  CODE  UME    LOAD TRUCKS     DESTINATIONS
  6                                   T001  5000.   5.  1000       S001
  5                                   T002 12000.   5.  2400       S001
  4                                   T003  6000.   5.  1200       S001

USER SELECTED PATHS
      GENERATED   SALE  NEXT BEST  MINIMUM PATH   PERCENTAGE OF
     ROUTE NUMBER NODE  PATH NO.    CRITERION    VOLUME ALLOWED
                                                    OVER PATH
           1        6      1           COST          100.00
           3        5      1           COST          100.00
           6        4      2           COST          100.00
NO USER DEFINED PATHS INCLUDED
```

Figure 4. Continued.

```
TRAFFIC ASSIGNMENT FOR LOADED TRUCKS

     FROM     TO       TIMBER          ASSOCIATED
     NODE    NODE      TRAFFIC           VOLUME


       2       1        4600            23000.0
       3       2        4600            23000.0
       4       3        3400            17000.0
       4       6        1200             6000.0
       5       4        2400            12000.0
       6       3        1200             6000.0
       6       4        1000             5000.0


TOTAL UNDISCOUNTED HAUL COST ON ASSIGNED PATHS = $       100830.50
```

Figure 5. Output for least-cost network and routing pattern.

```
PLEASE GIVE OUTPUT TO    ED SULLIVAN
*** OUTPUT FROM ASCII FORTRAN TIMBER MODEL ***
YOUR INPUT CRITERIA FILE NAME IS   CRIT4.
YOUR INPUT LINK FILE NAME IS   LINKS.
YOUR INPUT TRAFFIC FILE NAME IS   TRAF3.
YOUR WORKING MATRIX FILE NAME IS   MAT4.
YOUR MESSAGE FILE NAME IS   MESS4.
YOUR DOCUMENT FILE NAME IS   DOC4.
@XQT ITS*BBTIMBER.TIMBRIIX
TRANSPORTATION TIMBER MODEL PROGRAM

OPT3  RUN

SALES-
SALE  SALE DESCRIPTION   ROAD MILE  SALE VOL-    TRUCK NO.OF  B  ALLOWED MILL
NODE                          POST  CODE  UME    LOAD TRUCKS     DESTINATIONS
  6                                 T001  5000.   5.  1000       S001
  5                                 T002 12000.   5.  2400       S001
  4                                 T003  6000.   5.  1200       S001

 3 PERIODS
    INTEREST RATE =  .03
    PROJECT COSTS OCCUR   .10 THROUGH PERIOD
    HAUL COSTS AND PROFITS OCCUR  .50 THROUGH PERIOD

NUMBER OF LINES OF TIME PERIOD SALE VOLUMES =   0

  PERIOD 1 BEGINS YEAR  0 ENDS YEAR  1
  PROJECT YEAR SET TO  0 HAUL YEAR SET TO  1
  PROJECT COST DISCOUNT FACTOR 1.0000 HAUL DISCOUNT FACTOR  .9709

  PERIOD 2 BEGINS YEAR  1 ENDS YEAR  6
  PROJECT YEAR SET TO  1 HAUL YEAR SET TO  4
  PROJECT COST DISCOUNT FACTOR  .9709 HAUL DISCOUNT FACTOR  .8885

  PERIOD 3 BEGINS YEAR  6 ENDS YEAR 10
  PROJECT YEAR SET TO  6 HAUL YEAR SET TO  8
  PROJECT COST DISCOUNT FACTOR  .8375 HAUL DISCOUNT FACTOR  .7894

    CHANGES IN PROJECT DATA
    PROJECT     UNDISC. COST BEGINS  ENDS
       1           50000.      1      3
       2            1300.      1      3

TOTAL FOREST HARVEST CONSTRAINTS
TIME MINIMUM MAXIMUM
PER. VOLUME  VOLUME
* 1  3000.0 23000.0 VALUES ASSUMED*
* 2  3000.0 23000.0 VALUES ASSUMED*
* 3  3000.0 23000.0 VALUES ASSUMED*
NO USER ADDED PATHS GIVEN


**CHOSEN PATHS**


  GENERATED ROUTE NO.   2
     PATH NODES       6-    4-     3-    2-     1-
  TIME             TRUCK-  UNIT
  PER. VOLUME      LOADS    COST TOTAL COST
  ---- ------     -------  ----- ----------
   3   5000.       1000    3.50    17475.
                       (  2.76)(  13795.)
  ALL TIME PERIODS COMBINED:
       5000.       1000    3.50    17475.
                               (  13795.)
```

Figure 5. Continued

```
GENERATED ROUTE NO.   7
   PATH NODES       5-    4-    3-    2-    1-
 TIME            TRUCK-  UNIT
 PER. VOLUME     LOADS    COST TOTAL COST
 ---- ------    -------  ----- ----------
  3   12000.      2400   3.40    40760.
                      (  2.68)(  32177.)
 ALL TIME PERIODS COMBINED:
      12000.      2400   3.40    40760.
                              (  32177.)


GENERATED ROUTE NO.  12
   PATH NODES      4-    3-    2-    1-
 TIME            TRUCK-  UNIT
 PER. VOLUME     LOADS    COST TOTAL COST
 ---- ------    -------  ----- ----------
  1   3000.       600    1.86     5565.
                      (  1.80)(   5403.)
  2   3000.       600    1.86     5565.
                      (  1.65)(   4945.)
 ALL TIME PERIODS COMBINED:
      6000.      1200    1.86    11131.
                              (  10348.)

FOR VOLUME  23000.00 TOTAL COST = $   69366.50   DISCOUNTED = $   56319.86


SALE TIME PERIOD HARVEST VOLUMES IN OPTIMAL SOLUTION
 SALE NODE TOTAL VOLUME FOR    VOLUMES HARVESTED DURING TIME PERIOD:
           ALL TIME PERIODS       1      2      3
     6          5000.            0.     0. 5000.
     5         12000.            0.     0.12000.
     4          6000.         3000.  3000.    0.


PROJECTS IN OPTIMAL SOLUTION
 PROJECT NUMBER  COST    DISCOUNTED COST
                                             FROM     TO
                                             NODE    NODE
     1       $ 50000.  $ 50000.
                                               3       4
                                               4       3


     TOTAL COST FOR   1 PROJECTS  $     50000.
     DISCOUNTED COST $    50000.

     SOLUTION PRESENT WORTH = $  -106320.


TRAFFIC ASSIGNMENT FOR LOADED TRUCK TRIPS
                 ALL TIME PERIODS
 FROM   TO   TIMBER  ASSOCIATED   TIMBER VOLUME/(TRUCKS) DURING TIME PERIOD:
 NODE  NODE  TRAFFIC   VOLUME           1           2           3
   2     1    4600     23000.        3000        3000        17000
                                 (   600)  (   600)  (  3400)
   3     2    4600     23000.        3000        3000        17000
                                 (   600)  (   600)  (  3400)
   4     3    4600     23000.        3000        3000        17000
                                 (   600)  (   600)  (  3400)
   5     4    2400     12000.           0           0        12000
                                 (     0)  (     0)  (  2400)
   6     4    1000      5000.           0           0         5000
                                 (     0)  (     0)  (  1000)
```

different from other network optimization approaches found in the literature. This approach has particular strengths and weaknesses compared with other ways of addressing this type of problem, and these warrant discussion.

Perhaps the closest relative to this approach is the classical transshipment problem, discussed in mathematical programming textbooks such as Dantzig (9, chapter 16). The transshipment problem forms the basis for the Integrated Resource Planning Model, which also has been developed for use in U.S. Forest Service land management planning (10). The classical transshipment formulation can easily be extended to a mixed-integer formulation in which links of the network can be used only in conjunction with a lump-sum capital investment, in the manner of the Timber Transport Model. The transshipment problem can also be embellished with multiple time periods and various practical constraints such that the problems that are addressed by the two methods begin to look similar.

The essential difference between a mixed-integer adaptation of the transshipment problem and the Timber Transport Model lies in the explicit analysis of routes. The transshipment problem does not deal with routes explicitly; therefore, the traffic on any link is not able to be traced to a particular origin-destination source. Thus, particularly in solutions that involve link capacity constraints, it is not always clear how particular shipments should be routed in order to achieve the optimal solution, because determination of who is diverted from congested links is, in large networks, somewhat difficult.

On the other hand, the explicit route analysis incorporated in the Timber Transport Model places a burden on the user to be sure that all significant routing possibilities are considered, a burden not encountered in the transshipment formulation. Although this forces users to examine intermediate output carefully and leads to a better intuitive understanding of how the system is operating, there is no question that a substantial level of effort is involved in finding a solution to a large, practical problem.

Another difference in using explicit routes in the analysis is that it allows determination of optima based on other-than-minimum paths. For example, if the second least-cost route for an origin is preferred because the least-cost route is unacceptable due to vehicle size limitations or some other reason, in the Timber Transport Model, the ineligible route may be dropped prior to optimization.

These simple examples are intended to give an introduction to what the model can do, and, intentionally, many features are excluded that are typically used in day to day transportation planning. For example, instead of minimizing project plus user costs, specification would have been possible for market prices for delivered commodities for each origin, and then the optimization could determine whether shipments from all origins are, in fact, economic, in the sense of contributing to the net worth of the solution. Also, a variety of other constraints are available, including budget constraints on expenditures for projects in different time periods.

IMPLEMENTATION CONSIDERATIONS

The Timber Transport Model is programmed in FORTRAN for the Department of Agriculture's Univac 1100 series computer. In this form, the FORTRAN program produces a matrix for the mixed-integer optimization problem; it then solves the problem by using Sperry Univac's Functional Mathematical Programming System. A second version, with smaller problem size capability, exists on the University of California's CDC 6400 computer, for which the mixed integer optimization problem is solved directly by using a FORTRAN subroutine.

The mathematical program formulations used in the optimization are described in the following section. Additional information about the program is available (7). Copies may be obtained from the Systems Unit of the Institute of Transportation Studies, University of California, Berkeley.

OPTIMIZATION METHOD

Cost Minimization

The mixed-integer formulation for minimizing total transportation cost has the following form:

$$\text{Minimize } z = \sum_{ijkt} C_{ijkt} \cdot X_{ijkt} + \sum_{r} D_r \cdot I_r \tag{1}$$

subject to the following constraints:

Each timber node must export a given timber volume after all time periods,

$$\sum_{jkt} \alpha_i \cdot X_{ijkt} = \overline{T}_i \quad \text{for all timber nodes (i) and periods (t)} \tag{2}$$

Each timber node may export up to a given timber volume in each period,

$$\sum_{jk} \alpha_i X_{ijkt} \leqslant T_{it} \tag{3}$$

Resource use constraints are necessary for cost minimization,

$$\sum_{ijk} \alpha_i X_{ijkt} \geqslant \overline{V}_t \quad \text{for each time period (t)} \tag{4}$$

Resource conservation constraints are optional to cost minimization,

$$\sum_{ijk} \alpha_i X_{ijkt} \leqslant V_t \quad \text{for some periods (t)} \tag{5}$$

$$\sum_{ik} \alpha_i X_{ijkt} \leqslant K_{jt} \quad \text{for all mills (j) and periods (t)} \tag{6}$$

The traffic on certain links during certain time periods is optionally limited,

$$\sum_{ijk} [P_{lt}] \, \beta_{it} X_{ijkt} \leqslant F_{lt} \quad \text{for some links (1) and a time period (t)} \tag{7}$$

The timber volume to each mill in each period has an optional lower bound,

$$\sum_{ij} \alpha_i X_{ijkt} \geqslant \overline{K}_{jt} \quad \text{for all mills (j) and periods (t)} \tag{8}$$

The traffic on certain links during any period is optionally limited,

$$\sum_{ijk\epsilon} [P_l] \, \beta_{it} X_{ijkt} \leqslant F_l \quad \text{for some links (1) and any time period} \tag{9}$$

Certain routes require certain projects to be built,

$$\sum_{ijk\epsilon} [U_r] \, X_{ijkt} - h_r I_r \leqslant 0 \quad \text{for all projects (r)} \tag{10}$$

where

X_{ijkt} = number of truck trips in period t on the kth route that connects timber node i and mill node j; [An optional formulation that includes the variable Y_{ijkt} to represent independently routed return trips is not described. It follows directly from partitioning $\{X_{ijkt}\}$ and $\{C_{ijkt}\}$ into two equivalent subsets that apply, respectively, to the loaded and unloaded travel direction.]

C_{ijkt} = round trip cost per truck in period t and route ijk;

I_r = -1, if project r is built, 0 otherwise;

D_r = investment cost associated with project r;

$\bar{T}_i$ = timber volume (MBF) that must be exported from node i in all time periods;

T_{it} = timber volume (MBF) exported from node i in period t;

V_t = upper limit on total timber harvested in period t;

$\bar{V}_t$ = lower limit on total timber harvested in period t;

K_{jt} = maximum allowed volume for mill node j in period t;

$\bar{K}_{jt}$ = smallest acceptable timber volume (MBF) for mill j in period t;

$\bar{F}_l$ = maximum permissible daily (or monthly, etc.) traffic (trucks) on link l during any time periods;

F_{lt} = maximum permissible daily (or monthly, etc.) traffic (trucks) on link l in time period t;

α_l = average truck load (MBF) for timber from node i;

β_{it} = proportion of timber from node i in period t that is transported during the link capacity period;

$\{P_l\}$ = set of routes (ijk) that use link l during any time period;

$\{P_{lt}\}$ = set of routes (ijk) that use link l during any time period;

$\{U_r\}$ = set of routes (ijk) that require project r to be built;

h_r = arbitrary large constant greater than the total number of trucks from all timber nodes in all periods that use project r;

i = particular timber node;

j = particular lumber node;

k = sequence number of a particular route for a given combination of i and j;

t = particular time period;

r = particular investment project; and

l = particular link in the transportation network.

In the formulation, the Xs and Is are unknowns computed by the solution algorithm. All other parameters are known. All costs are automatically discounted prior to use. Project investment costs are the sum of initial outlays plus a discounted stream of maintenance costs incurred in subsequent periods.

Present Worth Maximization

The option to maximize present worth is achieved by modifying the previous mixed-integer program as follows.

The objective function becomes

$$\text{Maximize } Z' = \sum_{it} W_{it} \left(\sum_{jk} X_{ijkt} \right) - \sum_{ijkt} C_{ijkt} X_{ijkt} - \Sigma_r D_r I_r \qquad (11)$$

Constraint I becomes

$$\sum_{jk} \alpha_i X_{ijkt} \leq \bar{T}_i' \quad \text{for all timber nodes, (i) and all periods (t)} \qquad (12)$$

Resource conservation constraints are necessary for present worth

$$\sum_{ijk} \alpha_i X_{ijkt} \leq V_t \quad \text{for each period (t)} \qquad (13)$$

Resource use constraints are optional for present worth

$$\sum_{ijk} \alpha_i X_{ijkt} \geq \bar{V}_t \quad \text{for some periods (t)} \qquad (14)$$

The program optionally includes budget constraints

$$\Sigma_r S_r I_r \leq B \qquad (15a)$$

or

$$\Sigma_r S_r I_r \geq \bar{B} \qquad (15b)$$

where

W_{it} = present value of timber cut in period t at node i,

T_{it} = volume of timber available for harvest at node i in all time periods,

V_t = upper limit on total timber harvested in period t,

$\bar{V}_t$ = lower limit on total timber harvested in period t,

S_r = budget cost for project r,

B = upper limit on budget funds available, and

$\bar{B}$ = lower limit on budget funds.

All other parameters are as defined in the preceding section.

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REFERENCES

1. E.C. Sullivan. A Transportation Analysis Technique for Natural Resources Management. Proc., First International Conference on Transportation Research, Transportation Research Forum, Bruges, Belgium, June 1973, pp. 772-788.
2. J. Long. Computer-Aided Timber Transport Analysis: Five Case Studies. Institute of Transportation Studies, Univ. of California, Berkeley, Special Rept. UCB-ITS-SR-77-2, Feb. 1977.
3. J.J. Byrne, R.J. Nelson, and P.H. Googins. Logging Road Handbook: The Effect of Road Design on Hauling Costs. U.S. Department of Agriculture, Handbook 183, Dec. 1960.
4. R. Tangeman and C. Weller. Manual Network Analysis Method for Timber Sales, Rev. ed. U.S. Forest Service, Engineering Tech. Rept., Oct. 1977.
5. H. Hide and others. The Kenya Road Transport Investment Cost Study: Research on Vehicle Operating Costs. Department of the Environment, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Rept. LR 672, 1975.
6. B.C. Butler, Jr. and others. Relating Vehicle Operating Costs to Low Volume Road Parameters in Brazil. TRB, Transportation Research Record 702, 1979, pp. 320-327.
7. B. Barnes and E.C. Sullivan. Timber Transport Model: Version 2.0. Institute of Transportation Studies, Univ. of California, Berkeley, Res. Rept. UCB-ITS-RR-79-18, Sept. 1980.
8. E. Sullivan and M. Strome. A Network Model for Forest Transportation Analysis. Institute of Transportation Studies, Univ. of California, Berkeley, Res. Rept. UCB-ITS-RR-79-16, Dec. 1979.
9. G.B. Dantzig. Linear Programming and Extensions. Princeton Univ. Press, Princeton, NJ, 1963.
10. M.W. Kirby and others. Guide to the Integrated Resource Planning Model. Forest Service, U.S.

Department of Agriculture, Berkeley, CA, Jan. 1980.

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Optimization of Roadway Structural Design and Maintenance Strategies with Special Reference to Developing Countries

ALEX T. VISSER AND W.R. HUDSON

The decision to build a bituminous road instead of an unpaved road is often based on preconceived bias, historic preferences, or incomplete analysis. An analysis that uses systems methodology to minimize the total cost, which comprises road construction, maintenance, and user costs, is a just and scientific basis for such decisions. The preferred decision rule is the first-year benefits criterion; i.e., it is advisable to pave the road when the ratio of the net first-year benefits of paving to the difference in construction cost between a paved and unpaved road equals the opportunity cost of capital. For the cost figures and policy constraints adopted, paving becomes an economic proposition at an average daily traffic rate of 575. A reduction of the available maintenance funds to half the optimum has a limited impact on unpaved roads but could have severe implications for paved roads. The major factor that affects warrants for paving is the lower cost of the gravel for unpaved road-wearing courses relative to the cost of materials for paved roads. Dust, accidents, and the depletion of natural resources are factors that cannot yet be included in an economic evaluation, but they should be taken into account together with the results from the economic analysis in a utility analysis.

One of the major problems for developing countries is the development of an adequate road network under conditions of limited available financing. Despite this constraint, frequently the philosophy is to construct paved or black-top roads because this is considered a sign of development. Thus, gravel roads are often not even considered as an alternative, although in many instances they provide the most economical road network, as will be shown. In addition to this problem associated with the development of a road network, two further problems exist in the rational determination of warrants for paving.

1. In certain cases a road authority may use an average daily traffic (ADT) level of 200-250 (such as in South Africa) or even as low as 100 (in Texas) as a fixed warrant for constructing a black-top road. These traffic figures are usually of a historic origin based on experience and availability of funds, and no cognizance is taken of the benefits and cost of paving. As funds become more scarce, the traffic warrants for paving usually increase.
2. Because of limited funds for upgrading gravel roads, some road authorities perform fairly crude economic analyses. In New Zealand (1) the benefit/cost ratio of upgrading, taken over the analysis period, is used as a basis of evaluation. Vehicle operating costs are based on Winfrey (2), and maintenance costs are computed as an annual average. In Spain (3) the increased speed, and thus the greater productivity of trucks, is used as a basis for evaluation.

The World Bank recognized these problems and applied an economic analysis to requests for funds by using the highway design and maintenance standards model (HDM) (4). In HDM the road user cost predictions are based on results obtained in the Kenya study (5), and the paved and unpaved road deterioration relations are based on the American Association of State Highway Officials (AASHO) road test (6) and Kenya results (7). The HDM is a highly sophisticated program that requires a great deal of information to run.

The aim of this paper is to investigate warrants for placing a bituminous surfacing on gravel roads by using a comprehensive economic analysis in a program developed to evaluate the interaction of the different cost components. Obviously, the warrants are linked to the costs of constructing and maintaining both road types and the road user costs associated with the road condition.

The analysis is based on the study executed in Brazil, as a follow-up to the Kenya study, by the Brazilian government and the United Nations Development Program (8,9). This study was designed to extend the results of the Kenya study over a wider range of factors. In the investigation of warrants for paving an unpaved road, it is necessary to know the interaction of pavement construction and maintenance and road user costs for both types of road. A systems approach for the evaluation of unpaved roads was developed by Visser (10) that also contains details of specific computations and a computer program. The results obtained (10) are used as a basis for the arguments in this paper. The paved road deterioration relations used are those presented by Visser and others (11).

In this paper an exposition is given of the method of analysis and factors that influence the costs on unpaved roads so that the factors that have the most influence on the final outcome can be studied in greater detail for the development of warrants. Different methods of conducting an economic analysis are discussed, and warrants for paving are developed for each method. The lack of maintenance funds in developing countries is a factor that needs to be taken into account, and the influence of this on the warrants is investigated. Finally, aspects such as dust, accidents, and the depletion of gravel resources are dealt with.

ANALYSIS OF COSTS ON UNPAVED ROADS

A systems approach was used to develop a methodology (10), termed the maintenance and design system (MDS), to evaluate the interaction of road maintenance and the resultant road condition on road user costs. Road user costs include maintenance, interest, and depreciation, and tire, fuel, and oil costs. The MDS also has the capability of making optimum use of maintenance strategies, in terms of number of bladings per year, and regravelling frequency based on the minimization of the total cost of construction, maintenance, and road user costs over the analysis period. The consequences of road maintenance can thus be determined. All costs are discounted by using a discount rate that is a shadow rate of interest by which future monetary values are expressed in current terms. At present a value of 10 percent is recommended for use in South Africa.

Visser showed that the main factors that affect the regravelling strategy were traffic-related, namely ADT and the traffic growth rate (10). In addition, the total discounted regravelling cost is affected by the unit cost of gravel, the analysis period, the discount rate, and the salvage value of gravel that remains at the end of the analysis period. Factors such as road geometry and gravel quality have a minor effect. Blading strategy was found to be mainly dependent on ADT (blading frequency was linked to traffic growth) and gravel quality. The total transport cost was consequently found to be mainly influenced by traffic factors, road geometry, analysis period, and discount rate. Gravel quality was important for roads that carry more than 150 vehicles/day. Road geometry becomes important because it has a major effect on vehicle operating costs. The above-mentioned factors that are sensitive to the outcome of the costs on unpaved roads will be treated in greater detail in the development of warrants for paving.

Methods of Economic Analysis

Two criteria that are customarily used in analyses to evaluate the economic feasibility of improvements (4) are the break-even traffic volume and the first-year benefits criterion.

The break-even traffic volume is defined as the base year ADT at which the net present value of paving just equals the costs of leaving the road in the current gravel status. If the base year ADT exceeds the break-even traffic volume, the relevant internal rate of return will be greater than the discount rate used in the net present value computation, and paving is therefore economically justified. This approach assumes that the result over the analysis period is all that needs to be justified, and no consideration is given to the subsidization of losses or smaller benefits that may occur early in the life of the project by profits or larger benefits that may accrue in later years.

The preferred decision rule is the first-year benefits criterion; i.e., the road should be paved when the ratio of the net first-year benefits to the construction cost equals the relevant interest rate or opportunity cost of capital. In a situation where the budget is strictly limited, the relevant interest rate is in fact the return on the marginal highway project that must be foregone. This may be several times the market rate of interest. Road paving projects ordinarily meet the conditions that are needed to ensure that the first-year rule will yield the correct solution.

Information Used in Analysis

The unit costs used in the analysis were those in force in Texas in 1980, shown in Figure 1. Some of the user cost relations (12,13) from the Brazil study were adjusted for use in MDS to reflect vehicle use rates and relative costs of the different components, as suggested by the American Association of State Highway and Transportation Officials (AASHTO) Red Book on user benefit analysis (14). Regravelling and paving unit costs were deduced from contract rates received by the Texas Highway Department in 1980.

The relative proportion of the four vehicle classes shown in Figure 1 was maintained for each of the traffic levels. A good lateritic gravel, whose properties are shown in Figure 1, was used as a wearing course material for the evaluations.

Geometry along a rolling terrain, which is quantified in Figure 1, was selected as a representative geometry for low-volume roads in developing countries.

For the paved road analysis the traffic and road geometry characteristics were maintained as for the unpaved road. A standard pavement structure over the 15 California bearing ratio (CBR) subgrade was selected, which consisted of 150-mm gravel subbase, 150-mm graded crushed stone or gravel base, and a single surface treatment surfacing. According to the draft Technical Recommendations for Highways (TRH) 4 (15), this structure can accommodate up to 3 million equivalent 80-kN axle loads, which is the range normally found on lightly trafficked roads, such as those under consideration.

The deterioration relations that were used in the evaluation of the paved road performance were developed elsewhere (11) and are shown in Figure 2. The change in roughness is determined mainly by repairs that are made to the road, such as the filling of potholes or patching. For the purpose of the exercise, the need for repairs was assumed. Resealing of the road closes the cracks in the road surface and thus retards further deterioration to wider cracks and ultimately reduces or delays the need for repairs, hence the plateau in the curve showing the development of repairs. A reseal, furthermore, does not have the capability of restoring the riding quality, and thus the effect of repairs prior to the reseal remains. The roughness profile is used to compute road user costs on the paved road.

INVESTIGATION OF WARRANTS FOR PAVING

The ensuing discussion is based on the results obtained with the unit costs shown in Figure 1. Different unit costs may lead to different results, although the general conclusions will probably remain similar. The analysis of the unpaved road was based on the premise that the road remains trafficable at all times. This therefore excluded earth roads, which are adequate for traffic volumes only up to about 40 vehicles/day and can be closed during periods of heavy rainfall. Traffic volumes investigated ranged from 50 to 500 vehicles/day.

Break-Even Traffic Volume

The total cost concept, which consists of the sum of the costs of construction, maintenance, and those incurred by the road user, is used. The break-even traffic volume is the volume that prevails in the first analysis year when the total cost on an unpaved road equals the total cost on a paved road. Paved road construction and maintenance costs can vary widely from region to region, and these were therefore kept as a dependent variable in the evaluation; this variable then equals the total discounted cost on the unpaved road minus the discounted user cost on the paved road. The discounted paved road construction and maintenance costs as a function of the ADT during the first analysis year are shown in Figure 3 for different analysis periods.

The paved road construction costs only include the pavement layers above subgrade to make the analysis comparable to that for unpaved roads. A reasonable discounted construction and maintenance cost over 20 years for the selected paved road structure is about \$50 000/km based on 1980 contract prices obtained from the Texas Highway Department (10). In Figure 3 the break-even traffic volume is thus 400 vehicles/day for the unit costs employed. The break-even traffic volume for alternative discounted construction and maintenance costs can be read from Figure 2.

Often the analysis period used for gravel roads is less than the 20-year period applied to paved roads because of the tremendous uncertainty about traffic volumes on newly developed roads. Because the design life of a paved road is usually about 20 years, the value of the pavement at the end of shorter analysis periods needs to be considered. The salvage value of a paved road is fairly difficult to compute. Two approaches can be used (<u>15</u>):

1. The residual structural salvage value is the saving in cost of constructing a new pavement as opposed to placing an overlay on the existing pavement to bring it to the same condition as the new pavement and to ensure that it can carry the same traffic as the new pavement to the same terminal condition.

2. The salvage value of recycled layers is the difference in cost between furnishing new materials and the cost of taking up and recycling old materials.

Both these salvage value calculations are uncertain, and therefore, for the purpose of the analysis, the outcome will be generalized in terms of relative costs. The salvage value for the gravel roads is taken as the cost of placing the remaining wearing course material. In Figure 3 it is obvious that the

Figure 1. Information used in analysis.

```
BASIC DESIGN INFORMATION
************************

 LENGTH OF THE ANALYSIS PERIOD (YEARS)                            10.0
 LENGTH OF ROAD LINK (KM)                                         10.0
 MINIMUM TIME BETWEEN REGRAVELLINGS (YEARS)                        1.0
 REGRAVELLING STRATEGY                                            SPOT
 SALVAGE VALUE OF GRAVEL SURFACING (PERCENT)                       100
 DISCOUNT RATE OR TIME VALUE OF MONEY (PERCENT)                   10.0
 ROAD WIDTH (M)                                                   10.0
 LENGTH OF DRY SEASON (DAYS)                                       210

TRAFFIC DATA
************

 AVERAGE DAILY CAR TRAFFIC AT START OF AN PERIOD                   300
 AVERAGE DAILY BUS TRAFFIC AT START OF AN.PERIOD                    10
 AVERAGE DAILY MEDIUM TRUCK TRAFFIC AT START OF AN PERD            100
 AVERAGE DAILY HEAVY TRUCK TRAFFIC AT START OF AN PERD              90
 ANNUAL GROWTH RATE OF CAR TRAFFIC (PERCENT)                       5.0
 ANNUAL GROWTH RATE OF BUS AND TRUCK TRAFFIC (PERCENT)             5.0

MATERIAL INFORMATION
********************

 PERCENTAGE OF SURFACING MATERIAL PASSING 0.074 MM SIEVE            15
 PLASTICITY INDEX OF SURFACING MATERIAL (PERCENT)                    5
 SOAKED CBR OF GRAVEL SURFACING (PERCENT)                           80
 SOAKED CBR OF INSITU ROADBED (PERCENT)                             15
 TYPE OF SURFACING TO BE EVALUATED                                GRAV
 SURFACING MATERIAL TYPE                                          LATR
 IS ROAD WITHOUT SURFACING PERMITTED TO BECOME IMPASSIBLE

CONSTRUCTION INFORMATION
************************

 AVERAGE GRAVEL THICKNESS AT START OF AN.PERIOD (MM)                 0
 MINIMUM ALLOWABLE THICKNESS OF ADDED GRAVEL (MM)                   75
 ROUGHNESS AT START OF ANALYSIS PERIOD (QI*)                        50

COST INFORMATION
****************

 COST OF GRAVEL SPREAD,DLRS/M3                                   12.00
 COST OF MOVING REGRAVELLING EQUIPMENT TO LINK,DLRS                  0
 COST OF NEW CAR,DLRS                                             6500
 COST OF NEW BUS,DLRS                                           135000
 COST OF NEW MEDIUM TRUCK,DLRS                                   19000
 COST OF NEW HEAVY TRUCK,DLRS                                    40000
 COST OF GASOLINE,DLRS PER LITER                                   .28
 COST OF DIESEL FUEL,DLRS PER LITER                                .27
 COST PER CAR TIRE,DLRS                                             50
 COST PER BUS OR TRUCK TIRE,DLRS                                   250
 DAILY COST TO OPERATE MOTOR-GRADER,DLRS                           200

ROAD GEOMETRY
*************
 PERCENTAGE OF LENGTH OF LINK IN EACH CATEGORY
                                 GRADE (PERCENT)
   CURVATURE        *  0-2  *  2-4  *  4-6  *  6-8  *  8-10 *
TANG AND RAD>400M   *  50.  *  10.  *  10.  *   0.  *   0.  *
400M>RADIUS>200M    *  10.  *  10.  *  10.  *   0.  *   0.  *
200M>RADIUS>100M    *   0.  *   0.  *   0.  *   0.  *   0.  *
RADIUS<100M         *   0.  *   0.  *   0.  *   0.  *   0.  *

PRINTOUT CONTROLS
****************

 IS PRINTOUT OF REGRAVELLING STRATEGIES REQUIRED                     N
 IS PRINTOUT OF ROUGHNESS STRATEGIES REQUIRED                        Y

COMPUTED INFORMATION
********************

 MINIMUM ALLOWABLE GRAVEL THICKNESS (MM)                            49
```

Figure 2. Deterioration relations on selected paved road.

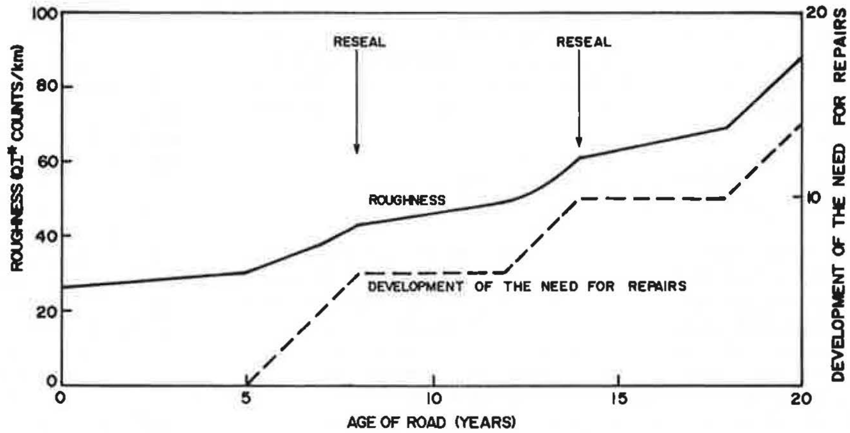
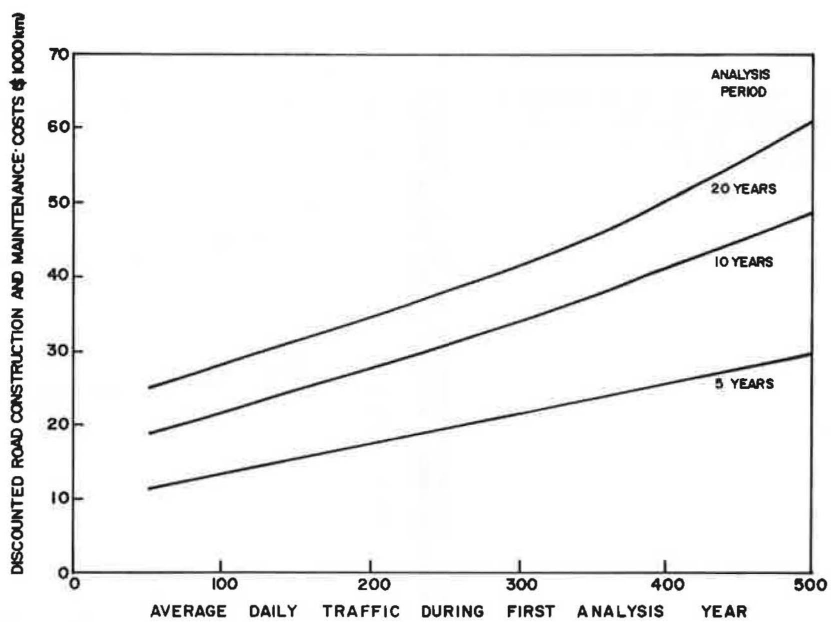


Figure 3. Discounted paved road construction and maintenance cost for a total cost equal to that on unpaved roads.



same break-even traffic volume will be obtained for a shorter analysis period provided that the discounted road construction and maintenance cost minus the discounted salvage value bears some relation to this cost over a 20-year analysis period, shown in Figure 4. The ratios for the 5- and 10-year analysis periods are fairly constant and are approximately 2.0 and 1.25, respectively. Thus, if the computed salvage value is such that the discounted construction and maintenance minus salvage cost over the 5-year period is less than half that of the 20-year period, then the break-even traffic volume will be lower than that computed from the 20-year period. The converse is true if the ratio is greater than half.

Before the decision to use a 20-year or shorter analysis period can be made, the decisionmaker needs to weigh the uncertainty in traffic over a 20-year analysis period against the uncertainties in computing the salvage value at intermediate points in the life of a pavement structure. The use of a 20-year analysis period would be recommended for most cases.

First-Year Benefit Criterion

The road should be paved when the ratio of the net first-year benefits (i.e., savings in road user costs and road maintenance costs) to the additional construction cost of a paved road over an unpaved road equals the relevant interest rate. For the analysis the unpaved road maintenance cost used was that found to give the least total cost. Figure 5 shows the plot of the net first-year benefits as a function of ADT. The ratio of the net first-year benefits to the paved road construction cost of $30 000 and $40 000/km, minus the unpaved road construction cost of $12 000, is also shown. A construction cost of about $40 000/km is approximately equivalent to the discounted construction and maintenance cost of $50 000/km used in the break-even traffic example. The ADT at which a paved road should be constructed for a discount rate of 10 percent is approximately 575 vehicles/day, which is almost 50 percent greater than the break-even traffic volume. This result is similar to that reported by Harral and others (4). It illustrates the extent to which larger benefits later in the life of the pavement subsidize earlier smaller benefits in the break-even traffic volume calculations.

Although the first-year benefit criterion is the preferred one, care should be taken in its application. A cheap paved road structure may be designed that may last a very short period and then require extensive maintenance and even rehabilitation. Stage construction may be considered to fall into this category. In such a case the discounted rehabilitation cost or cost of excessive maintenance that occurs after the first year should be added to the construction cost to give a more realistic assessment.

IMPACT OF REDUCED MAINTENANCE FUNDS ON THE WARRANTS

One of the major concerns about the establishment of a road infrastucture is the effect that a reduction of the maintenance budget could have on the optimally selected pavement design. Frequently the construction of roads in developing countries is financed by international loans or grants; however, maintenance is funded by the government from internal sources through the treasury department. A reduction in funds available to the treasury frequently results in a cut in the maintenance budget. The influence of such a cut on the warrants for paving thus needs to be investigated.

The unpaved road analysis was constrained to provide access throughout the year. A cut in the maintenance budget could therefore not affect the regravelling strategy, and its result would only influence routine blading. Within the policy constraints evaluated, a complete cut in the funds available for maintenance is infeasible, because the road would become impassible within one to two years, depending on the traffic. The influence of reducing by half the funds available for blading on the discounted paved road costs is illustrated in Figure 6. The break-even traffic approach is used because road engineers are better able to relate to costs than to benefits or discount rates. To convert to the first-year benefit rule, the break-even traffic volume should be increased by half; for example, an ADT of 400 becomes 600 for the first-year benefit rule. The reduction of available routine

Figure 4. Ratio of discounted maintenance and construction cost of 20-year analysis period relative to other analysis periods to give same break-even traffic volume.

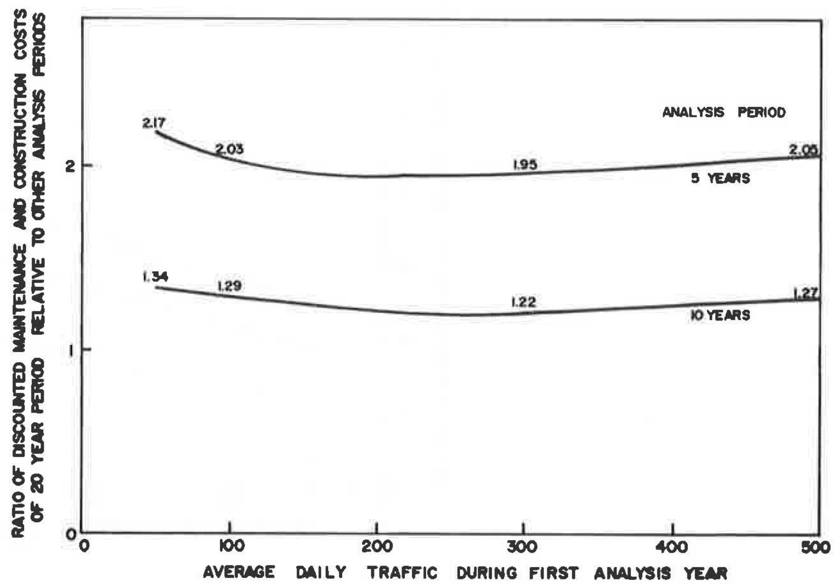
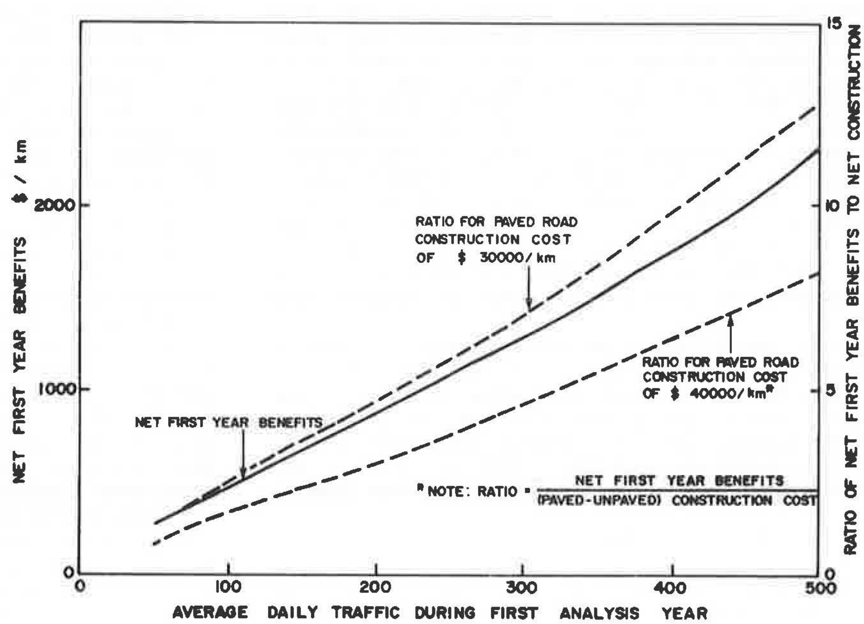


Figure 5. Results of computations according to first-year benefit criterion.



maintenance funds by half has a minimal influence on the warrants for paving. This is because the routine maintenance cost is a relatively small part of the total cost, and the slope of the total cost curve in the proximity of the least total cost blading frequency is flat. However, for one or two bladings per year, the curve is very steep, as was shown earlier (10).

The likelihood of a cut in the routine maintenance budget will also affect the paved road alternative. The filling of potholes and repairs usually have a high priority, and therefore, a cut in the budget would probably result in the deferment of a reseal. In Figure 2, if the reseal due after eight years is deferred and the need for repairs continues at the same rate, then an expensive rehabilitation or overlay may be necessary after about 12 years. A paved road may therefore be more sensitive to reductions in the maintenance budget than unpaved roads under the system of constraints analyzed.

In cases of extensive cutbacks of the maintenance funds, the system of constraints under which the road system is maintained may be altered. Passability throughout the year may be eliminated, and this would then have the effect of a reduced regravelling expenditure, since the reversion of a gravel road to an earth road would be feasible. Figure 7 illustrates the relative cost components of the total cost. In those cases in which the road is permitted to revert to an earth road, a careful economic analysis is necessary to evaluate the impacts of the reduced regravelling expenditure on additional road maintenance and delay costs. Such an approach would probably not be economically feasible, and an analysis could provide data to back up technical arguments.

EFFECTS OF REGRAVELLING COSTS ON WARRANTS FOR PAVING

At the optimal blading frequency the average road roughness of an unpaved road is similar to the average roughness of a paved road during its service

Figure 6. Discounted paved road construction and maintenance costs for different unpaved road blading budgets.

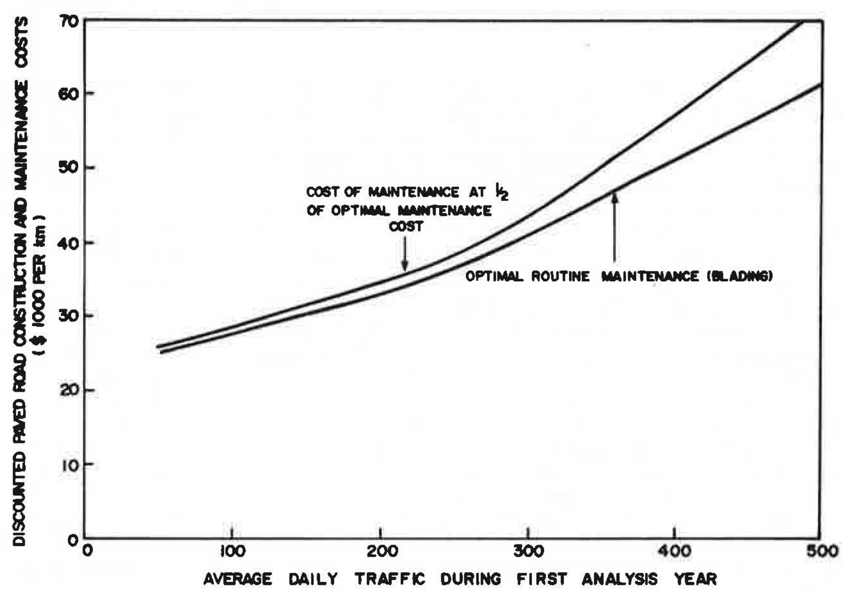


Figure 7. Illustration of relative costs on unpaved roads.

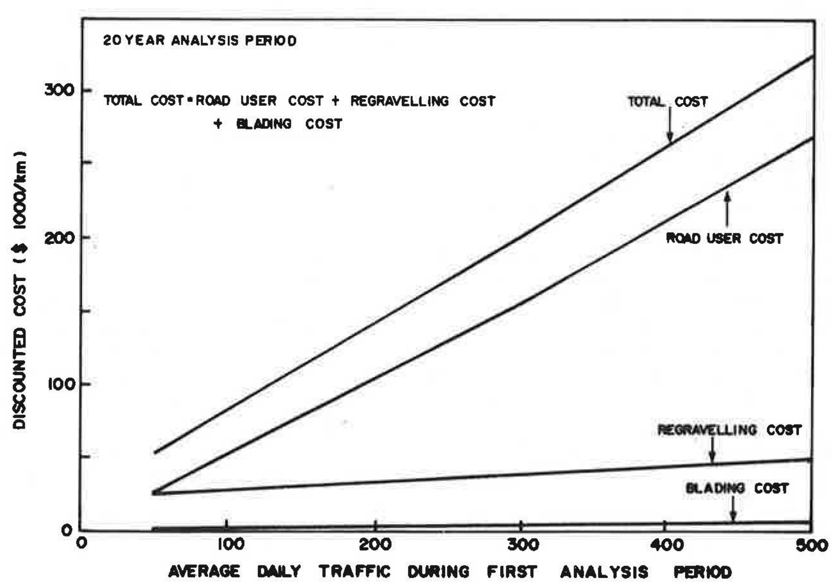
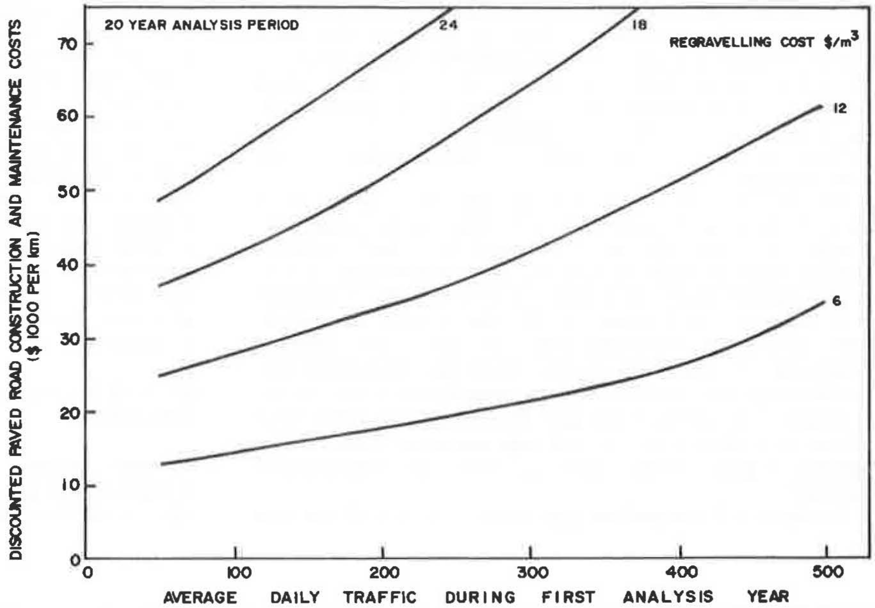


Figure 8. Discounted paved road construction and maintenance cost for different unit costs of gravel.



life based on observations in Brazil (e.g., average roughness of optimal blading for 500 ADT is a QI of 47 for the good laterite gravel over a 20-year analysis period) (10). Consequently, the discounted road user costs during a 20-year period are similar for the two road types, unlike results reported by Winfrey (2). However, over the short term the user costs on a new paved road are lower than on an unpaved road and follow the results presented by Winfrey. Broadly speaking, the main determinant for paving is therefore whether the discounted paved road maintenance and construction cost is less than the discounted regravelling and blading cost. In Figure 7, the regravelling and blading costs are approximately equal to the paved road cost of $50 000/km used previously in the ADT range of 400-500 vehicles/day. The regravelling cost thus has an important impact on the warrants for paving.

The discounted paved road construction and maintenance costs for different unit costs of gravel are shown in Figure 8. In the previous evaluations a regravelling cost of $\$12/m^3$ was used. At current transportation costs, if the gravel had to be transported an additional 30 km, then the unit cost would increase from $12 to $\$18/m^3$. If the paved road construction cost remained at $50 000/km, then the break-even traffic volume would reduce to about 200 vehicles/day, or about 300 vehicles/day according to the first-year benefit rule. Conversely, if the gravel wearing course material were abundantly available alongside the road, as reflected by a cost of $\$6/m^3$, then paving would not be economically desirable until a traffic volume greater than about 600 or more vehicles/day is reached.

The cost of constructing a paved road is usually related to the unit cost of gravel wearing course material, thus a greater unit cost may not necessarily lead to a reduction in the traffic volume for paving because of increased paving costs. In some cases this rule may not hold. In the Orange Free State, in South Africa for example, a decomposed dolerite is frequently used as a gravel wearing course. Depending on the state of decomposition, hard rocks of unweathered parent material about 100-200 mm in diameter are found. The use of this material as a gravel wearing course results in frequent regravelling because the fine material on the surface is abraded away between the rocks to leave a very rough surface. Blading is then ineffective in reducing the roughness. This type of material is adequate for use in a paved road and is frequently abundantly available. Better quality material is often transported over 50 km or more; therefore, the construction cost of a paved road is not linked to the cost of the better quality wearing course gravel material. A similar situation exists in Brazil, where the fine-grained coffee soils in the states of Sao Paulo and Parana become impassible during the rainy season. However, when this material is used in a paved road structure and if they can be kept dry, the road performs adequately.

These results show that it is not feasible to generalize warrants for paving and that every case should be evaluated individually. As a rough guideline, the traffic volume at which paving should occur is about 50 percent higher than the traffic volume at which the paved road construction and maintenance costs over a 20-year period equal the discounted regravelling and blading costs.

DUST, ACCIDENTS, AND INTANGIBLE INFLUENCES

In the foregoing analysis, quantifiable aspects of construction, maintenance, and user costs were included. However, certain aspects, which may hold important implications, have not been quantified.

Dust

Dust is often considered as a nuisance, but it is difficult to express comfort in monetary terms, because road users are not willing to pay for this sort of comfort. The effect of dust on crops and livestock has been mentioned, but apparently little has been quantified. For example, in cotton-growing areas gravel roads are sometimes paved because the dust affects the cleanliness of the cotton, which may then render it unacceptable. The influence of dust on crops and livestock is an aspect that requires further attention; if the impact is significant, it could greatly reduce the traffic volume at which a road should be paved.

Accidents

Zaniewski and others reached the conclusion that there is no reliable information for making distinctions in accident rates among pavement types and conditions (16). Other researchers (17,18) have shown that accident costs on low-volume roads are an inconsequential part of the total cost of road use. A global estimate of accident rates on two-lane paved and unpaved roads in South Africa showed that the rate for unpaved roads is about 0.63/million vehicle km, which is about four times lower than that for paved roads. This is ascribed to greater concentration by the driver at traffic volumes where traffic interference is minimal. It is thus possible that if a gravel road is paved at low traffic volumes the accident rate would increase, thus reducing the benefits of paving.

Exhaustion of Natural Resources

An aspect of gravel roads that is receiving increasing attention is the depletion of gravel. Initially, the best wearing course gravel is used, and as regravelling becomes necessary, material needs to be obtained from further afield. When the road requires paving, the gravel materials then have to be transported over long distances. This situation ties in with the findings of maintenance personnel, who have shown that the heaviest traffic on gravel roads has often been associated with those roads that have the worst materials.

A utility analysis, which ranks different alternatives in terms of subjectively assigned weights for unquantifiable and quantifiable factors, may be a method in which the depletion of natural resources can be considered. The effect of dust on comfort could then be evaluated in the same manner.

CONCLUSIONS

In the evaluation of warrants the preferred method of analysis, namely the first-year benefit rule, gave traffic volumes for paving that were about 50 percent larger than the break-even traffic volume that is frequently used. For the cost data obtained in Texas, the traffic volume at which a gravel road would be paved was about 575 vehicles/day. This figure is considerably larger than the 100 and 250 vehicles/day figure that is often actually used in different parts of the world. The difference in the predicted volume compared with warrants actually used may be attributable to a heavy weighting being given subconsciously to nonquantifiable aspects such as dust, comfort, and depletion of natural resources. Another factor may be the difficulty of maintaining regular bladings weekly or biweekly over a long period of time.

Based on observations in Brazil, the average roughness of a gravel road that is optimally maintained is similar to that of a paved road during its life. Consequently, road user costs are similar on paved and unpaved roads. The major implication for paving is thus the difference in discounted paved road construction and maintenance cost and the unpaved road regravelling and blading cost. The latter costs are primarily a function of the unit cost of gravel spread and compacted. It follows that each road should be evaluated individually and that generalizations may result in uneconomical practices because of the peculiarities of gravel cost that are usually unique for any particular road.

The impact of aspects such as dust, accidents, and depletion of natural resources has not yet been adequately quantified for inclusion in an economic analysis. These aspects are best evaluated, together with the results from an economic analysis, in a utility analysis.

Under the system of constraints in which the unpaved roads were analyzed, the reduction by half of the funds available for blading, compared with the optimal funding, had a minor effect on the warrant for paving. A complete reduction, however, would result in the road becoming impassable within 1-2 years. These results do not mean that blading funds should be reduced because it is likely that many organizations operate at the reduced budget level. A reduction in maintenance funds appears to have a greater impact on paved roads than on unpaved roads within the system of constraints used. Paving therefore does not seem to be the recommended procedure under conditions of uncertainty with regard to the availability of funds for maintenance.

RECOMMENDATIONS

The paper has illustrated the value of a detailed analysis of both economic and unquantifiable factors. The philosophy and procedure should be applied when decisions have to be made regarding the paving of unpaved roads and in the development of a new infrastructure in regions that were previously economically inactive.

The recommended procedure for determining the traffic level at which an unpaved road should be paved is use of the first-year benefits criterion. A special computer program, MDS, was developed to facilitate determination of the unpaved road design and maintenance strategy with the lowest total cost. The MDS program can be run on a desk-top computer and is thus readily available to most practitioners. An adapted version of this program was used to compute the user costs on paved roads. To run these programs, unit costs of materials and their characteristics, and other costs specific to the region are needed.

Care should be taken in applying the first-year benefits rule to those paved road designs in which the initial construction costs are deferred, such as for stage construction, and it is recommended that later costs of a special nature should be included as discounted construction costs.

REFERENCES

1. B. Cox. Can You Afford to Seal That Road? Road Research Unit, National Road Board, Wellington, New Zealand, Newsletter, Nov. 1980, p. 16.
2. R. Winfrey. Economic Analysis for Highways. International Textbook Company, Scranton, PA, 1969.
3. J. Losa Hernandez. Caminos Economicos--Proyecto, Construción, Conservación. Ediciones Mundi-Prensa, Madrid, 1979.
4. C.G. Harral, T. Watanatada, and P. Fossberg. The Highway Design and Maintenance Standard Model (HDM): Model Structure, Empirical Foundations, and Applications. Paper presented at the Second International Conference on Low Volume Roads, Ames, IA, Aug. 21, 1979.
5. H. Hide, S.W. Abanayaka, I. Sayer, and R.J. Wyatt. The Kenya Road Transport Cost Study: Research on Vehicle Operating Costs. Transport and Road Research Loaboratory, Crowthorne, Berkshire, England, Lab. Rept. 672, 1975.
6. The AASHO Road Test--Report 5, Pavement Research. HRB, Special Rept. 61E, 1962.
7. J.W. Hodges, J. Rolt, and T.E. Jones. The Kenya Road Transport Cost Study: Research on Road Deterioration. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, Lab. Rept. 673, 1975.
8. Inception Report, Research Concept, and Procedures. Research on the Interrelationships Between Costs of Highway Construction, Mainte-

nance, and Utilization, GEIPOT, UNDP, IBRD, Brasilia, Brazil, April 1976.
9. Midterm Report, Preliminary Results, and Analysis. Research on the Interrelationships Between Costs of Highway Construction, Maintenance and Utilization, GEIPOT, UNDP, IBRD, Brasilia, Brazil, Sept. 1977.
10. A.T. Visser. An Evaluation of Unpaved Road Performance and Maintenance. Univ. of Texas at Austin, Ph.D thesis, 1981.
11. A.T. Visser, C.A.V. Queiroz, F.M. Borralho, and B.K. Moser. Highway Costs Research: Pavement Studies. International Symposium on Pavement Evaluation and Overlay Design, Rio de Janeiro, Sept. 9-14, 1979.
12. R.J. Wyatt, R. Harrison, B.K. Moser, and L.A.P. DeQuadros. The Effect of Road Design and Maintenance on Vehicle Operating Costs--Field Research in Brazil. TRB, Transportation Research Record 702, 1979, pp. 313-319.
13. A. Chesher, R. Harrison, and J.D. Swait. Vehicle Depreciation and Interest Costs: Some Evidence from Brazil. Paper presented at the World Conference on Transport Research, London, England, April 1980.
14. A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements. AASHTO, Washington, DC, 1977.
15. Structural Design of Interurban and Rural Road Pavements. National Institute for Transport and Road Research, Pretoria, South Africa, Draft TRH 4, CSIR, 1980.
16. J.P. Zaniewski, B.C. Butler, G. Cunningham, G.E. Elkins, and R. Machemehl. Vehicle Operating Costs, Fuel Consumption, and Pavement Type and Condition Factors. Federal Highway Administration, Interim Rept., June 1980.
17. C.H. Oglesby and M.J. Altenhofen. Economics of Design Standards for Low-Volume Rural Roads. NCHRP, Rept. 63, 1969.
18. R.S.P. Bonney and R.S.Millard. Planning and Research for Road Transport in Developing Countries. Conference on Civil Engineering Problems Overseas, Tech. Session IV, Institution of Civil Engineers, London, June 1966.

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