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Low-Volume Roads: Second International Conference

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Preface

In its volume, World Road Statistics, the International Road Federation lists more than 17 million km of roads. Approximately 92 percent of them are in the secondary road or "other" category, and a large portion of those that are considered surfaced have either minimum surface treatments or chip seals. New low-volume roads continue to be built in the United States and throughout the world.

The problem is how to design and operate these low-volume roads under constraints such as limited funding, labor-intensive construction and maintenance, inflation, design criteria that may not be appropriate to low-volume roads, stage construction, complex intragovernmental requirements, inappropriate safety requirements, wide-ranging traffic volumes and vehicle loads, environmental considerations (air and water quality, erosion, and landslides), and providing maximum socioeconomic services in route selection.

The Second International Conference on Low-Volume Roads was organized to facilitate the exchange of information on the practical application of engineering principles and current practice in the design, construction, and operation of low-volume roads. The papers in this Record were presented at this conference, which was held at the Scheman Center, Iowa State University, August 20-23, 1979; several informal presentations were also made. Although not all of the above constraints were addressed by the formal papers, informal presentations and panel and other discussions generally covered the entire range of problems.

The conference was sponsored by the Division A Committee on Low-Volume Roads under the direct responsibility of the Conference Steering Committee, whose members are listed on the reverse of the title page of this Record.

The Second International Conference on Low-Volume Roads was partially funded by the Federal Highway Administration and the Agency for International Development. Iowa State University, through its Civil Engineering Faculty, Engineering Extension, and Continuing Education Service, hosted the conference along with the Iowa Association of County Engineers. The following organizations cooperated to make the conference possible:

COSPONSORS
- Agency for International Development
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- U.S. Army Engineer Waterways Experiment Station
- U.S. Forest Service

THE CRAFT OF HIGHWAY ENGINEERING

Ray Millard, The World Bank

There are, it is said, more scientists alive now than have existed in previous generations since man emerged. Similarly with highway engineers. Fifty years ago, there were approximately 15,000 men in the world who would describe themselves as highway engineers; now there are at least ten times that number. And with them is an enormous army of other specialist professional people concerned with roads and road transport; traffic engineers, transport economists, statisticians and mathematicians, planners, medical men and psychologists concerned with road safety, environmentalists, and perhaps the latest specialization to emerge, the professional protester. This expansion reflects, of course, the huge growth of road transport during the present century and the changing attitudes of the public to it.

Over this period, four stages can be distinguished. In the first, the aim was to provide the roads needed for the rapidly expanding numbers of road vehicles, and the primary need for expertise was in road building. It soon became apparent that this great increase in man's mobility had brought with it a new plague, the scourge of road accidents. The science of medicine was developing alongside, and as man learned to control the diseases which afflict him, the toll of road accidents increased to the extent that in North America and Europe, it killed and maimed more active people than any of our more traditional diseases. Stage 2 came with the growing concern for road safety with new forms of professional specialization in which highway engineers were joined by law enforcement officers, statisticians and medical men.

Traffic control measures were first introduced to promote road safety. But soon, it became evident that they were needed for another purpose, to ease the flow of the large numbers of motor vehicles that were by now crowding on to the roads, particularly in towns and cities. By this time, road building had become a major item of capital expenditure in most countries, and there was a need to determine priorities and standards for road building in a way which was manifestly logical and apparently fair. The third branch of the profession to sprout produced the transport economists and planners with a new armory of expertise, benefit-cost analysis, origin and destination surveys, mathematical modelling, economic and physical planning, and so on.

Then, in the last two decades has come the fourth stage, a growing concern about the impact of road transport on man's living environment. New roads had been driven through our towns and cities breaking up the existing patterns of community life, and road traffic had joined industry as a major pollutant of the air we breathe. The din from road traffic afflicted the lives, the work and the sleep of urban dwellers. In rural areas, new roads penetrated into areas of natural beauty, and the road builders were branded as the despoilers of the countryside. To these deleterious effects of road transport has been added concern at our profligate use of the world's energy resources.

It was natural that these developments should first become manifest in North America, and it follows that the techniques for coping with them generally first emerged in the USA. At this conference, our concern is with low volume roads. So our emphasis on those four stages is somewhat different from the attitude of say the urban planner, or the U.S. Federal Highway Administration. With my colleagues from the World Bank, I have a special concern with roads in the developing countries of Africa, Asia and South America. Our overall aim is to build up the resources of developing countries so that they can take their place fully fledged socially and economically in the comity of nations. Part of these resources lies in the stock of professional people who are able to cope with the planning, building, and maintenance of their road networks. And this is the theme I want to take for this paper. It is a huge theme, impossible to cope with comprehensively in one short paper. So I shall concentrate on three particular aspects and on these I shall be dealing with where I believe that things have gone wrong. The three aspects are:

1. Soil mechanics as applied to highway engineering;

2. Bituminous road surfacings;

3. Economic evaluation of highway projects.

I shall be primarily concerned with roads in developing countries, but I believe that much of what I have to say will be relevant to those of you who are concerned with lightly constructed roads here in North America.

The first and obvious comment is that the highway engineers from developing countries coming to America and to Europe for their academic training, have found much of what they have been taught irrelevant to their highway problems at home. Behind this lies a more basic problem--the deep rift between the "scientific" and the practical approach to highway engineering.

The hope was that if our graduates in training could be instructed on the scientific principles which lie behind highway engineering practice, they could return to their own countries and apply these principles to the solution of technical problems in their own countries. This hope has proved illusory. Some, but not many useful scientific principles have emerged.

The body of knowledge in highway engineering remains empiric rather than rigorously scientific. So, the knowledge taught in our engineering colleges is generally derived from a synthesis of local experience. No wonder it is often irrelevant and sometimes downright misleading in other parts of the world.

This is very obvious in the use to which soil mechanics has been put in highway engineering, and I want to illustrate it with a story. A major landslide in an East Asian country had carried away half a mile of road and part of a village. A huge crowd had gathered to decide what to do, including the Minister of Public Works (himself an engineer), the Chief Highway Engineer and many of his technical staff. They were all charging around discussing where to rebuild the road and the houses. Not one of them was asking why the landslide occurred. No one was saying, "Let's find out what happened and why. Then when we rebuild, we can be reasonably sure it won't happen again." This was a failure in comprehension. Despite their training as engineers (many of them in America and Europe), they did not comprehend that it was possible to find out what had happened. Still less did they comprehend that it would be useful to do so. Their engineering courses had dealt with slip-circle analysis and with drained and undrained tri-axial tests, but they didn't see the relevance. And they had had no instruction on the soils of their own country, on their properties and their engineering defects and uses. This was bad ground. It had always been bad ground. They would rebuild the road and perhaps this time, with luck, it would stay there.

Science had produced an understanding of some of the mechanisms of landslides, but in those engineers' minds, this had no connection with real life. This illustrates very clearly our dilemma. The scientific approach involves setting out to acquire a basic understanding of why things happen as they do, of the stability of slopes, of the stresses and strains in road pavements under traffic. This is proving to be a long and tedious business; witness the tremendous research effort still going into establishing a rigorously scientific method of pavement design. The practical people, impatient to produce technical answers to the problem which beset them, produced ad hoc tests which aimed to simulate in an approximate way, the reactions between the loaded vehicle and the road structure. In America, the crop of such tests was formidible: R. R. Proctor and his compaction test, later followed by the heavier Modified AASHO Compaction Test; Plate Bearing Tests and later the California Bearing Ratio Test originated by O. J. Porter then with the California State Highway Department; Benkelman in the Bureau of Public Roads with his deflection beam, and in the field of bituminous materials; Bruce Marshall of the Mississippi State Highway Department, Prevost Hubbard and F. C. Field of the Asphalt Institute; and Francis Hveem of the California State Highway Department, all produced mechanical testing regimes for the design of asphaltic concrete mixtures. And there were others perhaps less notable in the Highway Engineers Hall of Fame. Very little comparable came from Europe over this period. The American dominance does not only derive from the earlier motorization of America. It springs also from something deep in the American culture, an innate optimism expressed in the belief that it is possible to produce simple, mechanistic models of natural phenomena and a determination to produce an answer which could be adopted for rapid industrial use. Some waste might be implied by the approximations that were necessary, but this was of no consequence in a society dedicated to technical innovation and change.

The ad hoc approach produced immediately useful solutions in the areas where they were developed. The physical tests they used were extensions of the eye and hand of experienced men. They were, in effect, a means to make this experience numerate, and their value depended a great deal on the skill with which the test results were correlated with road behavior in the area in which they are being used.

There is an intrinsic danger in this approach; the correlations with road behavior are necessarily local in character. When one moves away into another environment, for example into a different climate or to use different road making materials, the correlation disappears and the test results can be quite misleading.

The CBR test provides a graphic example. As originally conceived, its main use was intended for testing road making gravels. A good road making gravel would have a CBR of 100%. It was the U.S. Corps of Engineers which pushed the test as a means of evaluating subgrade soils. They had a wet environment in mind, and it was they who decreed that the test should be done on compacted samples which had been soaked in water for four days. Their aim, of course, was to design for the worst conditions they were normally likely to encounter. And they can have had little idea of the confusion that their edict would have. I have met, many times, engineers working in arid parts of Africa and other areas of the world, who were solemnly soaking CBR specimens for four days and declaring that the resultant CBR value indicated the soil strength which should be used in designing their pavements. That provides an example of experience being translated from one part of the world to another with wasteful effects.

Fortunately, this subject provides an example of science and practice combining to produce a sound engineering answer. The strength of a subgrade soil depends not only on the nature of the soil; it depends also on the state to which it has been compacted, and above all, it depends on the prevailing moisture conditions in the soil.

The state of compaction achievable is a matter to be judged from local engineering experience. Prevailing moisture conditions may also be determined from local engineering experience. But science has gone one better. Experts in physics and climatology have joined to produce a theoretical basis for determining the critical moisture conditions in soils under sealed surfaces over the range of physical and climatic conditions encountered in different parts of the world. And, important to us as engineers, this theory has been tested by field observations and found to be correct. Some engineers may have sufficient curiosity to want to explore the theory. But for most of us, it will be sufficient to see the results incorporated in design recommendations we can easily understand. An example of this is shown in

Table 1. This table, incidentally, illustrates a trend which is likely to continue, to use the CBR value as an index rather than a directly measured entity. Indeed, in some parts of the world, the trend is to delineate design CBR values on a basis of soil identification, together with a knowledge of the achievable state of compaction and the prevailing moisture conditions at the site concerned.

Table 1. Estimated minimum design CBR values under paved roads for subgrades compacted to 95 per cent of Proctor maximum dry density.

Depth of water table[a] from formation level	Minimum CBR (per cent)[b]					
	Non-plastic sand	Sandy clay Pl = 10	Sandy clay Pl = 20	Silty clay Pl = 30	Heavy clay Pl ≥ 40	Silt
0.6m (2 ft)	8	5	4	3	2	1
1.0m (3.3 ft)	25	6	5	4	3	2
1.5m (4.9 ft)	25	8	6	5	3	
2.0m (6.5 ft)	25	8	7	5	3	
2.5m (8.2 ft)	25	8	8	6	4	See
3.0m (9.8 ft)	25	25	8	7	4	Note
3.5m (11.5 ft)	25	25	8	8	4	
5.0m (16.4 ft)	25	25	8	8	5	3
7.0m (23 ft) or more	25	25	8	8	7	

Notes:

1. With structured clays, such as the red coffee soils of East Africa, laboratory CBR tests should be undertaken whenever possible. Soils of this type can be identified by the fact that their plasticity, as indicated by the Atterberg limits, tends to increase when the soil is worked and its structure is broken down. If CBR tests cannot be undertaken, an approximate estimate of the effective subgrade CBR for this soil type will be obtained by using the values quoted in the Table for sandy clays (Pl = 20 per cent).
2. This Table cannot be used for soils containing appreciable amounts of mica or organic matter. Such soils can usually be identified visually.
3. Laboratory CBR tests are required for pure silt subgrades with water tables deeper than 1.0m (3.3 ft).

[a]The highest seasonal level attained by the water table should be taken.

[b]This table is abridged from Road Note 31 (Third Edition), "A guide to the structural design of bitumen-surfaced roads in tropical and subtropical countries," (HMSO 1977), and these CBR values are for use with the design chart in that Road Note.

Poorly consolidated soils present special problems to the road builder. They are usually transported soils and are commonly found in river deltas. They occur in the lower reaches of the Mississippi, along the coast of West Africa, in the delta areas of the great Indian rivers, in the rice plains of Thailand, in Malaysia and Indonesia and in many other parts of the world. They are generally saturated, i.e., all the voids in the soil are filled with water. A good basis for the scientific approach to road building over these soils was provided by the consolidation theory developed by Terzaghi. Again, the theory is somewhat complex and will be examined only by engineers who are scientifically curious. But its application is relatively simple; the amount and rate of consolidation of these soils under load is determined by using laboratory consolidation tests on soil samples. As load is applied to the soils, pressure is generated in the water. If the load is applied too quickly, these pressures become excessive and shear failures will occur. But under a controlled load, the pressures are gradually dissipated as water moves away into unloaded areas and the soil is compressed to an ultimate value at which the load is carried by the soil particles. The theory and the associated laboratory test provided a reliable means of determining ultimate settlement under load, and hence, of designing embankments over such soils. The test was also used to indicate the rate of settlement and generally it indicated that settlement would take a very long time, usually several years. This experience was at least partly responsible for the development of vertical sand drains, used to provide an artificial drainage path and so increase the rate of consolidation.

But, in practice, it was often found that these soils consolidated much more rapidly than the theory and the laboratory tests suggested. The reason when found, was an obvious one. It is that frequently these soils contain thin bands of more pervious sandy soils, bands which were laid down when, for some reason, the water which carried the original deposits was running faster than usual. These lenses of sandy soil proved horizontal drainage paths through which the pressures generated in the soil water under load can be fairly rapidly dissipated. The theory has now been elaborated to take account of this phenomenon.

I am in some danger of appearing to wander from the subject of this conference. But, in fact, it is very relevant. It is to support my belief that the basis for training young highway engineers in soil mechanics lies in giving them a knowledge of the soils and the rocks of their own countries, of what they are, why they occur where they do, and what are their engineering properties and uses and their limitations. And this applies with particular force to engineers concerned with low cost roads. They, above all, must know what can be done with the materials which are cheapest and ready to hand.

I could rattle on for hours about different facets of it. About black cotton soils, those highly montmorellinitic clays found in poorly drained areas and which present formidible problems to the road builder, about the halloysitic clays which form in areas of volcanic activity and which, though texturally classified as clays have an open porous structure which is free draining, and of the dangers of overcompacting these soils; about laterized soils, and about decomposing rocks--rocks which, on hand inspection, appear sound, but when they are used in the building of a road, the feldspars they contain collapse to become kaolinitic clay. It is a fascinating subject and one likely to fire the imagination of any young civil engineer. It would have given those engineers in East Asia the knowledge to understand why landslips occur in that part of the world and what they can do about them. This brings me to my main thesis. Road engineers are like doctors or carpenters, like teachers, or, for that matter, like lawyers and economists. To do their jobs well, they need to be craftsmen. They need to know the materials of their trade thoroughly, how they behave in given circumstances and how to modify this behavior to get the best results. It is useful, too, if they can explain things in scientific terms. But, the first requirement is the art, the almost intuitive understanding of the materials of one's trade built up by experience.

This theme is well-illustrated in the history of the use of asphaltic materials as road surfacings. We pick up the story at the end of the last century. Natural asphalts had been in use for some time to make durable surfacings for city streets. Refined bitumen from crude oil had appeared on the market, and there was a rush to use this material on roads, both in America and in Europe. Small enterprising companies set up in production, generally trying to make synthesized mixtures imitating the natural asphalts. Secret formulas and trade names proliferated. At this stage, the trade was well ahead of the buyer. Most engineers were content to buy on the assurance from the contractor that his was the best product that modern technology could produce. The more discerning soon realized that it wasn't so, that often they were getting a bad bargain. Amongst these discerning people was Clifford Richardson, Asphalt and Concrete Inspector to the District of Columbia. He decided to do something about it. He set out on a tour of Northern America and Europe examining asphalt surfacings. His aim was to determine why some asphaltic concretes proved very durable and satisfactory whilst others didn't, and then to indicate how to make sure you could get a durable asphaltic concrete every time. His method was to enquire from engineers as to how well their asphaltic concretes had performed, and then to take samples of both the good ones and the bad ones analyzing them to determine what they were made of. He spent several years at this task; and at the end of it, he was able to prepare and publish his conclusions. They were sand asphalts for the most part, and he produced specifications on how to choose the best sands, and on how much bitumen of what hardness to use in order to obtain very durable asphalt surfacings. He showed how the composition should be varied--softer bitumen, and more of it in cold climates; harder bitumen and less of it on more heavily-trafficked roads.

This was the craftsman's approach. It paralleled what Fanny Farmer had been doing a generation earlier. His specifications were essentially recipes for producing good asphalt surfacings and they included variations to take account of the effects of climate and traffic. The primary regard was for quality in the finished product and the method involved searching out the best materials locally available and combining them in cookery book style to produce extremely durable asphalts. One example, laid on the Thames Embankment in London in 1906, was still doing duty under very heavy traffic in 1955 and was then replaced, only because there were so many trenches cut through it to repair power mains, sewers, and water mains.

This tradition continued in Europe. In Germany, it was responsible for producing gussasphalt, that immensely durable mastic asphalt used on most main roads in Germany. In Great Britain, it produced rolled asphalt to BS 594.

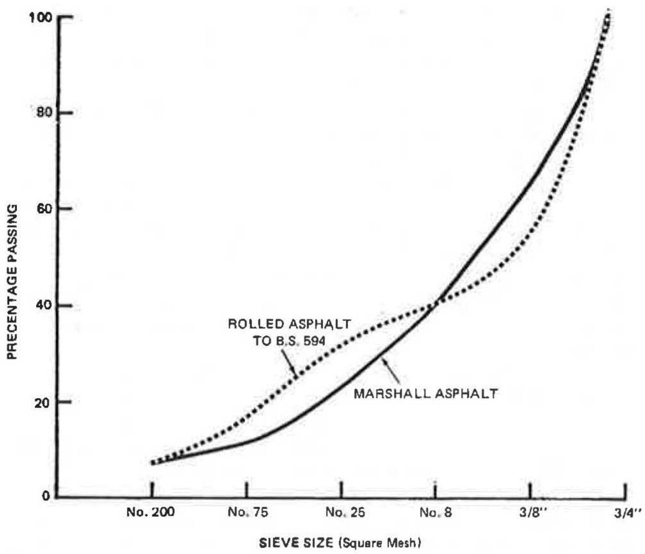
Richardson was not so much honored in his own country. In any case, a new prophet arose in the U.S.A.--Bruce Marshall of the Mississippi State Highway Department. He took another line altogether. His aim, a very worthy one, was to be able to make asphaltic concrete out of materials you could readily find nearby. He used mechanical tests aiming to measure physical properties of the asphalt which were relevant to their performance on the road. Other prophets followed the same line, producing the Hubbard-Field testing regime and the Hveem testing regime.

Then, the Corps of Engineers took a hand. They had a problem with the increasing weight of aircraft in providing adequate airfield surfacings, and immediately after World War II, another problem hit them, the advent of jet aircraft. Aircraft tire pressures increased enormously. Airfield surfacings had to withstand huge increases in weight and pressure, far higher than were needed on roads. Those mechanical tests, particularly the Marshall test were ideal tools to use in developing asphaltic concretes suitable to withstand these high pressures. In America, these "Marshall" asphalts came to be used as the standard surfacings for roads and airfields. They spread over the rest of the world, too. They spread to South America, to Africa, and to Asia. It was so convenient to quote the excellent specifications produced by the Asphalt Institute and the ASTM. They spread to Europe for airfields. But they were not used in Europe for roads. Europe remained entrenched in its traditional methods of making asphalt surfacings. In Great Britain, particularly, the line of evolution started by Clifford Richardson continued in the development of rolled asphalt specified in successive editions of British Standard 594.

The two materials Marshall asphalt and rolled asphalt are markedly different in composition and in performance. Marshall asphalt is usually made with crushed rock in a continuous gradation approximately to a fuller curve. Rolled asphalt was traditionally made with natural sand fines and with crushed rock as coarse aggregate, a gap grading (See Figure 1). Marshall asphalt is generally made with one grade of bitumen the same all over the world. With rolled asphalt, the hardness of the bitumen is adjusted according to the climate and the intensity of traffic.

You may think that these differences are natural and hardly worth bothering about. Some of you may even say, "Shucks, why don't they do it the good old American way? It works for us, why shouldn't it work for them?". But does it work for you? Particularly does it provide a good surfacing for the more lightly trafficked roads which are the theme of this conference? I think not. In the design of Marshall asphalts, the primary requirement is high stability. In cookery book terms, they are short, like Scottish shortbread. In scientific terms, they have quite a

Figure 1. Aggregate gradation for Marshall asphalt and rolled asphalt.



high modulus of elasticity which gives them good load spreading properties. But they cannot tolerate high strains; their tensile strength, particularly under dynamic loading, is not high. The specifications do give different test criteria for asphaltic concrete to be used under heavy, medium and light traffic. But these adjustments are not large and Marshall asphalts can generally be characterized by their high stiffness and resistance to deformation. They are eminently suitable for use on airfields and on the stiff, strong pavements used on heavily trafficked roads. But they are not so good on more lightly constructed roads which deflect under load. Rolled asphalt, on the other hand, has a lower stability. It is more prone to deform under load. Rolled asphalt, on the other hand, has a lower stability. It is more prone to deform under heavy loads. But it is more able to tolerate repeated flexure and has, therefore, some slight advantage over Marshall asphalt for use on more lightly trafficked roads.

Surface dressing or seal coat is even more effective under these circumstances. The thick film of asphalt effectively seals the road surface, binds it together and prevents water from getting in. But road engineers, particularly in developing countries, don't like seal coats. Seal coating requires skilled and experienced operators, and these skills may not be locally available. But the real reason is that they have accepted a technological myth--that Marshall asphalt is the most advanced, and most civilized and the most effective way of surfacing all asphalt roads. And this simply is not true.

In passing, it is worth recording that there are signs of a rapprochment between the American and the British methods of asphalt design. It is started in South Africa, which is perhaps not surprising. They are exposed to the technical influences of both America and Europe, and they have no vested interest in either. They have accepted that there are virtues in gap-graded mixtures for their conditions, i.e. the plums in the pudding mixtures produced by British Standard 594. And to design their mixtures, they use mechanical tests on the pudding, the fines-filler-asphalt mixture, employing design criteria derived from local experience. Great Britain has followed this lead, and in the latest edition of the British specification contains as an alternative, a mechanical testing procedure which can be used to determine the optimum asphalt content of the fine fraction of the mixtures.

But I want to return to consider what kinds of bituminous surfacing we should be using on the low volume roads which are the subject of this conference. By low volume, I am assuming that we mean roads likely to be carrying up to say 500 vehicles per day. Within this spectrum, we can expect that roads carrying over about 150-400 vehicles per day will require bituminous surfacings; that the pavement construction of these roads will be relatively light and that many of them, forestry roads for example, will be called upon to carry quite heavy vehicles with axle loads up to 11 tons or more.

What is the function of the bituminous surfacing under these circumstances? Clearly it will not be expected to add very much to the intrinsic strength of the pavement. If an asphalt premix is contemplated, it is not likely to be more than 2 inches thick. And if we are thinking of hot climates, the extra stiffness it will provide to the road structure is little more than would be provided by an extra 2-inch thickness of road base. The main functions of the surfacing are:

1. To seal the surface, preventing the entry of surface water which would weaken the road structure.

2. To protect the base from the disruptive effects of traffic.

These traffic forces between the tire and the road are very complex. In addition to the vertical gravitational forces, there are forces tangential to the wheel deriving from traction, braking, and turning; and there are other disruptive forces between local protruberances in the road and the tire. And when the road is wet, quite high dynamic stresses both compressive and tensile are generated in the water trapped in interstices, particularly when the pavement is deformed under passing loads. There are interesting side effects that have been observed. For instance, that road surfaces tend to polish and become smoother in dry weather; that on a given surface, the extent of this polishing is arithmetically proportional to traffic intensity; and that when weather comes, this polishing action ceases and the surface texture recovers at least some of its original roughness. This phenomenon is important in improving the skid resistance of heavily trafficked roads. To us, it is an interesting but not very relevant digression.

Our concern is to decide what form of asphaltic surfacing best provides the waterproofing and resistance to traffic wear, which are the prime requirements on more lightly trafficked roads. And we have to consider three influences on our choice:

1. The materials locally available for road building.

2. The effects of the local climate, predominantly the temperature range and the prevailing moisture conditions.

3. The technical and social influences, e.g. what levels of technical competence can be expected and what form of technology is appropriate to the region, ranging from the highly mechanized processes used here in America to the labor intensive methods of road building used in India.

All the time, we shall be bearing costs in mind, since our objective is to produce the cheapest solution. All these considerations lead in the same direction, that what is needed on the more lightly trafficked roads is some sticky substance which will seal the surface of the base and impart some cohesion to resist the disruptive forces of traffic. In some parts of the world, there are waste products available that can do this job, molasses residues and lignin sulphites, waste from one method of wood pulping, even waste sump oil from internal combustion engines. But they are not very durable and generally are worth using only in the immediate area of production. The predominant products are bitumen from crude oil and tar from coal. These materials can be applied to the base in fairly thick films so that they waterproof and remain intact under traffic stresses. Because they are sticky, they need the protection of a layer of stone chippings. This does raise some difficulties in parts of the world where there is no rock or gravel easily available. But elsewhere, the answer is clearly surface dressing. Surface treatment, seal and chip.

There is not time to go deeply into the mystique of surface dressing. Suffice it to say the specification can be adapted to meet almost all the climatic extremes encountered throughout the world, that the materials required are readily available in most of the world, that it is a cheap process (generally about a fifth of the cost of asphalt premix surfacings per unit area) and that it is readily adaptable for both highly mechanized work and for work using hand labor with simple equipment. Add for good measure that most of the secondary roads in Europe were built and improved using this process, and that in Australia and New Zealand the normal expectancy is that roads built with crushed stone bases and single surface dressings with traffic up to 2000 vehicles per day will last for at least 10 years, usually longer before periodic maintenance is required.

Where is the snag? Why isn't this process being more extensively used? There are some technical reasons. Surface dressing in rainy and cold weather can be a chancy business. But there are remedies for that. The predominant reason is that the process needs skill. Skill of the engineer in specifying the right combinations of bitumen and aggregate in given circumstances and above all, skill of the operators working on the road to ensure that the bitumen is spread uniformly at the rate required and skill that the stone chippings are clean, fairly uniform in shape and are uniformly spread. In Africa, and for that matter, in parts of America, surface dressing has been falling into disuse because there is no premium for such skills. We are back with craftsmanship and the need for training and motivation. Here, I must frankly confess I do not know how to cope. But I am utterly convinced that surface dressing is the preeminent method for the surfacing and periodic maintenance of lightly trafficked roads and that very large economics in the use of resources can be made by the more widespread, proper use of this process.

Now we turn to the last of the themes--the economic appraisal of highway projects. I move with some trepidation because my primary discipline is engineering--not economics. I had always believed that the benefit/cost study was invented by engineers and was built up to its present level of complexity by economists. I was heartened in that belief by discovering that John Loudon McAdam used benefit/cost calculations to convince a British Parliamentary Committee in 1820 of the need to spend more on the approach roads to London by considering possible savings in vehicle operating costs. But the confidence which this engendered evaporated when I remembered that McAdam was initially trained as a lawyer.

This lack of confidence tends to inhibit us as engineers when we are called to join in economic appraisals of our work. This is a pity because it is part of our function to demonstrate that the solutions we offer do represent the best value that can be obtained with the resources available. Behind the economists' calculations of costs and benefits, there are engineering realities and it is our duty to make sure that these realities are correctly interpreted in the calculations.

In recent times, we have come in for some criticism that we wish to push ahead with our road schemes with inadequate consideration of the economic and social consequences of what we want to do. Sometimes there is justice in this criticism, for instance in the wholesale advocacy of urban freeways during the 1950s and 1960s. But it is not an apt criticism with low cost roads. Our weakness lies elsewhere, that we have often not been able to express in precise economic terms those engineering realities to which I referred earlier.

Economic calculations presume an ability to predict future events with some certainty. How much will the road cost to build? How long will it last? What kind of maintenance will be needed and how much will it cost? How much traffic will it be called upon to carry; how will the traffic develop? How much will the nature of the traffic determine the design of the road we build and its costs? How much will the standards to which we build and maintain it affect the costs of operating the traffic over it? And in development roads, how much will the roads we build affect the economic and social life of the communities served by the roads? The engineer has a contribution to make in answering all these questions and it is vital that he should make, and be seen to make this contribution.

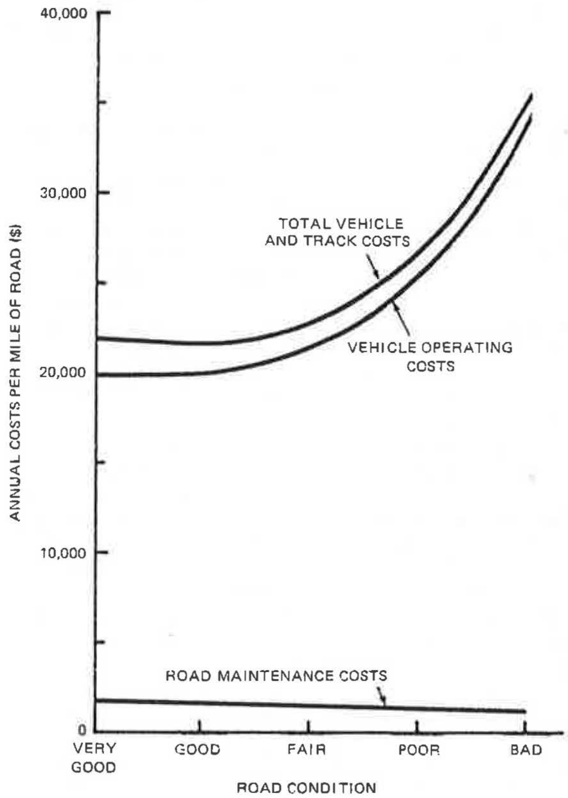
Engineering costs are frequently underestimated in preliminary studies. There are several reasons for this, one that detailed engineering frequently reveals foundation problems that were not discovered during preliminary studies, and another that the complexities of arranging site operations so that work can proceed smoothly are not adequately appreciated. There is also the temptation to underestimate costs in order to enhance the apparent viability of the project. The durability of engineering works often does not come up to expectation. Sometimes, this is traceable to faults in the design or inadequacies in the specification. More often, it is because work is not executed as planned, i.e. poor workmanship and inadequate control. Present traffic can be measured but there is often uncertainty in predictions of traffic growth; and if traffic loadings, particularly individual axle loads, exceed expectation, then early failure is likely to result.

One of the economic questions concerns the length of life to be assumed for a road. The question itself contains a fallacy, that a road pavement is an expendable commodity like a bar of soap, to be replaced as soon as it is used up. Roads are rarely discarded and replaced. Usually, they are strengthened and widened as traffic increases. The question is better rephrased as, "What is the most economical form of construction, improvement and maintenance strategy for the traffic that is likely to develop over say the next 20 years?"

Almost all countries in the world now have quite extensive road systems. Whilst some still need to improve and extend their main road networks, emphasis has been moving towards building up their minor road systems and above all towards more objective ways of planning and funding road maintenance. In Northern America, Europe, and Australasia, there are usually very capable highway maintenance organizations within state and local authorities, backed by an efficient private industry. In many parts of Africa, Asia and Southern America, these organizations are still embryonic.

On an existing road system, track costs are very much lower than vehicle operating costs. For example, on a well-maintained road carrying 100 vehicles a day, vehicle operating costs are in the region of $20,000 per mile per year. If, for any reason, the standards of road maintenance are relaxed, vehicle operating costs will rise. The interaction of the two costs is illustrated in Figure 2. The dominance of vehicle

Figure 2. Typical annual costs on a road carrying 100 vehicles per day.



operating costs is obvious even at the low traffic flow of 100 vehicles per day. It is also evident that total costs are scarcely affected by changes in the road condition between very good and good, but that once the road condition drops below fairly good, total costs rise sharply. I must immediately hedge any conclusions from this diagram, with reservations. It applies to an existing road or road system which does not need substantial investment to bring it up to good conditions. The terms good, fair, poor, and bad are subjective, and the vehicle operating costs, though of the right order, are imprecise. Nevertheless, the conclusions are obvious that it pays to maintain roads in a condition such that vehicle operating costs are not deleteriously affected. The practical application of this conclusion is prejudiced throughout the world by two things. One that the capital costs of bringing the road system up to a "good" standard may be high, and the other that the costs of building and maintaining the roads falls on the public purse whilst most of the savings accrue to road users, most of whom are in the private sector. It is interesting that in well-managed private organizations which operate their own vehicle fleet over their own roads, such as tea estates, rubber estates, timber concessions, the roads are often built and maintained to a high standard. I can take you to a timber concession in New Zealand where the road layout and the road standards and the vehicle types and operation have been determined by calculations aimed at minimizing total transport costs and where the timber extraction vehicles are under on-line computer control. On this estate, they have all the data available to work in this rational manner, and the roads are good, and well-maintained.

Over much of the world, we work in a climate in which public expenditure must at all costs be restrained. In road transport, "at all costs" is a misleading phrase because often vehicle operating costs are not known. Or, if they are known, they are subjects of considerable controversy.

Our attempts to build roads as economically as possible may be frustrated by outside interests. Here are two examples. Under supposed market pressures, road making machinery has become increasingly large and sophisticated. It is often now no longer possible to obtain the simple rugged machines that are most suitable for building low cost roads, particularly in developing countries. And government regulation or the absence of it may frustrate our efforts for instance in the enactment and enforcement of regulations on vehicle and axle loading.

Now, after all this mayhem, I must try to be constructive. First, it must be clear that economic appraisals are absolutely necessary. The days are long gone when road schemes could be initiated to meet an obvious but unquantified public need. We need to make sure that expenditures on roads will produce real economic and social benefits in the areas they serve. And we need economic calculations to determine the engineering standards to which individual roads should be built and road networks maintained. Now to look at some of the individual aspects which I have sprayed with cynicism.

There are ways in which the preliminary estimates of engineering costs can be made more accurate. On major road works, risks of unexpected foundation problems can be reduced by more detailed site investigations. This is not a remedy open to us on lightly trafficked roads. Our remedy is to find and employ engineers who know the area and who are aware of the foibles of the different kinds of terrain through which the roads are being built. We are back with the need to educate engineers on the resources of their own countries. Similarly, it is this local experience which can anticipate local difficulties in mobilizing effective construction teams. And, I think that we are learning that the deliberate underestimation of costs in order to promote a particular project, is a self-defeating exercise.

I am not certain how well we shall do in improving our estimates of the long term performance of highways. That many roads do not perform as well as expected is sometimes due to false economies in design. More often, it is because the roads are not built as designed. In this, I see a decay in workmanship. One of the beliefs was that the increasing sophistication of road building machinery would, by eliminating "human error," produce a more uniform product, more consistently close to what the design engineer intended. There are examples in which this improvement has been demonstrated, for instance in sprayers for liquid asphalt. But, overall, although quantity of output has increased, the effects on quality have often not been good. Workmanship has further declined; there are fewer and fewer engineers and road foremen who understand the materials of their trade and the subtleties of road making processes, and when the machines go wrong, the consequences can be quite large. On the bright side, there has been a considerable growth in expertise in the mechanics of quality control. Ready-mix concrete producers, for instance, have improved their competitiveness considerably by employing statistical methods of controlling their materials and mix-proportioning. And, perhaps I am somewhat conditioned by the indifferent quality control I see on many road building works in developing countries. If so, we are back with training, on the need in training engineers to emphasize the importance of quality control and to indicate how best it can be done. In this field, at least, physical and social environment are not relevant; the value of adequate quality control is the same everywhere. It is a necessity in getting good value for money in road engineering works.

Predictions of traffic growth are always likely to remain uncertain, and it is usual to undertake sensitivity analyses to determine the effects over the likely range of traffic growth. Fortunately, some calculations, for instance in the design of new pavements, are not very sensitive to variations in traffic growth. But all calculations are very sensitive to the assumptions made on the effects of road conditions on vehicle operating costs. Until recently, the data base for relating vehicle operating costs with road conditions was very scanty. In effect, it consisted of data collected by Robley and Winfrey on rural mail trucks in Iowa in the late 1930s; some information on the costs of operating trucks and buses India reported to the Indian Roads Congress in 1961; a study of the costs of operating some 200 trucks and buses over roads in East and Central Africa reported by Bonney and Stevens in 1967; and a review by deWeille in 1966 in which reported experience is nicely balanced with judgements to fill gaps in knowledge. Those of you with speculative minds may wish to ponder this small effort and its costs and benefits in comparison with the enormous worldwide research effort which has been put into pavement design over the same period. Fortunately, we have gone a long way to redress this imbalance in the last decade, and the results of some of this more recent work are being reported at this conference.

The important advance is that vehicle operating costs can now be related to a measured quality of the road surface, its roughness. Previously, a subjective assessment of road condition was used, generally "good, fair, poor, and bad," as I have used in Figure 2. This subjective assessment led to some lack of credibility. It was easy to believe that vehicle operating costs were not really as high as the economists made them out to be. Yours wasn't really a "bad" road. Considering the effects of last year's rain, and the miserable pittance available for maintenance, it was really "fairly good."

Now we can measure the roughness or the riding quality of the road and derive from it what the operating costs of the expected traffic will be. We can go further. We can measure the roughness over a complete highway network, and knowing the pattern of traffic movements, we can estimate the vehicle operating costs over the network. And we can examine the effects of changes in road roughness over the network on total vehicle operating costs. We can, for the first time, indicate how variations in the standards to which we maintain a road network affect the costs of operating the road transport system. This is an enormous step forward. Although it is too early to judge the ultimate impact of this work, it is in use in at least eight countries to help in planning highway maintenance and strengthening programs. Perhaps its most important benefit so far has been in gaining the agreement of financial officials to larger maintenance budgets by demonstrating the much higher total costs of neglected maintenance. For the first time, we have the prospect of being able to establish on a sound economic basis, what the level of expenditure on road maintenance should be.

There are three limitations on this ambition:

1. That in many countries, the roads have not yet been brought up to a standard at which they can be kept in reasonable condition by normal routine and periodic maintenance.

2. That though measurements of surface roughness can indicate what parts of a road network are in need of treatment, they will not usually indicate the nature and the scale and the cost of the work required.

3. There are other aspects of road maintenance for which different criteria apply, such as the slipperiness of the road surface, the need for traffic lane markings and the need for structural maintenance of bridges, side slopes and verges.

None of these is a disabling limitation. On one at least, there have been parallel advances towards becoming more numerate. In most countries, the network of asphalted roads has been vastly extended during the past 30 years. On many of these roads, the pavements are reaching the end of their useful lives. Under the fatiguing strains of traffic, cracks and potholes are becoming more evident. The pavements must be strengthened soon; otherwise, their condition will deteriorate still further and it will be necessary to spend much more money in reconstructing the pavements completely. When should they be strengthened, and by how much?

Earlier, I mentioned one Harry Benkelman of the US Bureau of Public Roads, as a member of our Hall of Highway Fame. It was he who, in the 1940s, introduced the use of a long beam and fulcrum to measure the deflection of pavements under a slowly rolling load. Since then, other forms of apparatus have been developed, all aiming to measure the in-situ strength of pavements. Some measure deflection and curvature under load, others use indirect methods of indicating the stiffness of pavement layers. All are empiric and all need careful correlation to make sure that the measurements mean in the prevailing local climate and with the particular road making materials employed. So far, the most careful and extensive correlation has been undertaken with the Benkelman Beam or its automated derivative, the Lacroix deflectograph. Such instruments have been used for the last decade to indicate when and by how much individual roads need to be strengthened. And in two countries at least, France and Ivory Coast, deflection measurements are being used to plan road strengthening programs on a regional and national scale. Skid resistance is not normally a critical aspect of the maintenance of low cost roads, but it is germane to note that methods and criteria are now available for examining the adequacy of road networks in this respect.

This paper is already over-long, and I am conscious of many interesting omissions. Finally, I must touch on roads and rural development.

Low cost roads tend to be roads associated with rural development. Often, therefore, we are planning and preparing economic justifications for road systems that do not yet exist. Much of the benefit will lie in the development which the construction of the roads will make possible. Here, there is an important difference in principle. In benefit/cost studies for the improvement of existing roads, we are aiming to make the most economic use of available resources. With development roads, the aim is quite different; it is to extend man's capacity to use the earth's natural resources, i.e. to create new sources of wealth. The building of the roads will not of itself insure that the new sources of wealth are efficiently exploited. In more developed countries, America and Australia, for example, or rural France, the prompting for the building of new rural roads can come from an articulate and vociferous rural population; or it can derive from the plans of rural development enterprises such as forestry departments or private logging concerns. In these situations, there is always a group of people with the power and the interest to set about the use of the natural resources to which the roads will give access. But in developing countries, the presence of this enterprise is often less certain. The local people are often not articulate and sometimes suspicious of change and innovation. Sometimes, in this situation, the effects of road building can be dramatic. In the early 1950s, feeder roads were being built in the West Nile area of northern Uganda. For the first time, itinerant Indian traders entered the area with bush-pan radios and cheap cotton goods for sale; within three years, there was a four-fold increase in the production of raw cotton from the area. About the same time, roads were being built into a low-lying area of Borneo eminently suitable for the cultivation of wet padi. In one part of the area, the local people did start to grow rice for market. In a contiguous area, they did not. The system of land tenure enabled the village elders to resist this innovation.

More generally, when crops are being produced for local consumption, the demand is obvious and the system is self-regulating. But when crops are being grown for export to international markets; cocoa, coffee, palm oil, rubber, etc., the pace of development will depend very much on world prices, and on the extent to which governments move to control farm gate prices.

Thus, there are often difficulties in making reliable estimates of the economic and social benefits of new rural roads. An input-output model suggests itself. The input consists of the costs of building and maintaining the roads and the output in some measure of the increased prosperity which is assumed to derive from providing roads where there were none before. Such a model may be simple in concept, but it is very complex in application. There may well be other necessary inputs, water supply, irrigation, agricultural extension services. And as I mentioned earlier, the increase in prosperity is likely to be critically affected by other factors than increased motorization. There are some clear examples in which the building of feeder roads has been followed by a surge in local production and presumably by the opportunity for the local inhabitants to live happier and more useful lives. But there is a growing feeling that such surges in local development do not automatically follow on the building of new roads.

One obvious solution is that road building should be planned in the context of overall rural development plans. The road network will then be planned and built as part of a staged development in line with other essential aspects. The need disappears for a separate and rather hypothetical economic justification of the road network; but economic calculations will of course, be used to determine the standards to which the roads should be built and maintained; they may even be pushed further to indicate the optimum density and layout of the road network.

But there are still pressures to consider programs of feeder road building as a primary means of accelerating rural development. They are tangible, quick and fairly capital intensive, all of which makes them attractive as subjects for the investment of capital aid. And our economists have been struggling for the last decade to evolve reliable methods of measuring and predicting the value of such schemes. Simple methods have failed and the current trend is to enlist a wider and wider range of expertise--sociologists, anthropologists, market analysts, and so on. Their work will be of enormous interest, and it should help in a better understanding of the intricacies of rural development. But it is not likely to produce analytical methods which can predict the future with the certainty normally expected in economic evaluations; there are too many external uncertainties.

The more hospitable and fertile parts of the world are already being farmed. They have been farmed for centuries. Indeed, it is the surplus produced from farming these areas which set us off on the path of social and economic development. It was used to build towns and cities, to establish culture and civilization. Now we are pushing into the more inhospitable areas of the world where living conditions are harsher, where the climate is more extreme and uncertain, and where the soils are generally less fertile. We have two motives, one to produce more food and other natural products to meet the needs of the world's rapidly expanding population and the other, a desire on the part of those of us who happen to have been born in more hospitable and prosperous parts of the world to extend this prosperity to those less fortunate people who struggle to make a living in areas where nature has been less bountiful. This latter is very much a twentieth century phenomenon; cynical people say that it is an attempt to expunge the guilt about the exploitations of the colonial era. This is nonsense. The real reason lies in the vast improvement of world communications. Fifty years ago, the farmer scratching his land-hoe in Senegal might have been on another planet; now he is our next-door neighbor.

Three considerations follow.

1. That the intention to improve the productivity of these fringe lands usually implies a considerable investment in infrast ucture (including, amongst many other things, feeder roads).

2. That this investment, of its nature, is generally not likely to produce quick economic returns. Indeed, attempts at rapid exploitation may produce irreparable damage to the environment--as in the dust bowl of North America and more recently in the forests of Indonesia and South America.

3. That it would be a gross error to seek to impose this development from outside.

It is this last consideration which causes the most difficulty. This is the method which worked in the past; it was invading people who brought agriculture to Western Europe, and in more recent times to America and to Australia. But those days are gone. Now the task is infinitely more complex. And I like to think that this is what the World Bank is now really all about. It started as a means to provide capital to war-torn Europe. The capital funds went to people who already knew what they wanted to use it for, and recovery was rapid. Now the Bank is engaged in a much larger enterprise to assist in building up the resources of the developing world. Part of this is, of course, in distributing the earth's wealth in a more equable way from richer to poorer. This can be a difficult business as is demonstrated in the demoralizing squabbles in the North/South dialogue between Europe and Africa. The more rewarding part is in building up the institutions of developing countries so that they, too, know how best to manage their own natural resources and the development funds available to them from outside. This is a cooperative enterprise, an educative process for all of us. And oddly enough, the preparation of this paper has helped in my own education. I have been reacting against the attempts to refine the economic analysis of feeder road projects by introducing a wider range of expertise. I now realize that this was an over-reaction, engendered by the fear that it would lead to even greater emphasis on the preeminence of short-term economic returns in determining the viability of projects. Perhaps temporarily, it will. But there is an overriding benefit that it will lead to a better understanding of the development process. And if we can bring in the local people, not as specimens to be studied, but as participators who want to learn how best to develop their country's natural resources, there is a chance that some practical good will come of it. Indeed, amongst the most promising feeder road projects, are those where local participation is strong, as in Mexico and in East Africa where local cooperatives are being mustered and provided with simple equipment and leadership to build roads which will connect them with the countries' main highway networks. A virtue of this approach is that it is self-regulating. The technology and funds are supplied from outside; the motivation and effort are from the local people themselves; they are not likely to build more roads than they can afford; nor less than they need.

This has been a somewhat rambling paper. I seem to have concentrated on attitudes rather than on technology. This was deliberate. It stems, oddly enough, from my belief in systems; in human systems and their capacity to adjust to new circumstances. And it voices my disquiet that things seem to be going wrong with our technology in a way which hampers us from adjusting to new circumstances. It tends to put obstacles between us and our real objectives. The CBR value and the economic rate of return become objectives themselves, rather than a help in defining the reality they are supposed to represent, the load carrying capacity of the soil and the value of a particular endeavor in promoting the welfare of mankind. The realities are, of course, always more complex than the simple models we try to build to represent them. And, as things change, as we move from one physical environment to another, or from one culture to another, our simple models may prove to be downright misleading. The safeguards against these errors come from an awareness of the physical environment and the culture in which we are working, from an intimate personal feeling for what is really going on and why it is going on like that. And here we have a definition of the purpose of education and training. There are some enormous fallacies about the purposes of education and training of technologists. One is that it is to turn them out fully equipped to move into practice. Another is that it is to make them better able to compete for their individual share of the world's resources. Both are wide of the mark. The real purpose is to equip us so that our eyes and our ears are open and our minds are ready to gain experience of how the world works and to put this experience to good practical use.

APPROPRIATE TECHNOLOGY FOR LOW VOLUME ROADS

G.A. Edmonds, International Labour Organisation

In recent years there has been a growing interest in the development and application of more labour-intensive methods of construction. This interest originated from the feeling that the technology presently used in road construction in developing countries was inappropriate both to the economic and social environment. Some 5-6 years ago both the World Bank and the ILO initiated major programmes in this area. The objectives of the programmes have been to ascertain whether it is feasible to use more labour-intensive methods in construction. Whilst some work has been done on irrigation the major effort has been concentrated on road construction. The studies and programmes carried out have shown that labour-based methods are indeed viable particularly for the construction of rural roads. This paper describes the work that has been carried out in Iran, Thailand, the Philippines, Nepal, Kenya, Guatemala and India. It shows how the initial concern with increased productivity and economic evaluation gave way to an emphasis on institutional, administrative and managerial problems. The paper highlights the major problems of the implementation of an effective labour-based programme. It is argued that the use of labour-based methods requires a re-appraisal on the part of engineers of their traditional attitudes not only to the details of design, management and organisation of low volume road programmes but also to the integration of these programmes into the general development of rural areas in the developing countries. Finally, it is suggested that to provide an environment in which labour-based techniques are considered as an alternative technology will require changes in fiscal and institutional measures to ensure that there is no inherent bias against these methods.

In most developing countries, investment in construction is of the order of 60% of the total public investment in the economy as a whole. Investment in road construction is usually higher than in any other form of infrastructure, being as much as 50-60% of the investment in construction.

Because the vast majority of road construction is financed and supervised by the government, there is the means to ensure that resources are used in the most effective manner. In fact, there is clearly some responsibility placed upon the government to make the most optimum use of the available resources. It is on the question of the way in which the resources should be used that we shall be concerned in this paper.

The Case of Road Construction

In recent years various investigations have been made on the technologies presently in use in developing countries in an attempt to show which were the most appropriate. Not surprisingly one of the first activities that came in for scrutiny was road construction. As we have seen, it consumes a large proportion of government investment. If it were possible therefore to make the techniques used in road construction more labour-intensive, whilst being still efficient, then this large government expenditure would be used to promote employment and reduce the level of dependence on foreign imports.

Apart from these obvious benefits, there are however other good reasons to consider road construction. There is, after all, a historical precedent for the use of labour-based methods. (In this paper the general term 'labour-based' has been used so as to avoid confusion between the terms labour-intensive and labour-extensive) In 19th century Europe and America, before the advent of the internal combustion engine, roads were built by men using simple hand tools and animal-drawn equipment. Furthermore, in countries like India, roads built using large equipment are the exception rather than the rule. Then, again, the tasks involved in road construction are relatively simple and can be carried out by relatively unskilled labour who require little training. Much of the labour to be used will be from the rural areas and the tasks, tools and techniques utilised in the major activities such as earthworks are not dissimilar from those used in agricultural activities. In addition the amount of employment created per unit of investment is potentially large. Moreover, employment creating activities which take place in the rural areas, as much of the road construction programmes do, could help to limit rural/urban migration.

Construction equipment is a major import in many developing countries. Furthermore, import of such equipment implies the future importation of spare parts. The cost of using the equipment is extremely high, requires skilled personnel and relies on fuel which is permanently subject to international economic factors. Any reduction in the level of equipment imports would reduce the pressure on limited foreign exchange resources.

There is also scope for the increased use of labour-based methods in the extremely important activities of road maintenance, particularly for routine activities such as ditch clearance, culvert maintenance and minor road surface repairs.

The case for an evaluation, at least, of the feasibility of using labour-based methods is therefore very strong. It would be wrong however, to assume that there are no fundamental objections to their use. Indeed some of the arguments against their use are quite potent and it is useful at this stage to present them.

First, and perhaps most important, is the feeling that the use of labour-based methods will automatically produce a reduction in standards of construction. There is certainly some justification for this objection, particularly in relation to the compaction of earthworks and the final surfacing where it is true that it is extremely difficult to provide the same standard using labour-based methods. It would be wrong to suggest that labour-based methods should be used when they produce inferior quality. Indeed, the approach should be that the methods used should be the most appropriate and if this means the use of equipment, then that should be the solution. There is, however, another aspect to this issue. The standard of construction is specified by the design. Often the design is orientated towards the use of equipment and it is therefore not surprising that labour-based methods cannot meet the requirements. In addition, as suggested later in this paper, the standards of construction may be artificially high.

Second, it is suggested that the productivity of labour-based methods is low. This naturally has repercussions in terms of the duration and cost of projects. It is pointed out that low productivity means either an extension of the project time or that the number of labourers on the site has to be very high causing managerial and logistic problems. Unimproved, traditional labour-based methods do have a very low level of productivity. On the other hand, there is a large variation in the productivity of manual methods. In the activity of excavation for instance, the World Bank (1) noted a six-fold difference in productivities for different parts of the world. Clearly, there is a potential for improvement of productivity given the right environment. There is therefore some validity to this criticism and consequently the improvement of productivity has been a major factor in the work on the development of viable labour-based methods. In one respect, however, low productivity may not be detrimental. There are certain construction activities which are non-critical, that is, the extension of their duration will not adversely affect the overall construction time. On these activities, it would be feasible to use labour-based methods even if they are not as productive as equipment.

Third, it is generally assumed that the cost of using labour-based methods is prohibitive. There is, as is discussed later, an automatic bias in most project appraisal criteria against these methods. Nevertheless, certain comparative studies have given support to this objection. It is worth noting, however, that in general these studies attempted to evaluate what would happen if labour-based methods had been used on a project which had used equipment. The comparison is therefore carried out in a framework orientated towards the use of equipment. One of the main themes of this paper is that to equitably assess the most appropriate methods the whole construction process must be "opened up" to allow the consideration of alternatives. In the final analysis, however, cost is supremely important. The Public Works Ministry or Highways Department has a limited budget for which it is accountable. It has to be sure that the best use is being made of that money. The use of labour-based methods must be shown to be not only socially but also economically appropriate.

Fourth, there is a strong feeling that these methods require a high level of supervision which gives rise to high overhead costs not only on site but also in terms of additional training programmes. It is certainly true that labour-based methods require a different kind of supervision. After all, one is considering the management of large bodies of men, not fleets of equipment. Whether the supervision is more costly is open to question and in any case should not be considered in isolation.

Finally, there are institutional forces within the construction industry which militate against the use of more labour-intensive methods. The design methods, specifications, conditions of contract and methods of tendering all tend to reflect the dominating influence of expatriate consultants and contractors and their equipment-oriented framework. Even the method of selecting contractors is often based on the amount of equipment they have, and not on any ability to manage men. There is a natural inertia within the industry which could make the introduction of labour-based methods difficult.

In summary, there seem to be two categories of problems. First, the purely technical problems of low productivity, limitations of quality and high cost. There are then other problems which seem to have more to do with existing systems in the construction industry. Any successful programme will have to show that the technical problems can be solved and then demonstrate how the design, management, planning and administration of road construction projects could be made to include the effective use of labour-based methods.

Translating Theory into Practice

It was with the above concepts in mind that, under the World Employment Programme, the ILO launched a major investigation of the most appropriate techniques to be used in road construction.

As noted above, the main technical problems seemed to be productivity, cost and quality. The initial work was therefore concerned principally with these issues. A series of studies was initiated which attempted to provide some form of comparison between equipment-intensive and labour-based methods. The comparison was made with the use of cost benefit analysis. The analysis, however, used "shadow" prices which allowed for the distortion of market prices and reflected social values placed upon the various resources. The studies in Iran (2) and Thailand (3) were therefore particularly concerned with the economic viability of the use of labour-based methods in road construction. Both studies indicated that these methods could be competitive; however, they also showed that their viability was particularly dependent upon the level of productivity. For instance, the study of the substitutability of labour for equipment on gravel road construction in Thailand (3) showed that the range of productivity increases required to make labour-based methods viable for all activities was 15-40%. Work carried out by the World Bank had already indicated that increases of this order were perfectly feasible given the right type of management and supervision (16). Recognising the importance of improved productivity, the study in the Philippines (4) was more concerned with the development of effective techniques than with the finer points of social cost-benefit analysis. The work done so improved productivity that, even using a cost comparison based on market prices, the labour-based techniques were cheaper.

In the first instance, therefore, the studies, carried out by the ILO and the World Bank, lead to the conclusion that the use of labour-based methods was technically and economically viable for a wide range of road construction activities.

The more recent work has indicated that even though labour-based techniques can be competitive, it is quite another problem to initiate their use on a large-scale. This is partly because of the inherent bias against their use by engineers themselves. There are, however, other basic difficulties in connection with management, administration, planning and organisation. The studies in Nepal (5), Pakistan (6), Kenya (7) and Guatemala (8) were particularly concerned with these issues.

The study in Nepal showed that the mobilisation and motivation of large labour forces required a type of planning and organisation structure significantly different from equipment-intensive projects. It also showed that the sequential method of working may not be appropriate for labour-based methods and that a more segmented approach may be better.

In Pakistan, the ILO was requested to advise on the possibility of using labour-based methods in the construction of the proposed 1220 km Indus Super Highway. Preliminary findings showed that it would be more feasible to think in terms of the most effective mix of labour-based and capital-intensive methods and a first step was made in preparing guidelines on how the planning for this might be done.

In Pakistan, it was more a question of how to improve the existing techniques and indicate how they might be used in conjunction with equipment. In Kenya, on the other hand, a study that originated as a feasibility study on rural roads has been transformed into assistance with the operational management of the Rural Access Roads Programme (RARP). This programme aims to construct some 12,000 km of rural road throughout Kenya. The methods used will be as labour-intensive as is commensurate with technical and economic efficiency. It is hoped that the lessons learned in the development of this programme can be applied elsewhere in similar situations. To a certain extent, this is already being done in Guatemala where assistance is being provided to the government in the implementation of a labour-based rural roads programme. Moreover similar programes are now being initiated in Ethiopia and Botswana.

Programes in the field can demonstrate the feasibility of using labour-based techniques. Nevertheless, to reach a wider audience, it is necessary to disseminate the information that has been gathered. The reports on Iran, Thailand, Philippines, Pakistan and Kenya provided the detailed information. The Manual on the Planning of Labour-intensive Road Construction (9) attempted to provide policy-makers and planners with a basic knowledge of labour-based methods. It is hoped that this will provide a detailed argument for the use of these methods and the foundation from which a choice regarding the most appropriate technology can be made.

In the following sections, the major aspects of appropriate road construction technology are discussed in detail; tools and techniques; appropriate design standards and the problems of project appraisal; a more detailed analysis of the planning, management and administration of labour-based programmes. Finally, some basic policy measures that could help the large-scale implementation of labour-based techniques are presented.

Tools, Equipment and Techniques

Most text books on construction management will almost certainly have a section on the choice and utilisation of construction equipment. A typical phrase from a well-known book (10) reads "correctly chosen and well operated plants will enable a construction project to be completed quickly and economically". Given that on a road construction project in the developed countries, equipment may account for upwards of 50% of the total cost, it is not surprising that so much attention is paid to this item. If the resource utilisation is changed, however, and it is labour which is the predominant cost item, then it is clear that much more attention has to be paid to their organisation and their level of productivity. On labour-based projects the choice of the right sort of tools is as important as the choice of the right type of machinery in a capital-intensive project (9). The cost of tools is not normally a major item even on labour-based projects, where they will account for 5-10% of the total. It is their effect on the productivity of labour, however, which is of particular importance. To the best of our knowledge, few detailed studies have yet been done on the difference in productivity for workers with "good" and "bad" tools. Nevertheless, all the experience reinforces the intuitive feeling that a well-designed, well-manufactured tool that is appropriate for the task increases the level of productivity.

Appropriate Tools

Much of the work carried out so far has shown that, as far as simple hand tools are concerned, the problems are principally to do with providing a reliable tool rather than providing an alternative. Attempts to introduce alternatives have generally met with failure partly because of unfamiliarity and partly because of a lack of understanding of social attitudes. The expertise and tradition of using traditional tools is not lacking. What is required is a range of shapes and configurations of blade to suit the soil conditions. It is known, for instance, that a fork-type blade is better suited to non-cohesive soils. Furthermore, we know that different shapes of blade are more effective with hard and soft cohesive soils.

If money is to be invested in the use of labour-based methods it will be necessary to spend much more time in considering the most appropriate tools for the particular job in hand. A labourer is perfectly well aware of the most effective method of using traditional tools; what is needed is the most effective shape or type of blade and its manufacture to a reasonable standard.

In the consideration of appropriate tools, the first lesson to learn is that we must beware of imposing preconceived notions of what is the best tool for a particular job. Many of the basic activities of road construction and maintenance such as excavation, loading and spreading, have counterparts in agricultural work. The labourers employed for road construction will generally be quite familiar with the most appropriate tools. Simple expedients such as reinforcing the blade of the tool at critical points to make it more durable for heavy excavation work seem to be the type of improvements that would be most useful.

Appropriate Equipment

It is when the activities of workers become interdependent, such as in excavating and loading, or where the operations are markedly different from agricultural activities such as compaction, bitumen spreading or hauling with light equipment that there is scope for the implanting of new techniques or the modification of existing ones. Consider for instance the question of haulage vehicles. Wheelbarrows, small trucks on rails, headbaskets, animal-drawn carts are all in general use in agricultural work. However, in agriculture, they are used individually and they can often be improved in relation to the haulage of soil or aggregates (11). The wheelbarrow, for example, apart from being generally poorly manufactured is often also badly designed. Often, the weight distribution is wrong and the size and type of wheel is ill-suited to construction sites. Furthermore, the efficient use of wheelbarrows often requires a careful consideration of the balance between the loaders and the haulers.

Animal-drawn carts are often used in rural areas for the transportation of produce. The unloading is carried out by hand so as not to spoil the saleable goods. As far as the movement of soil in construction works is concerned, one of the main objectives is to unload the cart as fast as possible. Some system of tipping or bottom discharging would therefore have obvious benefits.

For long haul distances it may be appropriate to use trucks or tractors and trailers. The important point to mention here is that the integration of labour-based and equipment-intensive methods requires special care. It is of little value asking labourers to excavate and load in a similar fashion to machines. If they are asked to load into trucks or trailers, therefore, the height of loading must be appropriate and furthermore the size of the truck must not be so big that it requires such a large number of workers to load it that they are in each other's way.

The work carried out so far indicates that even for those road construction and maintenance operations which are not directly comparable to agricultural activities, it is possible to adapt and modify agricultural equipment to do these operations effectively.

Some Examples of Improved Tools and Equipment

The ILO studies in India (12), the Philippines (4) and Kenya (7) were particularly concerned with the improvement and adaptation of tools and equipment. To illustrate what can be achieved it is worth, briefly, looking at the results of these three studies.

In India, the work was particularly concerned with the wheelbarrow and small trucks on rails. Both of these are effective over short haul distances. What the ILO team did, however, was to show, by detailed testing and evaluation, the most appropriate design of both wheelbarrow and trucks which would maximise productivity. In the case of the wheelbarrow, it seems surprising that there was, and still is, such diversity of design. Particularly when the relative positions of the load in the barrow, the effort of the barrower and the wheel itself are vitally important. The trucks on rails had been used for carrying coal and so needed adapting for hauling soil and for manual loading by providing easier methods of pushing the trucks and reducing the height of the sides respectively.

The Philippines study was a particularly good example of intelligent adaptation. To allow easy unloading, the traditional bullock-drawn carts were modified to allow bottom discharge. The traditional 'Chinese scraper' was taken as a model for a factory-made steel version which was relatively cheap to produce but much more effective. To eliminate the need for a motor-driven water bowser a bullock-drawn version with the traditional cart now used to carry oil drums with a sprinkler device was developed. The study, therefore, illustrated the three aspects of equipment improvement. First, the modification of existing agricultural equipment to make it more effective for construction work. Second, the development and mass production on the basis of a traditional technique and third, the innovative adaptation of local materials to provide a piece of equipment which normally would have had to be imported.

In Kenya, the work again concentrated on wheelbarrows and it was possible to develop an efficient prototype. One aspect of the work was of particular interest. Many labourers preferred metal-handled wheelbarrows even though the wooden ones were more durable. It appeared that their preference was based on familiarity breeding contempt for the wooden option which was a material they knew well.

Procurement Systems

The provision of small hand tools and light equipment for a project using conventional equipment is of relatively minor importance. In a labour-based programme it is absolutely vital that the requisite number of small tools are provided in the right place at the right time, otherwise the work will stop. It will often be necessary to develop special procurement systems for labour-based programmes. Furthermore, the increased attention that has to be paid to well designed and well maintained tools will place a much greater emphasis on having well qualified personnel to direct the provision and procurement of tools.

Design

It is often said that labour-based methods are incapable of complying with accepted standards of design. The general implication being that these methods are therefore inferior. The standard of design referred to, however, is one that is appropriate in developed countries. It would be better if we questioned the automatic acceptance of the appropriateness of these design standards to a developing country.

The Choice of Design

One can approach the question of design and the choice of construction techniques in two distinct ways.

First, and most commonly, it can be assumed that the design is fixed and it is merely necessary to choose the most effective construction methods.

Second, the design, either in terms of the pavement or the geometric alignment, is variable to allow the consideration of alternative construction methods. This may or may not also imply that the maintenance and operating costs are fixed.

Consider the first case: The road is designed on the basis of certain economic and engineering criteria. In the simplest terms, it goes from A to B and the route it takes will generally be governed by the economic objectives it hopes to achieve. The actual engineering standards of design will govern the type of pavement, the vertical and horizontal alignment and the structures required to carry the predicted traffic load. What does this mean in terms of the choice of technique? The pavement design will generally be based on practice originating in the developed countries. Consequently, a bias towards equipment-intensive methods can happen in one of two ways. First, it is, of course, possible to produce the required strength and durability by using different materials. The laying of certain materials, however, does not lend itself to the use of labour-based methods. It is, for instance, difficult to lay bituminous material this way; however, an effective alternative such as, for example, stabilised soil can be laid effectively using labour. The first point, therefore, is that care must be taken not to specify materials which limit the scope of labour-based methods unless there is no alternative. Secondly, there is a natural tendency, reinforced by developed country practice, for the pavement thickness to be as small as possible. Put another way, the compaction of the sub-grade should be as high as possible. Whilst compaction achieves a variety of objectives, it is possible to consider that there is some sort of trade-off between sub-grade improvement and pavement thickness. Effective compaction being one of the operations that is difficult to execute using manual methods, a reduction in compaction (i.e. equipment) cost may be justified if it is not more than offset by the increased material cost of extra pavement thickness. In the Manual (9) already referred to, an attempt has been made to quantify this trade-off. It should be recognised that, in the case of a fixed design, there is no question of affecting the recurring maintenance and operating costs. The suggestion is merely that the design, being fixed in relation to certain standards, should allow the use of materials which may be more suited to the use of labour-based methods.

The question of the level of design standards leads directly to the second case when the design is considered as variable depending upon the techniques involved. What is being suggested here is that alternatives should be evaluated not in terms of construction cost alone but in terms of total costs. That is, one may be prepared to accept a lowering of design standards, reflected in a reduction in construction costs, if the increased cost of recurrent maintenance and vehicle operation did not exceed the reduction. Naturally, one is thinking of modifying the design standards so as to make labour-based methods more attractive. For instance, it is often suggested that gradients should not exceed a certain value, say 6%. This may require heavy earthworks when traversing hilly terrain. A relaxation of the gradient limitation may allow the road to take a more direct route, reducing the earthworks which will reduce costs and allow labour-based techniques to be considered. Naturally, it would be necessary to assess whether the attendant increase in operating costs offsets the reduction in construction costs. Taking another example, the standards for minimum horizontal curvature are dictated by the design speed. A reduction in design speed would allow the road alignment to more easily follow the terrain contours, again reducing the level of earthworks and not only reducing cost but also favouring the consideration of labour-based techniques. In regard to design speed, it is in fact possible to quantify the relationship between design speed and construction costs for various types of terrain. Taken to its logical conclusion, the argument for choosing designs which minimise the total cost of construction, maintenance and operating costs could mean the actual change of route. If the direct route necessitated heavy earthworks, large structures and major rock excavation, it could be possible to use an alternative, more circuitous route, which minimises earthworks and structure, favouring the use of more labour-intensive construction methods. This could of course produce a change in economic benefits which would have to be considered in the analysis.

The evaluation of total costs would compare the present value of the maintenance and operating costs over the life of the road with the construction costs for each design alternative. The evaluation should also consider the question of whether the maintenance methods to be employed will be capital-intensive or labour-intensive as there would clearly be additional secondary benefits with the use of the latter.

In a limited way, the study in Kenya (7) attempted to assess whether the total cost of providing a gravel road in the initial stage was less than that of providing an earth road with guaranteed periodic maintenance.

Road Maintenance

A greater investment in road maintenance would have major benefits not only in deferring the reconstruction of the roads but also by conserving foreign exchange. Even in the developed countries, maintenance is a relatively labour-intensive operation. The operations are simple and can be efficiently carried out by manual methods. Even regrading can be done by animal-drawn scrapers or graders. On the other hand, many rural roads are built to a width greater than justified by the projected traffic flow to allow a mechanical grader to maintain it. This ignores the fact that there are labour-based methods available and also assumes that there will be a mechanical grader available to maintain the road.

For simple routine maintenance, the scheme of having a "maintenance man" or group has been used in many parts of the world. Supervised effectively, and provided the worker (or group) is given the right incentives, this can be very effective. It has the advantage that the responsibility for maintenance is easily defined. This means not only that there can be effective supervision, but that the local people also know who is to blame if the road is improperly maintained. Often the worker or group can be given a certain minimum kit of tools (wheelbarrow, pick and shovel) to execute the work.

One reason for the lack of emphasis on maintenance is its limited cost in relation to construction and vehicle operating costs. It is also true, however, that whereas a great deal of work has been done on quantifying vehicle operating costs, little attention has been paid to maintenance costs. There are probably as many maintenance costs formulae as there are countries in the world. Moreover, most of them are based on relatively poor data. Whilst it is our intuitive feeling that labour-based methods could and should be used in maintenance operations, it is clear that a more rational data base is required, so that the options can be quantified.

Project Appraisal

At some stage in choosing the most appropriate technology for a road construction project, it is necessary to make a cost comparison, for in the final analysis it is on the basis of cost, in its broadest sense, that the choice will be made. It is extremely important, therefore, that the cost estimates accurately reflect the use of resources.

Of the data required for a reasonable evaluation, two problems are of particular importance in developing countries : (i) unreliable statistical data, and (ii) use of inappropriate costing formulae, especially in relation to equipment.

The first factor is fairly common and the obvious remedial measure is the development of a reliable data base. In the case of construction costs, reliable data is particularly needed for the estimation of hourly costs for various types of equipment. One should beware of using the manufacturer's suggested unit costs, however, since conditions under which equipment is operated in developing countries are very much different from those in industrialised countries. The manufacturer's suggested unit costs are, in general, lower than those prevalent in most developing countries. One main reason for this is that equipment yearly utilisation rates are usually much higher in industrialised countries than in developing countries (9).

The use of inappropriate formulae also seriously affects the reliability of estimated costs. In particular, many contractors and public works departments make use of inadequate formulae for the estimation of equipment depreciation costs. Often, these formulae lower these costs, and therefore favour the adoption of capital-intensive technologies.

The Use of Accounting Prices

The studies in Iran, Thailand and the Philippines (2, 3, 4) involved the use of social or accounting prices. For those not familiar with this system, a brief explanation is necessary. For some time, economists have suggested that in developing countries the true scarcity or surplus of resources is not reflected accurately by their market price. In short, the cost does not reflect supply and demand. To counteract this distortion, a system of shadow or opportunity costs has been developed which more accurately reflects the costs of these resources to society. Many books and articles have been written on this subject and the calculation of these shadow prices is now well understood. In general, the shadow price of labour is lower than the market wage and the opportunity cost of capital is higher than the market rates of interest and foreign exchange. When projects are costed using these shadow prices the bias towards capital-intensive methods is removed and a true indication is given of the social feasibility of various alternative methods of construction. Put another way, it may be that labour-based methods are more costly when evaluated at market prices. However, if one then takes into account such factors as income distribution, employment and the relative values of consumption now against consumption at some later date, it is possible that these methods are socially more desirable. Naturally, even though project evaluation is carried out using shadow prices, the government still has to pay the market cost of the project. If a labour-based alternative costs more than the capital-intensive at market prices, it is then the government's choice whether the increase in employment and redistribution of income towards the lower paid justifies the payment of this subsidy. Social cost-benefit analysis merely quantifies the choice so that it can be used in more accurately evaluating the use of resources.

Project Management

The process of management consists of seven recognised processes which can be grouped under two main headings viz.: planning and executive functions. In very broad terms, the planning functions (forecasting, planning and organising) deal with material things whilst the executive functions (motivation, controlling and co-ordinating) deal with the human aspects of operations. The seventh process, communication, ties all the other functions together. One of the principal functions of management is to motivate the members of the organisation. In the industrialised countries, this generally refers to projects utilising large fleets of equipment. If one replaces the major pieces of equipment by large bodies of men then it is clear that a certain amount of re-thinking has to take place in relation to the various management functions defined above. The following sections discuss what reorientation may be necessary in the light of the seven basic management functions.

Forecasting

By clearly defining the objectives of the project in terms of progress, duration and quality it is possibly to specify in some detail the level of resources required at any particular time or location. This, of course, is a routine activity on any project. However, on a project involving the use of labour-based methods the forecasting function is given an additional role to play. It is first necessary to predict what will be the demand for labour. As in the demand for equipment, this will be based on the level of productivity expected. In recent years, a sufficiently sound body of knowledge has been developed for estimates to be made for most labour-based construction activities. There are, in addition, various methods of making an assessment of productivity rates (9). Apart from the reliability of data, the problems of forecasting labour demand are not severe. Labour supply is more complex. At the project level, it is necessary to assess the availability of labour for each activity in relation to the seasonal fluctuations of labour supply and the demand of other projects in the area. The project manager must have a clear idea of the relative levels of labour supply and demand so that the detailed plans can allow for any shortfall in supply and either defer activities until there is sufficient labour or used equipment for these activities. In its broader sense, forecasting in the case of labour-based construction also means an assessment of the needs of the large labour force. The recreational, social, health and welfare facilities that must be provided need to be assessed at the initial planning stage. Moreover, the number of supervisory personnel required will have repercussions in relation to any existing training programmes.

Planning

One of the major problems of using labour-based methods in road construction is that the flow of work has to be relatively even and not subject to large or frequent changes. A relatively stable labour force is more effective because morale is not reduced by constant hiring and firing and because management problems are reduced. Further, delays in projects that are equipment-intensive can often be easily dealt with by bringing extra machines on to the site. This is, often, simply not possible with labour-based methods owing to the number of workers that are required to do the same work as a machine. It is imperative therefore that (i) the project is planned in such a way that the labour demand does not vary enormously and (ii) that there is an even flow of resources to the project. As far as an even level of output is concerned, there are of course various techniques such as critical path network, PERT, bar or Gantt charts or the more recent Time and Location Chart, which can be used to arrange the activities in the optimum way. Once the basic requirement of a relatively steady labour demand is specified, these techniques are then used in the normal way. These techniques can also be used to ensure the effective integration of labour-based and equipment-intensive techniques.

The even flow of resources to a project is perhaps more difficult to achieve because the project manager is dependent upon external factors. Nevertheless, it will be possible to assess what resources are required at any time during the duration of the project. Of specific importance is the supply of hand tools and any planning process should provide some indication, based on the life of each type of tool, of when deliveries will be required.

A planning system relies for its effectiveness on the reliability of the data used and the efficiency of the reporting systems which allow the system to be up-dated. A good planning system can provide the basis for effective control of the project. In the case of labour based projects, which are particularly dependent upon an even flow of output, a system which will ensure a steady flow of resources is vital.

Organisation

Having predicted the requirements and planned their use the next step, prior to implementation, is to provide an organisation structure capable of executing the project. It can be argued that if labour-based methods are to be used the whole organisation structure used for equipment-intensive projects must be changed. Whilst there is some evidence from China that this can work, it is certainly not yet proven in general. Our experience has been that what is required is an adjustment of the structure to take account of the fact that the main resource being employed is labour not equipment. This means that gang size and distribution is extremely important. So also is the question of whether the gangs are arranged along functional lines or whether each gang or group of gangs is given a certain section of the road to deal with. Equally as important as the balance and distribution of gangs however is the type and level of supervision. How many workers can one gang leader effectively control? How many gang leaders can one foreman direct? The organisation structure can be extremely important when motivation and co-ordination are considered. In the former case, the workers must feel that they are in touch with the project management and the objectives of the project. In the latter case, good co-ordination is a function of communication which, in turn, is dependent upon an effective organisation.

Motivation

If a large labour force is to be used it is of paramount important that they are motivated to achieve their potential in terms of output. It is fair to say that in most countries financial reward is the main motiviation for good performance. The ILO studies have clearly indicated that workers paid on a piece-rate system produce much higher output than under a daily paid system. The system of pay, however, if it is to be a good motivator must not only be fair but must appear to be fair. In the case of task or piece rate, the targets set must be scrupulously defined to ensure that there is no exploitation of the worker.

Money, however, is not the only motivator. A sense of belonging, good relations with fellow workers and adequate social and welfare facilities are others. A good project manager will ensure that all the aspects are well taken car of. Furthermore, the initial system of recruitment must be seen to be just so that there is no friction caused between the worker who receives a job and the less fortunate person who does not.

Controlling

In the discussion on planning, we noted that a good system will ease the problems of control. The physical and financial control of projects can be relatively simple as long as the original plan is constantly up-dated by means of data derived through an effective communication system. The supervisory staff must, therefore, be trained to provide clear and concise information regarding attendance, output and future requirements. An effective communication system must exist which allows this information to be channelled to the project management for evaluation, decision and action. Not only will this ensure that the project is provided with the resources required at the right time, but also it will pin-point inefficient working or work that is behind schedule. This can be immediately investigated. In this way, a good control system can instill in the workers a feeling of participation.

Co-ordination

Labour-based methods require a greater number of supervisory staff. The methods may also be integrated with the use of equipment. To ensure that integration is possible and that the large labour force is working as a coherent mass, not as a set of independent units, it will be necessary to bring the supervisory staff together at regular intervals to discuss the work programme. This would also be the time when labour relations problems could be discussed.

Finally, no better summary of the problems involved can be provided than the words of a famous writer on construction management (10). "Not only is it necessary for a manager to be able to organise technologies but he must also be capable of organising individuals. He must be able to view the management activity as a whole and not from the bias of his own particular education and training. In the organisation of human beings, it is necessary for the manager to realise that improvements in the workers' physical and social conditions together with the sharing of a sense of participation in the enterprise are the two effective prime movers of effective performance."

Policies for larger scale implementation

In the previous pages, the problems of using labour-based methods in road construction have been discussed in detail. In the long term, one is concerned with the most appropriate use of resources in the construction of roads in the developing countries. The use of appropriate technology is not an emergency measure to counteract unemployment. The use of the most appropriate technology will automatically reduce unemployment because it will be using the available resources including labour, in the most appropriate manner. It is, therefore, not merely a question of providing a proper framework of design, planning and management. The long-term, large-scale implementation of appropriate technology in road construction, as in other sectors, will require that various policy measures are adopted in keeping with the shift away from the technologies which are patently inappropriate but for various reasons are presently preferred (13).

Fiscal Policies

In section 5, the use of social cost benefit analysis was discussed. This is a tool which can be used to take account of the distortion of market prices in developing countries. This distortion tends to favour capital-intensive methods. The use of accounting or shadow prices does not make any change to the actual prices, it merely takes account of the distortion to give a more rational allocation of resources. In the public sector, the assessment of the most socially profitable techniques can be made on the basis of the shadow price coefficients provided by a central planning agency. The manager in the public sector then executes projects up to the maximum number his market budget will allow based on the accounting prices. In the private sector, however, the use of social accounting would be extremely difficult to implement. It would definitely require some form of government intervention to ensure that the contractor perceives that there is no loss in profit if he uses the alternative technologies. In the long term, however, the use of these technologies should prove to be profitable and a great deal depends upon educating the private sector in the use of labour-based methods. The government can, however, take various financial and fiscal measures which would assist in the acceptance and implementation of appropriate technology in road construction. These range from the very simple to the very complex.

Surcharge - one very simple method for allowing for the distrotion of prices is to accept that it exists, to recognise that it favours equipment-intensive methods and to take the labour-based alternative where it is technically competitive as long as the market price is not more than a certain percentage more expensive than the capital-intensive methods. In the Philippines this has been done and labour-based methods are accepted if the increase in cost is not more than 10%. In a way, this is an implicit social accounting procedure as it implies that the distortion in market prices is of the order of 10%. This may seem crude and would, of course, require substantiation. However, given the complexity of the calculations to provide social prices it may administratively be the most acceptable.

Tariffs on imported equipment - in an effort to promote import substitution and to protect domestic consumer goods industries, many developing countries have adopted systems which provide high tariffs for consumer goods and low tariff rates for capital goods and raw materials. This has the effect of allowing the importation of construction equipment at low tariff rates and thus effectively distorting the real cost to the economy of using the equipment. There are various ways in which this problem can be solved. One is to provide discriminatory tariffs on equipment to be used on activities that can be carried out labour-intensively. This may be extremely difficult to implement in practice as a machine may carry out various functions some of which could not be done labour-intensively. An alternative is to make the importation of equipment dependent upon the provision of a licence which would be provided by the Government only after it was convinced that the equipment was needed.

Adjustment of the market rate of interest - many governments impose low ceilings on interest rates in order to prevent private banks from exploiting their monopoly power. However, if the capital market does not function properly low ceilings may have negative effects. Small investors are often unable to borrow money since the banking institution prefer to deal with the modern organised sector, where risks of default seem to be lower. Under these conditions, large contractors can easily borrow money to import construction equipment, and find it profitable to do so since the low ceilings on interest rates result in a decrease in the equipment rental rate. In some cases, moreover, given high inflation rates, low ceilings on interest rates imply a negative real interest rate on loans, and imports of equipment and the adoption of capital-intensive technologies become even more attractive. The proper government policy if there are low ceilings on interest rates is to remove the ceilings completely or to set them at much higher levels. High ceilings on interest rates will lead contractors to think twice before asking for loans in order to import construction equipment, and may thus induce them to reconsider their position with respect to efficient labour-based technologies.

Taxation of owned capital equipment - a number of factors, including low ceilings on interest rates, difficulty of importing spare parts for construction equipment, and lack of proper equipment for repair and maintenance facilities, tend to make contractors import more pieces of equipment than may be stricly necessary in order to be sure that construction work will not be slowed as a result of lack of equipment. The government may put an end to this situation by imposing a tax on owned capital equipment. The cost of keeping construction equipment idle will therefore increase, and contractors will refrain from importing more equipment than is really needed. Furthermore, the imposition of such a tax will increase equipment rental rates. A tax on capital equipment is, however, difficult to administer. The government must annually assess the value of pieces of equipment at various degrees of depreciation. Needless to say, such an assessment is difficult to make and may lead to a large amount of litigation between contractors and the government. This type of government intervention is, therefore, not very practical and should not be attempted unless other types of action are politically or economically infeasible.

Wage policies - The question of the most appropriate measures in relation to wages is perhaps the most difficult of all. This is partly because the difference between the market and shadow wage is often so marked. Other issues such as minimum wage legislation and trade union activity also have an effect.

Action in relation to wages is often seen as an effective way of favouring labour-based methods. All too often, however, the action is counter-productive because it directly interferes with market supply and demand. For example, the use of wage subsidies has been advocated. In its simplest terms this is the payment, by the government, to the contractor of a subsidy for each man-day of unskilled labour used on a project. Unfortunately, the result of this is that the contractor can continue to use capital-intensive methods, the increased labour force stands and watches, and the contractor pockets the subsidy (14). It is not sufficient to make it financially feasible for contractors to use methods which require more labour. They must also be convinced of the technical and managerial efficiency of these methods.

Finally, in regard to fiscal policies, a comment from the Iran study (2) is particularly apt. "The argument concerning the relative merits of specific tariffs, quantity restrictions and wage subsidies does not turn on any simple notion of economies in government finance. The issue is basically one of ease of administration and relative efficiency of the set of measures adopted. Tariffs or quantity restrictions would appear to have a certain appeal as they are more easily administered than a direct wage subsidy. By themselves, however, they will not guarantee the adoption of appropriate technology. In the final analysis it will be the practitioners who choose the technology and any fiscal measures must go hand in hand with the appreciation of the technical feasibility of using appropriate technological methods."

Policies on standards and specifications

The ILO's employment strategy mission to Sri Lanka noted that "the problems in the construction industry were aggravated by the stolid enforcement of regulations, professional codes of practice or contractual procedures, the inadequacies of which have long been recognised in the very industrialised countries from which they were copied; copied moreover in the face of unquestionable evidence of their ineffectiveness in local conditions". The situation in Sri Lanka is probably no different from that in many other developing countries. The unsuitability of European standards and specifications is most blatantly illustrated by such examples as the application of snow-loading criteria for roof construction in tropical Africa. Less obvious, but more common, however, is the specification of materials, methods of working and testing procedures which would be perfectly appropriate in Europe and North America but not in the developing countries.

There is no suggestion here that the systems and procedures used in the construction industries of developed countries are inefficient. Within their own environment, they are generally effective. To expect them to work as efficiently in an environment where the basic economic situation is totally different, where the administrative systems are limited and where the information flow to the local construction sector is poor, is somewhat optimistic. The basic dichotomy in construction caused by the division of responsibilities for design and construction becomes an even greater problem in countries where contractors lack even the basic financial and managerial expertise.

In regard to specifications and codes of practice, therefore, governments need do no more than make an investigation of whether they are orientated towards the environment in which they are supposed to operate.

As regards methods of tendering it is clear that certain aspects of the tendering procedures are biased towards the use of equipment. One can think of the selected tender where only those with, among other things, a minimum plant holding will be eligible to tender. As far as government is concerned, what is recommended is a more flexible approach to tendering procedures. These procedures were developed in the industrialised countries where there is an inherent tradition of contract responsibility and where the design, construction methods and procurement of materials are already institutionalised. They may apply also to vast development projects: however, it would be surprising if they are wholly appropriate to construction industries whose members have extremly limited financial and managerial resources.

The influence of foreign consultants and contractors

If one accepts that infrastructure development is vital to economic development, it almost goes without saying that this implies some form of foreign involvement in the construction industry. This may merely be in the form of feasibility studies or managerial assistance. More commonly, it will involve the use of foreign consultants and contractors. Properly arranged, their involvement can be mutually profitable. Unfortunately, certain conflicts often present themselves. As the ILO's study in Iran showed (2), foreign firms will draw their technology from an international market in whcch innovation has consistently moved in the direction of equipment-intensive methods. As the consulting engineer is often involved at the feasibility, design and supervision stages, the technology used is dictated by his expertise and background and all too rarely by a detailed consideration of the most appropriate resource allocation. In addition, one has a contractual system based on the industrial country practice, and a contractor who, if not foreign or foreign-owned, will have been schooled in the efficient use of equipment; it is not surprising therefore that the technology chosen is capital-intensive.

The use of foreign expertise in the development of a country's infrastructure is a fact of life and has many beneficial effects. Properly handled foreign firms can be a great asset in assisting in the growth of the local construction sector. It would be wrong, however, to view the activities of foreign firms in any philosophical way. Principally, their object is to make a profit. Whilst governments should support this objective it should also ensure that profit is generated in a beneficial way for the country. This means that when the government, as a client, hires a consultant to carry out a feasibility study for a road project, it makes it quite clear that the study should assess the construction technology to be used. It means that in its dealing with foreign contractors it should insist that labour is used to the fullest extent compatible with technical and economic efficiency whilst the importation of equipment is kept to a minimum. Governments can also take measures to support the growth of local contractors. This can be done not only by preferred treatment but also by putting money into training courses on financial management, administration and the most appropriate construction techniques. In the long term, each developed country must have a viable domestic construction industry, whether public or private,for its own economic stability.

Training and Education

The use of labour-based technologies would require managerial and supervisory skills significantly different from those required for equipment-intensive techniques. In addition, the education of engineers would have to place much greater emphasis on the use of appropriate technology. These two separate but interdependent issues are of vital importance. For, if one cannot convince the practitioners, and if one does not have the requisite level of supervisory staff, the use of construction methods which rely heavily on labour will be merely an interesting idea.

As far as training is concerned, it is still not clear whether it is better to have separate training courses for labour-based programmes or to attempt to integrate this training into the normal curriculum for foremen and supervisors. What is clear is that labour-based methods do require a higher degree of supervision than machine-intensive projects and thus governments will need to invest heavily in training.

As far as education is concerned, it has been recognised for some time that the civil engineering courses of many universities in developing countries appear unrelated to the environment in which the engineer will work. They tend to be carbon copies of courses in developed countries and the suggestion that they should be re-orientated to the needs of the developing countries is often interpreted as being requests to turn back the clock. However the developed countries use capital-intensive technologies because they are rich; they are not rich because they use capital-intensive technology. There is, of course, a need for engineers to understand sophisticated techniques and analysis. Indeed, these are vital to the effective management of labour-based schemes. There is also an onus upon civil engineering faculties to provide courses which will give students the capability to make the most appropriate use of the resources at their disposal.

At the end of a fairly detailed coverage of the problems of the most appropriate technologies in road construction the overall problem of what are the required policy measures for large-scale implementation appears particularly broad. In general, however, what is required is a flexibility of approach, firm direction and support from the highest level and motivation for all concerned in the road construction process so that the most appropriate methods in terms of resource allocation are also the most efficient in the execution of projects.

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APPROPRIATE TECHNOLOGY AND LOW COST TRANSPORT

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This paper is concerned with the provision of appropriate transport facilities in the rural areas of developing countries. It is argued that the technologies applied in the past have been inappropriate to, and ineffective in meeting the transport needs of their poorest people. Further, that there are alternative and more appropriate transport technologies which can better meet many of these needs. Past transport strategy has been dominated by an institutional preoccupation with the provision of roads suitable for conventional motor vehicles. The supply of those vehicles has been left largely to the private sector and their technological appropriateness unquestioned to the extent that the type of vehicle is not a variable in road design. The result has been high road construction costs, slow network development, and the neglect of the movement needs of small scale farmers and of traditional forms of transport. A range of basic vehicles is described whose technology is shown to be more appropriate to the needs of many rural developing communities. It is suggested that attention should be focussed on improving the technology of basic vehicles with a corresponding re-appraisal of track requirements. The application of a more appropriate technology requires that rural transport planning should explicitly include an appraisal of the small farmer's movement needs and the constraints within which these must be met.

This paper is specifically concerned with low cost transport *in developing countries.* Since 1969 the International Labour Office and the World Bank have challenged many of the orthodox views about the type of technology most appropriate for the construction of roads in developing countries. We go further to consider not just the construction of the road, but to question the whole notion of the most appropriate forms of *low-cost transport* for developing countries and the scope for technologies intermediate between the traditional and those introduced from the developing world.

In most developing countries transport has generally received more resources in the past three decades than any other sector, yet in the same period the number of persons living in absolute poverty has increased and their conditions of life worsened (1). Whilst past transport policies may not have caused the increase in poverty, they have not prevented it. The implication is that a *continuation of past strategies is unlikely to alleviate or improve the conditions of the poor.* For this reason it is important to examine past transport strategy to determine in what way it might be modified to better serve the needs of the poor.

Past Transport Strategy

The 'rural transport problem' in developing countries has been seen as one of providing or improving the quality of access. The term *access* has meant almost exclusively 'road access'. Past road investments have favoured the construction or improvement of major rural highways rather than urban or minor rural roads, although this emphasis is changing slowly (2). The only recognition that developing countries might have special rural transport problems has been the effort devoted to the subject of 'low-cost roads'. Until 1976 there had been no comparable effort devoted to 'low-cost vehicles' or to any other type of road vehicle particular to developing country needs (3, 4).

In retrospect this omission is very odd. Transport comprises a system: some form of vehicle and a 'track or route upon which that vehicle moves'. We have expended considerable time, money and effort in seeking to optimise the one to the almost total exclusion of the other.

Road Design Standards

Concepts such as 'design speed','safe stopping sight distance' and many others that underlie the geometric design of modern highways have never been satisfactorily interpreted for developing country needs. One of the few recognised text-books on road design for developing countries recommends essentially US standards - probably the most generous ever developed - for the geometric elements of 'low-cost' highways (5). Moreover, the design, and hence cost, of a modern road is predicated upon the performance characteristics of the private car and goods lorry: the assumed speed that car drivers 'desire' dictates the standards of horizontal and vertical curvature, and the expected axle load of goods lorries dictates the road's structural strength. Yet neither of these vehicles has been shown to be essential, much less optimum, for rural developing regions. In most developing countries the current level of motor vehicles per head of population is extremely low (Table 1) (29) and the expectation is that it is unlikely to increase very rapidly. A UNIDO study of 93 developing countries (6) showed that in 1968 average motor vehicle density was 9.2 units/1000 population. By 1980 it was forecast that this will have risen to only 11.8 units/1000 population.

For rural societies, simpler and cheaper vehicles might be more appropriate. Slower and lighter vehicles would allow the alignment, strength, and width of roads to be reduced, with, potentially, a considerable saving in costs.

The usual argument against such a course of action is that by so doing user costs would rise, leading to an increase in the total cost of transportation. However, the appeal of this argument is superficial. It has merit only because we are unable to account fully for the consequences of road improvements, a situation which artificially exaggerates the importance of those benefits we can quantify. Time savings are not taken into account with current techniques for the economic appraisal of road improvements, (7) and by general admission the so-called 'secondary' or developmental and social benefits are as yet unquantifiable. In fact the most recent evidence suggests that the secondary effects of transport investments in isolation - such as the building of rural highways - may in fact be *harmful* to certain sections of the community. Studies (8, 9) have shown that benefits resulting from transport improvements tend to accrue to the already advantaged. Often there are also appreciable dis-benefits which are usually overlooked and, invariably, affect the poorest groups. Examples found were:

1. Reductions in labour demands following the replacement of traditional movement methods by motor vehicles.
2. Concentration of capital as cottage industries are bankrupted by the increased competition from larger-scale industries in the towns.
3. Changes in agricultural production towards transport-intensive products which do not necessarily benefit the poor.
4. Concentration of land holdings.

Road Construction Costs

A direct consequence of road design for use by conventional motor vehicles is that construction costs remain high, particularly when equated with the resources of the poorer developing countries. The World Bank recognises three classes of 'rural roads', by which is meant low volume, feeder or tertiary roads (2). These are:

Class I — major roads which also fulfil principally a rural access (as opposed to inter-urban) function but which cannot be classified as feeder roads because of their regional importance. Approximate costs: \$20,000-\$350,000 per km.

Class II — Feeder roads connecting villages and small markets with larger regional centres and/or major transport arteries. Approximate costs: \$10,000-\$100,000 per km.

Class III: — Farm-to-market roads representing the lowest class of roads available for transport and normally linking a number of farms to the closest market/administrative centre or transport artery. Approximate costs: \$5,000-\$25,000 per km. for major construction not minor upgrading.

If one compares the figures in Table 2 (30) with the construction costs for rural roads it is apparent that many of the poorer countries can afford only relatively insignificant additions to existing road systems. Consider the case of Sierra Leone. If *all* the current annual expenditure on highways were concentrated on Class I rural roads then it could afford between 5 and 80 km of road per year, which would add between 0.2 and 2.6 per cent to the length of the present system. The significance of this is that in most developing countries the density of the *existing* road network is very sparse (Table 3) (30) and lags far behind the rest of the world. In practice the overall length added to the sytem would be *lower* than the figures quoted above since part of the expenditure would be on more expensive primary highways.

Unless the cost of road building can be drastically lowered, then for most of the poorest countries road network development will be extremely slow. A reduction in construction costs is unlikely so long as road design is necessarily equated to use by conventional motor vehicles.

The Role of Conventional Motor Vehicles

For many years a belief has been fostered in what might be termed the 'economies of modernity'. The figures in Table 2 illustrate the foundations of this belief (10). The implication is that 'primitive is slow and expensive' and 'modern is fast and cheap'. But the comparisons assume either full loads or equal load factors, (i.e. similar degrees of utilisation) not just in the short term, but over long periods of a year or more. Is this likely? It is not always clear if the figures are costs or charges to the user. If they are costs, are they based on market or economic prices? Has any allowance been made for the fact that it costs society nothing for the track over which loads are carried on the head, by mule or bicycle, but a road suitable for motor vehicles will cost several thousand dollars per kilometre to build and several hundred a year to maintain? What, if anything, are the employment consequences of substitution among different modes of transport? Are all modes of transport equally possible: will the terrain, length of journey, size of consignment, etc. allow a free choice between them. *Most important, are they equally available and affordable to all would-be users?*

In reality different modes of transport serve characteristically different movement demands, yet supposed unit costs are the most common basis of comparison in analyses of the suitability of transport modes.

Government and Institutional Involvement in Road Transport

Government involvement in the provision of motor vehicles has, by and large, been regulatory: either permitting reasonably free import or making it very difficult where foreign exchange has been an acute and continual problem (e.g. Burma, Bangladesh, Sri Lanka and Tanzania); although some have participated in capitalizing production ventures, the bulk of the capital has come from international motor vehicle suppliers. In developing road transport the implicit assumption has been that the private sector would supply whatever vehicles were necessary to make efficient use of the roads provided by the government. That this supply would appear has been taken for granted and that it might not be appropriate hardly considered. There are countries that have simultaneously made significant (public) investment in highways combined with severe restrictions on the (private) import of vehicles: the latter restrictions negate the possibilities of receiving benefits from the former investments.

Few developing countries - China, India and Papua New Guinea (11) being exceptions - have attempted to restrict the number and type of vehicles to those considered appropriate to their stage of development. Restrictions because of foreign-exchange considerations or the desire for local manufacture are not uncommon: but restrictions because of alleged technological inappropriateness are.

Transport Needs of the Poor

The available evidence (12, 13, 14, 15) suggests that typically the poor:

1. Are engaged in agriculture working small plots of land either for themselves or as landless workers.
2. Are engaged in subsistence farming or generate only small marketable surpluses.
3. Have a family cash income of only a few tens of dollars a year.
4. Are poorly served by almost all public amenities including transport.

The most significant transport needs of the poor are those which relate to agricultural activities, since it is through the generation of marketable surpluses (and thus income) that other goods and services become affordable. However, transport needs *at the farm level* have very rarely been studied. Roadside surveys of the commodities carried by motor vehicles are a poor substitute since (i) they are too far along the marketing chain to be able to isolate individual consignments and the distance over which they are being moved; (ii) none of the studies have been sufficiently extensive to give any adequate measure of seasonal fluctuations in travel; and (iii) they give no indication of on-farm transport needs.

One farm level study was carried out in Kenya by the World Bank in 1976 (16, 17). This suggested that most transport needs could be characterised as the movement of small loads (10-150kg units) over relatively short distances (1-25km). On-farm the range of loads was likely to be the same, but the typical distances were shorter (1-13km). The amounts of water and wood required for household use were noteworthy (50 and 30kg respectively), since it was estimated that their collection occupied 3-6 hours per day.

Since the farmer must follow a fairly rigid schedule to obtain optimum yields, it is important that on-farm transport for crop production and household needs should not be so time-consuming as to delay operations. For example, if some crops are not sprayed on time the results may be disastrous. Yet the spraying of cotton with insecticide required about 200-300 litres of water/hectare. For a four hectare plot this is 800-1200 litres and between 7-10 sprayings are normally recommended, i.e. between 6-12 tonnes of water, a formidable amount if headloading is the only available means of transport (17). In the studies in Kenya it was concluded that on-farm transport was already a burden if not an outright constraint on small farm activities.

Off-farm transport comprises two elements: between farm and roadside, and between roadside and collection point/market. An example of how large these elements can be is given by the definition in a study of Nepalese peasant agriculture (8) of the terms 'on road' to mean at the roadside or within a few hundred metres, 'near road' to mean up to half a day's walk from the road, and 'off-road' to mean more than half a day's walk.

There is a dearth of information about the magnitude, frequency and duration of the small farmer's movement needs. What is clear, however, is that the transport requirements can be substantial, even when only small areas are planted. Table 5 presents data from Malawi on the yields and inputs per hectare for different crops (18).

Discussion

Recent rural transport strategy has pursued development from the top downwards using developed country technology. That is, a progression from major primary, to secondary and only latterly to tertiary highways all built on the basis of design philosophies imported from the developed countries: equally, a reliance on developed country motor-vehicles with only very recently (3) a small step in the direction of lower cost, but still motorised, vehicles and the complete neglect of traditional forms of transport. The result is skeletal road networks that in the poorer countries plainly do not serve effectively the majority of the population, and vehicles so expensive that they are beyond the means of all but the affluent. Moreover, past transport planning has failed to recognise that many people live remote from the (motor-vehicle) road system, and have movement needs that could never be satisfied by conventional vehicles.

Thus for the rural poor it would be difficult to conceive of a more inappropriate technology: often unrelated to basic movement needs, inaccessible, scarce, expensive, difficult to use and maintain, and frequently wholly dependent on foreign resources in terms of manufacture, energy, spare parts and operating skills.

To define a more appropriate *transport* technology it is necessary to ask the question: 'What are the appropriate *vehicles* for rural areas of developing countries'?

Appropriate Vehicle Technology

Most attempts at defining 'appropriate technologies for developing countries' agree on the necessity of meeting 'local needs' and the requirements that they be employment generating, compatible with incomes, and capable of manufacture and maintenance using indigenous skills and resources.

Given the variations in incomes; in topographical, road, farming and social systems; and in local resources and capabilities, there cannot be '*a universal vehicle*' appropriate to all the rural transport needs of developing countries. *Rather, the need is for a graduated choice of vehicles whose performance matches need and whose cost is in sensible relation to income.*

Consideration of the characteristics of the rural poor, their transport needs and the criteria of an appropriate technology leads to vehicles radically different in concept from conventional motor vehicles. The consequences of variations in operating environment, loads, cost, technical simplicity and the use of local resources lead to a progression of human, animal and, at the extreme, simple motorised means of movement. We term these collectively as *basic vehicles*, which may be defined as:

the range of devices from aids to goods movement by man himself up to but excluding, conventional cars, vans, buses and trucks.

Six categories of basic vehicles can be defined:

1. Aids to head, shoulder and backloading
2. Handcarts and wheelbarrows
3. Pedal driven vehicles
4. Animal transport
5. Motor cycles
6. Basic motorised vehicles

Many such basic vehicles already exist in different parts of the developing world, though often their use is localised. Some are primitive, being traditional devices which have remained unchanged for many years. Almost all are capable of improvement, using contemporary technical knowledge, so as to increase significantly their efficiency and usefulness.

Head, Shoulder and Back Loading

These are the most common methods of load-carrying in rural areas. Loads can be carried over steep, hilly or rocky terrain and aids to human porterage can usually be made at token cost by local people using available materials. However, human porterage is arduous physical work; it is also slow, and therefore time consuming. As a result loads are limited to about 40kg, and can only be moved short distances. There is also widespread concern that the habitual carrying of very heavy loads can cause physical disabilities and injuries. Yet for the foreseeable future human porterage is likely to remain an important means of rural transport. Therefore, efforts should be made to improve its efficiency and minimise its harmful effects. There have been few attempts to improve this means of transport. One notable attempt is the joint work of the Georgia Institute of Technology (USA) and Soong Jun University (Korea) on the Korean Chee-ke (19). The Chee-ke is a traditional form of back loading frame and "is very inefficient, difficult to handle, and very heavy when it is fully loaded. Nevertheless, in the light of Korea's hilly and rocky terrain, this piece of equipment can hardly be discarded (19) (Figure 1). Through a programme of research and development involving farmers, rural blacksmiths, traditional chee-ke makers, specialists in farm equipment and engineers, an improved chee-ke was produced. Six successive models were evolved before a satisfactory design was achieved (Figure 2). The improved chee-ke converts easily from a back-frame to a wheeled carrier.

Wheelbarrows

Except in China, the wheelbarrow does not appear to be widely used for rural goods movement. The Chinese wheelbarrow is of quite different design from the Western wheelbarrow found in most other parts of the world (20).

The Western wheelbarrow has a relatively small diameter (up to 400mm) wheel positioned below the sloping front of the load tray. The centre of gravity of the load is well behind the line of the wheel axle.

The Chinese wheelbarrow uses a larger diameter wheel (about 700mm) with the load placed directly above it on a horizontal platform so that the centre of gravity is just behind the wheel axle (Figure 3).

The Chinese wheelbarrow is a more effective device than the Western type. Because the load is placed close to the wheel-axle the operator only has to support a small proportion of the load, sufficient to maintain control of the barrow. Thus more of his energy can be devoted to propelling the barrow forward. The large diameter wheel reduces rolling resistance. The tendency of the barrow to tip sideways is mitigated by the use of a shoulder strap attached to the handles of the barrow.

Studies carried out by the World Bank showed that the maximum load for a Chinese barrow was about 180kg, compared with about 120kg for the Western configuration (21).

Pedal Driven Vehicles

Bicycles are widely used in developing countries. They are relatively cheap (US$60-100), rugged and easy to use and maintain. They can carry a passenger or up to about 80kg of cargo, although the bicycle can become difficult to control when heavily loaded. Bicycles offer a significant increase in speed of travel over walking, average journey speeds of 16 km/hr being common, and can operate on narrow paths and tracks.

The type of bicycle which predominates in developing countries has not changed in any significant way for many years and is typical of the designs produced in Western countries thirty or forty years ago. It remains popular because its robustness and longevity better meet the needs of developing countries than more 'modern' designs. Its popularity reflects the fact that bicycles are used in a very different way in developing countries. They are habitually used to carry passengers and/or heavy cargo loads, on rough, unsurfaced tracks and paths, and are expected to stand up to arduous use for many years. They are a basic load carrier for rural areas rather than a convenient means of short distance personal transport. *Yet no bicycle has ever been designed to meet these very different operational requirements* (Figure 4).

There is a need for such a bicycle, specifically for use in developing countries, designed to carry a passenger and/or cargo and to be suitable for small scale local *manufacture* (Figure 5).

The load carrying capacity of a bicycle can be increased at low cost by attaching a trailer. Cycle trailers are widely used in many rural parts of Europe and are used by the Swiss Postal Service for the delivery of letters and parcels. However, except in French-speaking countries of Africa and Indo-China their use in developing countries is uncommon. This seems to be a case not of an inappropriate technology but of one that is *unknown*.

Animal Carts

Animal drawn carts are a major form of rural transport in the Asian region. In India it is estimated that they now number some 14 million and over 60 per cent of all goods carried from farm to market are moved by animal cart (22).

The traditional Asian cart costs US$100-180, has a maximum payload of about 1 tonne, and moves at 3.0-4.5 km/hr. It has large diameter iron-rimmed wooden wheels which cause damage to surfaced roads. To overcome this problem, manufacturers in India produce a pneumatic tyred ADV (animal drawn vehicle) wheel which runs on ball bearings and is fitted to a specially fabricated steel axle. The cost of a cart with a steel axle and ADV tyres is approximately twice that of the traditional vehicle (22). Such carts can carry loads of up to 2.5 tonnes, yet their penetration of the market has been very limited and use is concentrated in urban areas and in the affluent agricultural regions with relatively good roads.

The use of animal carts is less widespread in Africa, even in areas where animals are used for draught cultivation. Donkey carts, with a payload of about 400kg are relatively more important in Africa than in Asia and the use of steel wheels with pneumatic tyres appears to be more common.

There are major deficiencies in the design of existing carts so that the available power of the animals is used very inefficiently (Figure 6).

1. The carts are excessively heavy.
2. Many are badly balanced so that a significant portion of the load bears down on the necks of the animals.
3. The traditional bullock cart yoke is inefficient.
4. The traditional wooden wheel is heavy, and uses inefficient bearings. The pneumatic tyred wheel is expensive, causes maintenance problems and does not perform well in muddy conditions (23).

There is a need for a complete re-appraisal of cart design, leading to devices which utilise the energy of the animals efficiently. This would result in carts with increased carrying capacity, would mean that a given load could be moved with less effort and would offer the possibility, in many cases, of using only one bullock instead of two. It is likely therefore that the cost of transport would be decreased and its speed increased.

Basic Motorised Vehicles

The single axle tractor is used extensively in China. It has a 7.5kw single-cylinder diesel engine and in addition to its agricultural functions it can be hitched to a trailer and haul a payload of 1200kg at 15 km/hr (24) (Figure 7). A similar device with a 4-5kw petrol-engine has been developed at the International Rice Research Institute (IRRI) in the Philippines specifically to meet the needs of the many small-scale Asian rice farmers (25) The machine was designed to make maximum use of standard components that are readily available in most developing countries. The engine, roller chains, sprockets, bearings and seals used in the power tiller are imported in most Asian countries for other uses. The remaining components can be produced by small metalworking shops. The power tiller was introduced in 1972 and is now produced by 12 companies in six Asian countries. It is sold in the Philippines at (US$1000) approximately half that of comparable imported machines. In the first two years of manufacture in that country some 700 new jobs were created in the manufacturing sector at a capital investment of about US$200 per workplace.

IRRI have developed a three-wheeled self-propelled cart, with a single, driven and steered front wheel. It has a payload of 720kg and a top speed of 15 km/hr. In Crete three-wheeled rigid chassis vehicles powered by single cylinder diesel engines have been developed in the past few years (Figure 8). Their evolution appears to have resulted from the use of single-axle tractors and trailers for goods movement, leading to a demand for vehicles similar in concept but specifically designed for transport use. The vehicles are produced on the island by small-scale manufacturers.

Conclusion

There exists a range of 'basic vehicles' from simple aids to goods movement by man to cheap motorised forms of transport. *Their technology can be related to basic movement needs and they are accessible, available (or potentially so), in sensible relation to incomes, simple to use and maintain, and could utilise local resources in terms of manufacture, energy, spare parts and operating skills.*

The present status of basic vehicles is that much good technology already exists which could be widely applied, but whose use is at present very localised. Where information on such technologies exists it is obscure, uncollated and certainly unknown to those who could make use of it.

While devices which meet the transport needs of the rural poor must be simple and low cost, this does not imply that their development is an easy task. Rather, experience suggests that the development of effective basic vehicles requires the application of contemporary technical knowledge and the very best technological skills.

All the vehicles described could be operated on roads of a lower standard and hence cost, than that prescribed by the requirements of conventional motor vehicles. Some may be described as 'two-dimensional' in that they have height and length but no significant width. This makes them suitable for use on footpaths and narrow tracks.

The ideas outlined in this paper do not, as yet, enjoy wide currency and the application of more appropriate transport technology will require major changes in policy. The most fundamental change required is to ensure that rural transport planning explicitly includes an appraisal of the needs of the small farmer and the constraints within which *his* choice must be made. The implication is that the most appropriate type of vehicle and the 'track' it requires will be *issues to be decided by local circumstances* rather than to be externally imposed by the assumed use of conventional motor vehicles. A transport planning process which includes the appraisal of the needs of the small farmer will be very different from that currently practiced:

1. The first step would be a small-farmer specific analysis of the magnitude, frequency and duration of transport needs and of the distances over which movements were required.
2. Cognizance would need to be taken of the proximity and structure, (condition, degree of integration) of all existing routes (footpaths, tracks and roads) and motor vehicle services.
3. Consideration of (1) and (2), existing incomes and/or credit facilities, and attitudes towards different forms of transport would indicate the likely range of functionally and economically appropriate vehicles.
4. The consequences of (3) in terms of current availability, ease of manufacture and repair from local resources, and employment generation would then have to be evaluated.
5. Finally, a selection would be made of the vehicle(s)/route(s) combination that would best meet local needs and consideration given to what forms of assistance were necessary for it to be provided.

Two crucial elements of this process are: (a) greater flexibility in the methods of route design, and (b) the direct participation of Government and aid institutions in overcoming the problems associated with the provision of appropriate basic vehicles.

There is a need for developing countries to generate their own road design standards based on local conditions which would incorporate, as appropriate, the requirements imposed by basic vehicles. Road design has been based on the needs for motor vehicles for so long that there is little available experience of designing for anything else, at least in the developed countries. However, some developing countries have experience with the provision of routes for basic vehicles (26, 27, 28).

There seems to be no logical reason why governments and aid institutions should not play as dynamic a role in the provision of basic vehicles as they have done in the provision of roads. *Indeed it seems irrational for them to do otherwise, given that the track and vehicle are complementary and mutually dependent parts of the road transport system.*

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Table 1. Road vehicle statistics for selected countries (1975)

	Passenger Cars 000's A	Commercial Vehicles 000's B	Increase 1953-1975 per cent A	B	Motor Vehicles per 1000 persons	Commercial Motor Vehicles per 1000 persons
Botswana	3.4[1]	6.8[1]	610	920	15.2	10.1
Burma	36.1[1]	39.3[1]	210	320	2.3	1.2
Chad	5.8[2]	6.3[2]	1500	230	3.0	1.6
Ecuador	43.6[1]	68.4[1]	870	520	15.8	9.7
Ethiopia	41.0[3]	12.7[3]	680	330	1.9	0.4
Ghana	55.5[1]	43.9[1]	440	250	10.1	4.4
India	756.5	434.4	350	220	2.0	0.7
Indonesia	383.1	231.5	540	350	4.6	1.8
Kenya	130.9[1]	23.8[1]	350	160	11.6	1.8
Malagasy	55.0[1]	51.0[1]	480	320	12.0	5.8
Malawi	11.2[1]	9.5[1]	300	350	4.1	1.9
Mozambique	89.3[3]	21.5[3]	710	610	1.2	2.3
Sierra Leone	14.8	6.7	490	740	7.2	2.2
Somalia	8.0[3]	8.0[3]	670	230	5.0	2.5
Sri Lanka	91.7	48.6	80	150	10.3	3.6
Sudan	29.3[3]	21.2[3]	440	240	3.2	1.4
Tanzania	39.1[1]	42.3[1]	270	550	5.5	2.9
Thailand	286.2[1]	264.3[1]	1220	1020	13.1	6.3
Australia	5012	1200	350	110	460	89
Sweden	2760	171	540	50	357	21
UK	13949	1872	400	70	282	33
USA	106712	24837	130	170	616	116

[1] 1974, [2] 1973, [3] 1972.

Table 2. Annual expenditure on new road construction

Country	Year	Amount US $ (millions) at 1976 values
Benin	1973	4.8
Botswana	1975	7.4[1]
India	1974	64.8
Malawi	1975	9.0
Mali	1974	1.4
Mauritania	1971	4.9
Niger	1974	15.6
Sierra Leone	1975	1.6
Sri Lanka	1975	5.8
Thailand	1975	4.1

[1] including maintenance expenditure.

Table 4. Cost of transport by various means

Mode	Cost of Transport $ per tonne km.
Mule (on a track)	1.00 - 3.00
Landrover (on a trail)	1.00
Tractor (with trailer on an earth road)	0.50
Truck - on gravel road	0.25
- on asphalted road	1.10

Table 3. Road statistics for selected developing and developed countries

	Change in length of road[1] network 1950-75 (per cent)	Percent paved[2] 1975	Density[3] in km/ 100 km²	GNP per capita $
Angola	110	11	6	370
India	200	35	37	140
Indonesia	40	25	4	220
Kenya	50	8	8	220
Malawi	70	14	11	130
Mozambique	40	9	5	180
Nigeria	140	17	10	340
Sierra Leone	130	17	10	200
Sri Lanka	110	70	48	190
Thailand	460	48	7	350
Tunisia	-	52	12	730
Uganda	70	-	8	230
Upper Volta	-	9	2	110
Zambia	140	11	5	420
Germany	260	95	187	6670
France	80	-	144	5950
Italy	50	93	96	2810
Spain	-	80	28	2750
Poland	-	57	95	2600
UK	20	96	150	3780
USA	20	80	66	7120

[1] These figures must be treated with caution since some of the changes are due to alterations in the classification of what is a 'road'

[2] Roads with an all-weather surface of bitumen or concrete

[3] To obtain approximate average road spacing divide 200 km by the figures in this column.

Table 5. Quantities to be transported (kg/hectare) for different crops in Malawi

Item	Maize	Tobacco	Groundnuts	Cotton	Rice Rainfed	Rice Paddy	Coffee	Pulses
Inputs								
Seeds	45		80	30	65			35
Fertiliser	250	250			620	400		
Yield								
Average	1009	560	449	1120	1234	3362	335	449
Best	2804	2241	963	1881	3362	4708	1007	560

Figure 1. Traditional chee-ke and typical terrain.

Figure 2. Improved chee-ke.

Figure 3. Chinese wheelbarrow.

Figure 4. A basic load carrier.

Figure 5. Oxtrike.

Figure 6. Traditional ox-cart.

Figure 7. Chinese single-axle tractor

Figure 8. Cretian 3-wheeler.

A METHODOLOGY FOR EVALUATION OF RURAL ROADS IN THE CONTEXT OF DEVELOPMENT

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Despite a good rate of national growth, rural poverty is on the rise in many developing countries. Transportation, particularly roads, is perceived as an important component of rural development. In an effort to impart a more valid basis for selection among investments, an evaluation framework capable of accounting for the various socio-economic objectives of the rural development effort in the assessment of rural transport projects is formulated and preliminarily tested. A potentially appropriate set of developmental objectives is identified, and possible measures proposed. Utility assessment techniques are suggested for developing decision maker's preference functions, and ultimately scaling project contributions to the criteria. These scaled measures of the criteria for each project are then incorporated into a single value structure as a basis for project ranking and thus decision making. Depending upon the decision maker's access to information and articulation of his preferences among the criteria, equal or cardinal weights may be directly assigned to the criteria, or an ordinal ranking of them may be done and an upper or lower bound decision rule used. The ranking of the projects varies with the approach. The proposed appraisal framework is seen as a simple but valuable tool in the project selection stage where a decision maker faces an array of potential projects and needs some means for evaluating their relative worths. Although a case study has been carried out, testing under actual field conditions remains to be done. Moreover, this is a first step effort, and certain refinements are needed.

1. Introduction

1.1 Rural Development and Transportation

Despite a good rate of national growth in many developing countries in the past decade, rural poverty is increasing, and the number of people living on the margin of existence is rising. It is estimated that some 80 percent of the three-quarters of a billion poor in developing countries live in the rural areas. Moreover, the rural population is growing at a rate of some 2 percent per year in spite of rural migration to urban centers (1).

It has been asserted that through rural development many of the sufferings of this large number of rural poor may be allevaited. As stated in the World Bank's sector policy paper (1), the basic objectives of rural development might be defined as: productivity improvement; employment and income generation for target groups; and provision of minimum acceptable levels of food, shelter, education, and health. The fuller development of the existing resources, the building of infrastructure such as transport and irrigation works, the introduction of improved technology for existing agriculture or new crops, and the creation of new types of institutions and organizations are some of the tasks lying ahead.

Transportation, particularly roads, has been perceived as an especially important component of rural development, in terms of stimulating local development, providing access to social services, and generally serving to integrate the rural population into the overall economy. Trends in recent years have been increasingly away from investments in the primary road network and toward greater emphasis on the rural feeder and penetration roads. Moreover, there has been an increased tendency to regard even these low volume roads as but one of several possible investments competing for scarce resources, the emphasis thus being placed increasingly on integrated rural development.

Evaluation of feeder road projects, therefore, cannot be done in isolation; rather, all complementary activities must also be taken into account. Moreover, such projects cannot be justified in the standard user savings framework. Project evaluation must encompass the social, political, economic, and technical implications of the road itself and of its connection with other projects in the integrated development package.

1.2 Multi-Objective Analysis and Appraisal of Rural Road Projects

Single objective analysis techniques traditionally used in evaluation of road projects, like savings in user costs, producer and consumer surplus, and change in national income, are incapable of taking into consideration the spectrum of objectives relevant to the rural development effort. The emerging method of project evaluation, multi-objective analysis, incorporates both economic and non-economic objectives into an evaluation framework. Much has been written concerning multi-objective analysis in recent years by researchers in such diverse fields as management, engineering, and psychology. Friesz and Evans (2) propose four categories into which most of the prevailing methodologies can be classified.

Under Category 1, each attribute is valued in terms of some common reference attribute, standardly termed a numeraire. The many dimensions or attributes characterizing a project may thus be collapsed into one dimension. The project's value is proportional to the total amount of the numeraire it exhibits; that is, the project with the largest numeraire is optimal. The techniques of category 1 are the more traditional benefit-cost analysis methodologies, like UNIDO (3) and Little and Mirrlees (4), the unifying characteristic being the single numeraire, for which they are referred to as the aggregate method of multi-objective analysis. Through the use of social pricing more than a single objective can be implicitly considered, as, for example, the growth objective in the case of UNIDO (3) and the equity objective in that of Squire and van der Tak(5). Non-economic objectives may also be considered through the use of metric conversions, although these are typically difficult to determine.

Under Category 2, the full set of attributes is not expressible in terms of a single numeraire, but sufficient consensus exists among the users/decision makers that a utility function can be defined to express their level of satisfaction with each alternative. The project with the highest utility level is optimal. For the techniques of category 2, the element of subjectivity still exists, but the value judgments are explicitly articulated by the appropriate elected or appointed official, rather than implied by the metric conversions. Keeney and Raiffa(6), in fact, have developed specialized techniques for determining the appropriate mathematical form of the utility function depending on the relationships among the attributes.

Under Category 3, neither the single numeraire nor adequate consensus exists to define a single utility. This may occur for one of two reasons: (1) there are several statistically distinct groups of users in the user population, each with a distinct utility function; or (2) an individual user group may have multiple objectives or measures of utility against which the projects must be compared, but they may be uncertain as to their relative importance. A single optimal project cannot, therefore, generally be determined; rather, the original list of alternatives might be narrowed to a set of efficient or non-inferior projects, with the final choice being made outside any analytic framework.

Finally, under Category 4, an iterative analytic process is used to arrive at a best compromise under a situation of multiple objectives or utility functions. That is: (1) the most efficient consequence of selected assumptions concerning the relative importance of each objective are presented; (2) a new set of assumptions is derived through inputs from the parties involved; (3) the associated efficient consequences are displayed; and (4) further iterations take place until a final decision is reached.

Turning to the problem at hand, feeder road appraisal can be visualized as a choice of many small projects, where each project has several important selection criteria, each measure is expressed in its own units, and the set of projects to be implemented is selected from a much larger set. In the case of Kenya in the Fall of 1974, for example, a program to improve some 12,000 km of minor and secondary roads out of a possible 30,000 km was being planned, and a second program to upgrade some 16,000 km of low class rural roads to all-weather roads out of a theoretically possible total of 100,000 km of unclassified roads was being investigated.

The situations characterized by categories 3 and 4 are realistic representations of scenarios in developed countries where there are numerous parties involved, each with its own interests and capabilities and each desiring participation in the decision process. In the context of the rural development effort and for the purposes of this research, it may be assumed that there is a universal commitment to the achievement of certain accepted goals; that is, that a single set of social preferences can be articulated with the help of the appropriate decision maker, who might be, for example, the director of the road authority or Minister of Public Works. In view of this assumption and the characterization of the rural road situation given above, it appears that the rural road appraisal problem falls into category 2.

It is thus proposed to focus this paper on structuring a multi-criteria appraisal framework for rural road projects along the lines of techniques discussed under category 2. The development of this framework in Section 2 centers around four activities: (1) identification and measurement of potential criteria to be included in the evaluation; (2) assessment of the decision maker's preference function for each of these criteria, and ultimately scaling the measures of the criteria; (3) combination of the criterion measures for each project to form an explicit value structure as a basis for decision making; and (4) testing and implementation of the proposed methodology in some case studies. The appraisal methodology proposed is seen as a simple but valuable tool in the project selection stage.

2. A Proposed Framework for Socio-Economic Evaluation of Rural Road Projects

2.1 Identification and Measurement of the Criteria

Five criteria have been selected for the framework for appraisal of rural road projects in socio-economic terms. These include: (1) economic benefits, (2) economic costs, (3) distribution, (4) accessibility to social services, and (5) employment. Contributions to these criteria to be included are those resulting from provision of the feeder road and its complementary investments. These represent one possible set of criteria, and are not intended to be a universal representation of the accounting of socio-economic objectives of rural development activities. It is the decision maker in the particular case who must be satisfied with the set of criteria. The appraisal framework structured here is independent of changes in the criteria and in their number.

Economic criteria have traditionally been used in the approaisal of transport projects, and their measurement is widely known and well documented (e.g., 7,8,9). Economic benefits may be measured in terms, for example, of user cost savings, producer and consumer surplus, or increase in national income. Sole use of the first measure is cautioned against since its application assumes that most benefits stem from savings on normal traffic and development benefits are of negligible importance; in feeder road projects the opposite is generally true. The second measure, producer and consumer surplus, although conceptually attractive is difficult in practice due to its requirement of forecasts of approximate demand and supply functions. This leaves the national income measure, a good approximation being induced agricultural production sinced induced economic activity in rural areas, at least initially, will be largely in the agricultural sector. The difference in the present expected value of agricultural activity in the case of project implementation and in that of the no-project alternative is thus the suggested measure for economic benefits.

The value of this induced agricultural production depends on a variety of factors, including the nature of government extension help, receptiveness of farmers to new ideas, availability of cultivatable land and adequate climatic and resource requirements, and existence of a market for the goods in conjunction with sufficiently attractive farmgate prices. Various mathematical models, including linear programming, aggregate regression analysis, and disaggregate behavioral modeling approaches, have been formulated as potential means of forecasting economic benefits of roads (e.g., 10,11,12). Alternatively, induced economic activity as well as measures of other project criteria might be predicted on a project-by-project basis by a specially selected team of interdisciplinary experts, consisting of agricultural economists, sociologists, engineers, and anthropologists, among others. These various forecasting techniques merit further discussion and research, selection among them being a function of the availability of data, transferability of approaches, and biases of the decision-making authority. For the purposes of this paper, the parameters that might be tabulated and used by an appraisal team in assessing the value of each of the five criteria are indicated. Table 1 illustrates a possible set of parameters for determining the expected value of agricultural activity on a yearly basis for each crop and each group of farmers; combining these over the life of the project and comparing this with the no-project alternative yields a predicted value of induced agricultural production.

As observed above, economic benefits occur over time. For project appraisal purposes, it is convenient to present them at discrete intervals, such as yearly, and to aggregate these over the life of the project to a single value, such as present value, using an appropriate discount rate. Alternative means of incorporating temporal considerations are possible, and the debate over discount rates is an active one, but is beyond the scope of this paper.

Expenditures related to the construction and maintenance of the road and to any complementary investments as a part of the development package constitute economic costs. In addition to initial road construction costs, these might include costs associated with maintenance and upgrading over the life of the road; building, staffing, and operating complementary health and educational facilities; and changes in agricultural activities such as extension workers, seeds, tools, and fertilizers. For economic costs, like economic benefits, time must be taken into consideration.

Explicit accounting of the distribution of economic benefits among project beneficiaries has long been recognized as an important aspect of the appraisal of feeder road projects. Alleviation of poverty among the poorest has repeatedly been cited as a primary goal of rural development. Prediction of the small farmers' share of induced agricultural production is, however, hard to make. The use of a proxy measure is proposed, a promising candidate being the cultivatable land owned by the relevant income group(s).

This appears to be a reasonably measurable proxy and reliable representation of the distribution of benefits to the target population. Its usefulness is illustrated by the following two extreme cases: (1) the affected community consists of 500 persons, all of whom presently exist on income levels below that of the target income group; some 750 hectares of cultivatable land is to be opened up and planted, and ownership distributed evenly among the population; and (2) the affected community consists of 300 persons, of whom some 270 are peasants either farming at a subsistance level or working for the five rich families of the community who own/control almost all of the cultivatable land; although induced agricultural production is expected to be large, the subsistence group's share is expected to be negligible. At the same time, use of this measure necessarily entails certain restrictive assumptions such as: cultivatable land owned by the target group is generally less than ten hectares per family, economic conditions of perfect competition exist, average productivity of the land is uniform, and share of economic benefits is proportional to land ownership.

Project implementation may affect accessibility to social services through improved transport and thus easier or new access to existing services, and through complementary investments and thus provision of new service facilities. These may be mobile or permanent facilities. Decisions pertaining to such alternatives are presumably made in the design stage during the formulation of the set of projects to be evaluated; in the appraisal stage, then, the objective is to assess each project's contribution to this criterion.

Using accessibility to health services as an example, five levels of service might be considered: local mid-wives (H1), visiting trained nurses (H2), permanent trained nurses (H3), visiting health clinic (H4), and permanent health clinic (H5). Assuming no deterioration in services as a consequence of the project, Table 2 indicates the possible changes that might occur and their associated utilities on a scale of 0 to 10. Assuming the utility of change to be the same across the affected populace, accessibility to health services can be quantified by multiplying the appropriate figure in Table 2 by the number of people affected. Accessibility to educational facilities might be similarly measured, four possible levels being: none, general, vocational, and adult education. Assuming these to be primary indicators of social services and of equal importance, their sum may be used as a measure of accessibility to social services. This is admittedly a rather simplistic approach; consideration of the distance and timing associated with each change as well as other social services might be incorporated as well.

Table 1: Assessment of Expected Agricultural Production Crop: Year:

Type of Group of Farmers[1]	Cultivatable Areas[2] A	"Old" Average Yield[3] Y_0	"New" Average Yield[4] Y_1	Predicted Average Unit Market Price P_1	FROR to Farmer[5]	Demand Conditions	Other Relevant Parameters[6]	Probability of New Cultivation[7] Pr	Expected Value $[Pr \times P_1 \times Y_1 \times A + (1-Pr) \times P_1 \times Y_0 \times A]$

[1] Grouping might be based on land ownership, for example.
[2] Includes land currently uncultivated and idle, as well as that under cultivation.
[3] Before project implementation.
[4] A predicted figure, after project implementation.
[5] Financial rate of return.
[6] E.g., risk aversion characteristics of farmers, which might decrease over project life.
[7] A likely rather subjective assessment dependent, for example, on the three previous columns.

Table 2. Assessment of the Utility of Change in Health Services

Type of Change	Subjective Assessment of Medical Experts
No Change	0
H1 to H2	2
H1 to H3	5
H1 to H4	7
H1 to H5	10
H2 to H3	3
H2 to H4	5
H2 to H5	8
H3 to H4	2
H3 to H5	5
H4 to H5	3

Consideration of employment in project appraisal raises the question of whether it should be treated as an end or as a means to meeting other ends or objectives. Kessing (13) argues that employment must be treated as a separate objective as generation of employment does not emerge naturally from the pursuit of traditional macroeconomic objectives, while UNIDO (3) argues that it is a means associated with the redistribution objective. Additional arguments to consider employment as a separate measure include its being an indicator of the mobilization of labor, an important condition for rural development, and a measure of relative labor intensity among projects.

Man-days of employment associated with project implementation throughout its life are suggested as the measure. Included in this is employment resulting directly from construction and maintenance activities as well as that expected from increased agricultural activity. Employment of extension workers and other government employees should not be included as it is assumed they would otherwise be employed elsewhere; moreover, interest in employment is primarily from the viewpoint of mobilization of the local labor. Although employment occurs over time, as do economic benefits and costs, its value is assumed constant, and no discounting is proposed. Possible refinements in this measure might include distinction between short and long-term employment and checks on expected labor availability over time relative to its planned use.

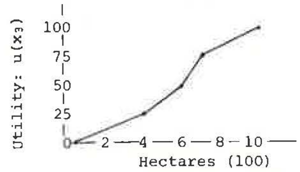
2.2 Scaling of the Criteria

Each criterion is measured in its own units: (1) economic benefits and costs in monetary terms, (2) distribution in hectares, (3) accessibility to social services in ATSS units, and (4) employment in man-days. At this point, the decision maker needs to be brought in to assist in transforming the spectrum of physical measures for each criterion into utility or psychological value terms.

Various utility assessment techniques might be applicable. In the first, the category technique, a number of discrete categories are specified for a particular criterion, and the decision maker assigns each project to one of these based on its contribution to that criterion; numerical worths can then be determined for each category, but the result is rather approximate. The second technique, the gamble, consists of lotteries constructed by varying the level of the measure or the probabilities of occurrence until the decision maker is indifferent between the lottery and a certainty equivalent; this tends to be a somewhat complicated and confusing, as well as time-consuming, technique. A third approach, the direct technique, is the most straightforward, requiring the decision maker to assign numerical values to the various levels of attainment of a particular measure. This can be done in two ways: (1) anchor one extreme point of the measure, and compare all of its other values to this; or (2) anchor the two extreme values of the measure, specify a few intermediate points, and use linear interpolation to complete the preference function.

The direct technique is generally the more attractive, and is used in the hypothetical testing of the appraisal framework in Section 2.4. A sample of its use in constructing the preference function for the third criterion, distribution, is given in Figure 1. In actual practice, the final selection of the utility assessment technique depends on the preferences of the decision maker and the topic of the assessment. Additional techniques are also available should one of these not seem appropriate.

Figure 1. Distribution Preference (Utility) Function



The measure of distribution arising from all projects under consideration ranges from 20 to 1,000 hectares. 600 hectares has been anticipated as the "50" point, 400 as the "25" point, and 720 as the "75" point. The distribution preference function is therefore:

$$u(x_3) = \begin{cases} 0.0658\ x_3 - 1.316 & 20 \leq x_3 \leq 400 \\ 0.125\ x_3 - 25 & 400 \leq x_3 \leq 600 \\ 0.208\ x_3 - 75 & 600 \leq x_3 \leq 720 \\ 0.0893\ x_3 - 10.7 & 720 \leq x_3 \leq 1{,}000 \end{cases}$$

2.3 Ranking of the Projects

Having identified the criteria of interest, delineated measures by which contributions to them might be assessed, and presented techniques for deriving the preference or utility function for each (i.e., scaling the physical measures), the final step in the formulation of the appraisal framework is combining the measures of the five criteria for each project into a single value structure by which the projects might be ranked. Completion of the analysis depends upon the decision maker's articulation of his preferences among the various criteria. Three scenarios are proposed: articulation of equal, cardinal, or ordinal weights for the criteria.

Implicit in the equal weights alternative, actually a subset of the cardinal weights case, is the assumption that all criteria are of equal importance. Thus, the projects are ranked by the value of the sum of the utilities over all criteria:

$$WVUC_1 = \sum_{i=1}^{n} \frac{u(x_i)}{n} \tag{1}$$

$u(x_i)$ is the utility function of criteria i, with n being the number of criteria.

If the criteria are truly equally important according to the best-knowledge of the decision maker, the analysis can proceed directly, using the above formulation with no further input from the decision maker.

The cardinal weights approach allows for differences in the relative importance of the various criteria, and assumes that explicit weights can be assigned to each. Projects are, therefore, ranked according to the weighted sum of the utilities over all criteria:

$$WVUC_2 = \sum_{i=1}^{n} \frac{w_i u(x_i)}{\sum_i w_i} \tag{2}$$

w_i is the explicit weight on criteria i.

To complete the analysis using this formulation, articulation of the cardinal weights must be elicited from the decision maker. In practice this often proves to be difficult due to conceptual problems in explicitly assigning the correct social weights and to political sensitivity issues.

In cases where the decision maker cannot or is unwilling to specify cardinal weights, the ordinal weights approach might be used in ranking the projects. Application of this alternative requires the decision maker to designate an ordinal ranking of the criteria reflecting their relative importance. Given this and utilizing concepts of linear programming, the analysis can be completed. The formulation discussed below is that initially developed by Cannon and Kmietowicz (14) for application to decision-making under uncertainty; further details of its derivation for application here are given by Chew (12).

As a first step, given an ordered set of criteria, the set of utility functions of the various criteria, and a set of projects, an upper and lower bound on the weighted score of each project can be determined based on that order of criteria. This can be formalized as two linear programming problems:

$$\text{Maximize (Minimize) } WVUC_3 = \sum_{i=1}^{n} w_i u(x_i) \qquad (3)$$

$$\text{Subject to: } \sum_{i=1}^{n} w_i = 1 \qquad (4)$$

$$w_i - w_{i+1} \geq 0 \quad (i = 1,\ldots,n-1) \qquad (5)$$

this reflects the ordering of the criteria

$$w_i \geq 0 \quad (i = 1,\ldots,n) \qquad (6)$$

That is, any set of cardinal weights which obeys the specified ordering will have a weighted score somewhere between these upper (maximum) and lower (minimum) bounds for each project.

Through the application of a couple transformations, two parameters and, thus, two possible decision rules, termed maximax and maximin, emerge for ranking the projects. Under the maximax decision rule, the projects are ranked according to the maximum weighted value of the criteria, or in other words, the highest score they can attain given the ordering of the criteria:

$$WVUC_3^{max} = \text{MAX} \sum_{j=1}^{i} \frac{u(x_j)}{i} \quad (i = 1,\ldots,n) \qquad (7)$$

Under the maximin decision rule, the projects are ranked according to the minimum weighted value of the criteria, or in other words, the lowest score they can attain given the ordering of the criteria:

$$WVUC_3^{min} = \text{MIN} \sum_{j=1}^{i} \frac{u(x_j)}{i} \quad (i = 1,\ldots,n) \qquad (8)$$

The ranking of projects produced by the maximax decision rule is of a more optimistic/less conservative nature. That is, if a situation arises in which the contribution to the most preferred criterion is exceptionally good relative to that to any of the other criteria which might be exceptionally poor, the maximax rule cannot take the latter into account. If this inability to account for an exceptionally poor measure is not a critical issue, as long as there exist more preferred criteria with exceptionally good measures, then use of the maximax rule may be justified. The maximin decision rule is, on the other hand, more conservative/less optimistic in nature. That is, the occurrence of an exceptionally poor criterion measure is taken into account by this decision rule, but it, in turn, is less able to reflect the occurrence of an exceptionally good one. If the ability to account for an exceptionally poor criterion measure is critical, as illustrated by the analogy of "a chain is as strong as its weakest link," then use of the maximin decision rule may be justified. One further limitation of both decision rules is that the set of projects are not ranked according to a single set of weights since that which maximizes (minimizes) one project will, in general, not be the same as that maximizing (minimizing) another. Finally, if the information is available and believed correct, use of equal or cardinal weighting techniques for ranking projects may be most appropriate.

2.4 Application/Testing of the Methodology

A hypothetical case study involving 36 alternative projects was designed to demonstrate the application of the overall appraisal framework (see Chew (12) for details). A typical project might be as follows:

> A 20-kilometer feeder road is proposed to join a small agricultural community of 600 persons to a small provincial market town served by a good secondary road. At present an earth trail, not passable by motor vehicles, exists and is mainly used for walking or transport by pack animals to the nearby town where the peasants may go to sell agricultural surplus and purchase consumer goods. The community appears to have rather suitable conditions (e.g., physical, ecological, demographic) for agricultural development.
>
> As part of a regional development effort, a package of investment projects has been proposed by the design team, including: upgrading the trail to a gravel road; agricultural extension services directed toward improving existing production, increasing the land under cultivation, and introducing a new crop, cocoa; establishing a health clinic in the community; and providing general education.
>
> The community has 109 families, of which 5 are relatively rich and own 45-50 hectares of land each, 34 own 2-10 hectares per family, and 70 are landless (50 renting a total of 100 hectares from the rich for subsistence farming, and 20 working for the rich families). Present production consists of cassava, rice, and maize, with a bit of livestock, on 405 hectares of land; an additional 113 hectares of cultivatable land is idle. It is proposed to bring this land under cultivation with a cash crop, cocoa, as well as an additional 70 hectares of nearby government land, the latter to be allotted to the 70 landless families. The target population is 104 families who currently own 278 hectares of land which will be increased to 348 hectares by the project....

Based on such information, contributions to the criteria were identified and measured as discussed in Section 2.1. This represents just one of the myriad possible scenarios for feeder road projects. Corresponding measures for the other 35 projects could similarly be determined; in the case at hand, a spectrum of plausible measures was simply developed. Using the utility assessment techniques of Section 2.2, preference functions were developed for each criterion, and utility values assessed for each project's contributions. The values developed and used in the case study are given in Table 3. These were then combined for each project, and used in ranking the projects as outlined in Section 2.3. The three mechanisms for the decision maker's articulation of preferences concerning the various criteria were tested under one or more sets of assumptions: (1) the equal weights alternative; (2) the cardinal weights alternative with the same order of criteria but three sets of weights; and (3) the ordinal weights alternative with the criteria in the same order as and in different orders from the cardinal weights approach, under maximax and maximin decision rules. Table 4 summarizes the various ranking techniques tried; Table 5 shows the alternative rankings of the set of projects thus achieved.

The ranking of projects by means of the equal weights assumption ($WVUC_1$) depends on their relative performance with regard to each criterion and in total. Projects ranked at the top tend to be uniformly good (e.g., Project 16), or relatively good in two or more criteria and not too bad in those remaining (e.g., Projects 14,13); those near the bottom tend to be uniformly poor (e.g.,Project 3), or relatively poor in several criteria and maybe even quite good in one or two (e.g.,Projects 19,20). Such generalizations must be treated with caution, however, as the ranking of projects is highly dependent on the particular set of projects. Moreover, assigning 36 separate rankings may be somewhat deceptive in that certain projects may be rather close in the numerical values underlying their rankings (e.g., projects ranked 5 to 14 are within 7%) and may thus be relatively equally desirable, at least for a first glance. It is, therefore, recommended that the WVUC values be viewed in conjunction with the rankings (see Chew(12)). Nevertheless, the strength of this appraisal framework is as a mechanism for sorting and ordering a large number of projects, and thereby selection of a group of potentially appropriate projects for further and more detailed inspection. These general comments pertain to all three ranking techniques.

The ranking of a particular project, when cardinal weights are specified ($WVUC_2$) depends on both the relative weights on the individual criteria and the project's performance relative to that of others in the set. $WVUC_2$'s behavior is rather similar to that of $WVUC_1$, as the weights on the criteria are nearly uniform. $WVUC_2^1$ and $WVUC_2^3$ show rather different rankings, however, as both their sets of weights favor the first criterion, $WVUC_2^3$ more so than $WVUC_2^1$. In the case of $WVUC_2^1$, for example, projects with a reasonably high measure on the first one or two criteria and maybe not so high on the others tend to rank high (e.g.,Projects 5,4), while those with a reasonably low measure on the first criterion and still relatively high on the others tend to rank low(e.g.,Project 30). Distinct differences exist in the rankings obtained from $WVUC_2^1$ and $WVUC_2^3$, as exemplified by Projects 17 and 22, differences in the emphasis on the second criterion being important here.

The ranking of projects under the ordinal weights assumption ($WVUC_3$) depends on the decision rule used, the ordering of the criteria, and the relative performance of the projects in the set. The five top-ranked projects under $WVUC_3^{max}1$ demonstrate the less conservative nature of the maximax decision rule in that the contribution of the most preferred criterion, economic benefits, overshadows those of all other criteria; Project 4 is an extreme example. Once contributions to the less preferred criteria become larger than that to the most preferred criterion, however, these begin to exert some influence, as in the case of Project 14. The conservativeness of the maximin decision rule is evident in the lowering of Project 4's ranking under $WVUC_3^{min}1$, and in the low ranks of Projects 19 and 20. The observation that the ordinal rankings under $WVUC_2$, $WVUC_3^{max}1$, and $WVUC_3^{min}1$ are the same is valid, but one cannot then proceed to assume that the rankings will also be similar, as demonstrated by Table 5. Some similarities exist as in the top-ranked projects, but striking differences also exist as in the case of Project 22. There is, within the specification of cardinal weights ($WVUC_2$), an almost infinite number of specifications which parallel the ordinal ranking of $WVUC_3^1$ but result in different rankings of the projects. The second ($WVUC_3^2$) and third ($WVUC_3^3$) sets of figures in the ordinal weights case demonstrate the sensitivity of project rankings to the preferential ordering of the criteria.

In order to better understand the use and implications of the various decision rules, the behavior of three projects across these alternatives is traced. The movement of Project 4 is particularly interesting as a result of its extremes in attainment of the various criteria: the highest possible utility score for the economic benefits criterion and lowest for the distributional one, with moderate to low scores on the remaining criteria. Thus, when equal weights ($WVUC_1$) or nearly equal cardinal weights ($WVUC_2^2$) are specified, it ranks around number 20. When cardinal weights with relatively higher weight on economic benefits and lower on distributional effects ($WVUC_2^1$) are applied, Project 4 moves up to position 3, and then up to number 1 when the extremes in the weights are made greater yet ($WVUC_2^3$). The more/less conservative natures of the maximax and maximin decision rules are well depicted by Project 4's behavior. It ranks number 1 with $WVUC_3^{max}1$ which has economic benefits as the most preferred criterion, and number 7 with $WVUC_3^{min}1$. When top priority is given to distributional effects, Project 4 drops to ranks 29 and 36, respectively, under $WVUC_3^{max}2$ and $WVUC_3^{min}2$. In the case of $WVUC_3^{max}3$ and $WVUC_3^{min}3$, the respective ranking is 16 and25 since its performance with respect to the preferred criterion is relatively poor.

The performance of Project 22 with respect to three of the five criteria is good (economic costs and employment) to excellent (distribution), and with respect to the remaining two is relatively poor. It ranks first for $WVUC_3^2$ which places distributional effects at the top. Its score on economic benefits is rather poor, and thus it ranks low, around 20, under $WVUC_2^3$ which puts nearly all its emphasis on this criterion, and under $WVUC_3^{min}1$ for which this criterion is most preferred. Its generally favorable performance with regard to the other criteria bring its rank up to 7 for $WVUC_3^{max}1$, and up into the range of 2 to 11 for the other ranking schemes.

Project 3's performance is relatively poor with regard to all the criteria. Correspondingly, it ranks rather low for all decision rules, although it tends to rise a bit when the maximin decision rule($WVUC_3^{min}$) is used because of its uniformly poor performance without any extreme lows in its utility scores.

The ranking of the alternative projects in the hypothetical case study naturally varies with the decision rule used because different value judgements and amounts of information have been provided in each case. It is not possible to suggest definitively which decision rule is the "best and only one". Its selection is most appropriately made on a case-by-case basis taking into account, for example, the nature of the projects involved and their expected contributions to development, the socio-political environment within which the planning is being done, and the type of value judgements the decision maker can and is willing to make. Adequate understanding by the analyst and proper education of the decision maker concerning the properties and implications of the various decision rules are essential to successful implementation of the proposed framework for project appraisal.

3. Summary, Conclusions and Recommendations

3.1 Summary and Conclusions

The plight of the rural poor in developing countries is a problem needing immediate attention today. Rural development is a formidable and multi-faceted problem which developing countries and lending agencies are finally beginning to face. Transportation is recognized as a necessary but insufficient ingredient in this process. Evaluation of rural road

Table 3. Utility of the Criteria Measures, $u(x_i)$

Project Number	Economic Benefits	Economic Costs	Distribution	Employment	Accessibility to Social Services
1	34.09	61.82	21.58	83.04	42.79
2	30.22	17.66	54.79	100.00	52.03
3	15.69	37.91	20.13	13.17	33.28
4	100.00	31.26	0.00	45.23	20.21
5	90.43	72.67	0.99	24.48	10.21
6	73.88	58.90	1.25	28.90	14.99
7	41.35	55.67	27.63	88.02	31.26
8	34.62	80.23	40.13	73.81	27.04
9	20.77	81.81	56.46	46.26	15.61
10	11.15	82.26	75.00	16.42	32.14
11	30.22	77.86	12.04	43.94	24.53
12	39.43	59.87	11.58	58.79	19.14
13	41.35	38.40	82.14	89.39	71.45
14	36.17	31.67	95.80	90.49	88.25
15	29.44	44.46	76.07	58.27	52.56
16	77.14	48.64	54.38	52.41	40.36
17	79.93	2.76	29.00	33.07	27.01
18	19.81	94.99	12.17	75.49	11.88
19	3.30	95.85	5.46	0.00	5.13
20	0.00	100.00	11.18	1.61	0.00
21	44.45	84.08	0.33	31.85	14.50
22	30.39	69.78	100.00	45.97	27.03
23	63.84	0.00	52.50	68.12	45.21
24	48.30	13.61	41.00	57.81	100.00
25	21.16	50.00	70.83	9.68	17.80
26	25.00	67.04	27.50	46.50	22.63
27	53.93	38.55	50.00	67.18	38.75
28	29.81	29.55	82.14	60.93	51.25
29	6.73	67.06	87.68	31.85	27.00
30	13.36	50.00	54.58	37.90	43.75
31	60.31	47.73	40.38	56.25	45.00
32	37.52	38.40	37.75	81.24	51.29
33	32.98	8.14	11.97	46.37	57.50
34	27.21	72.74	24.47	51.87	6.38
35	30.77	34.93	15.26	58.12	45.38
36	35.83	28.26	9.14	46.40	41.18

Note: For all criteria except economic costs, the lowest attainment is assigned a utility of 0, and the highest 100; the situation is reversed for economic costs where the highest cost is assigned a utility of 0, and the lowest 100.

Table 4. Decision Rules Used in the Case Study

$WVUC_1$: equal weights on the critera
$WVUC_2$: cardinal weights on the critera

Rule:	w_1	w_2	w_3	w_4	w_5
	Weights, w_i:				
$WVUC_2^1$	0.50	0.20	0.15	0.10	0.05
$WVUC_2^2$	0.22	0.21	0.20	0.19	0.18
$WVUC_2^3$	0.90	0.04	0.03	0.02	0.01

$WVUC_3$: ordinal ranking of the criteria

Rule:	Weighted Value of Criteria:	Preferential Ordering of Criteria, x_i:
$WVUC_3^{max_1}$	maximum	$x_1 > x_2 > x_3 > x_4 > x_5$
$WVUC_3^{min_1}$	minimum	$x_1 > x_2 > x_3 > x_4 > x_5$
$WVUC_3^{max_2}$	maximum	$x_3 > x_2 > x_1 > x_4 > x_5$
$WVUC_3^{min_2}$	minimum	$x_3 > x_2 > x_1 > x_4 > x_5$
$WVUC_3^{max_3}$	maximum	$x_2 > x_1 > x_3 > x_4 > x_5$
$WVUC_3^{min_3}$	minimum	$x_2 > x_1 > x_3 > x_4 > x_5$

where: > means "is preferred to"

i =
1 - economic benefits
2 - economic costs
3 - distribution
4 - employment
5 - accessibility to social services

Table 5. A Summary Comparison of Project Rankings Using Different Decision Rules

	Equal Weights:	Cardinal Weights:			Ordinal Ranking:					
Rank	$WVUC_1$	$WVUC_2^1$	$WVUC_2^2$	$WVUC_2^3$	$WVUC_3^{max_1}$	$WVUC_3^{min_1}$	$WVUC_3^{max_2}$	$WVUC_3^{min_2}$	$WVUC_3^{max_3}$	$WVUC_3^{min_3}$
1	14	16	14	4	4	16	22	22	20	8
2	13	5	13	5	5	31	14	14	19	22
3	22	4	16	16	17	27	29	13	18	16
4	16	31	22	17	16	7	28	16	21	31
5	15	13	15	6	6	13	13	15	10	9
6	24	6	8	23	14	5	10	28	9	10
7	8	14	24	31	22	4	15	27	5	18
8	2	27	31	27	13	32	25	9	8	7
9	28	22	28	24	21	6	9	29	11	5
10	31	17	27	21	23	21	8	10	34	1
11	27	23	2	13	31	12	16	31	22	27
12	32	8	7	7	8	8	20	8	14	13
13	7	7	32	12	18	17	2	30	29	32
14	1	24	1	14	10	1	5	32	26	26
15	23	15	23	32	11	14	30	2	6	11
16	9	1	9	8	27	23	18	25	4	12
17	29	21	29	1	29	24	7	7	13	15
18	10	32	10	36	7	22	23	26	16	29
19	18	28	18	22	9	11	27	24	1	34
20	30	9	5	15	15	28	24	23	12	6
21	5	12	4	28	24	15	31	34	7	21
22	4	2	30	2	2	34	19	1	31	25
23	12	11	12	33	28	35	1	3	27	30
24	26	18	11	11	1	26	32	17	15	14
25	11	34	26	35	20	36	34	35	24	4
26	35	10	34	34	34	2	26	18	2	28
27	34	26	6	26	12	25	11	11	28	35
28	6	29	35	9	19	9	6	12	30	36
29	21	25	21	18	32	18	4	20	25	20
30	17	35	17	25	25	33	21	33	32	19
31	25	36	25	30	26	3	12	36	23	3
32	36	30	36	3	30	30	17	19	17	2
33	33	33	33	10	35	10	35	6	3	24
34	3	19	3	29	36	29	36	5	35	33
35	20	20	20	19	33	19	33	21	36	17
36	19	3	19	20	3	20	3	4	33	23

projects, therefore, must begin to incorporate complementary investments in the development package, and to look at these projects from a socio-economic perspective.

The multi-criteria appraisal framework proposed above has been formulated in response to such concerns. A potentially appropriate set of developmental objectives is suggested, as well as possible means by which a project's contribution to these objectives might be assessed. These include traditional economic benefits and costs as well as distributional effects, accessibility to social services, and employment generation. This list is by no means inclusive; findings of further research may suggest new criteria as well as new means of measuring those currently of concern. The appraisal framework itself is independent of possible changes in the criteria and of addition of new ones. As the measures for each criterion are delineated in their own physical units, the direct or a similar utility assessment technique is suggested for developing preference functions for each criterion, and ultimately scaling the contributions of the projects.

The final step in the formulation of the appraisal framework is combining the scaled measures of the various criteria for each project into a single value structure as a basis for decision making. This requires articulation of the decision maker's preferences among the various criteria, for which three hypotheses are postulated. It is imperative that the decision maker be informed as to the implications and demands of these scenarios. Equal and cardinal weight assignments, for example, require the most information, but they also give the most reliable responses provided the weights are correct; ordinal ranking, on the other hand, is generally easier for the decision maker, but the maximax decision rule tends to be aggressive in ranking projects, and the maximin to be conservative. No definitive statement as to the appropriate decision rule can be made, other than that the selection of the approach is specific to the situation and cases under review, and is constrained by the available information and value judgments.

As a general conclusion, it has been demonstrated that it is conceptually possible to structure and implement a multi-criteria appraisal framework to account for the various socio-economic objectives of the rural development effort in the evaluation of rural transport projects. It appears that such a framework will be highly applicable in the developing country context, particularly in light of the large volume of resources anticipated to be allocated to rural development in the near future. The real strength of this appraisal framework is expected to be in sorting and ordering a large number of projects, for selection of a smaller group of potentially appropriate projects to be subjected to more detailed inspection.

3.2 Recommendations

The framework has been formulated at a conceptual level, and although a case study has been carried out to illustrate its implementation, it remains to be tested under actual field conditions. Some of the measures that have been advocated, for example, have yet to be collected by any appraisal effort. Such field testing of the framework constitutes the first recommendation for further action. In this way, policy makers' and other users' acceptance of the ideas and methods proposed can also be assessed.

Secondly, the multi-criteria appraisal framework proposed in this paper is a "first step" effort, and some refinements are clearly needed. Although the criteria are reasonably straightforward, for example, more research is needed on their predictive measurement. Use of land ownership by the target income group as a proxy for tracing distributional effects entails some restrictive assumptions, which might be removed through the structuring of a new measure, for example.

Finally, this research effort has focused only on the appraisal problem. Two other problem areas -- the design and implementation stages -- need further study, and are suggested as a third area for future consideration. Thus, for example, the design problem needs to be investigated as a multi-objective problem; identification of projects is a particularly critical step in the process, as the projects ultimately selected for implementation can only be as good as the best of those available.

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DESIGN THICKNESS OF LOW-VOLUME ROADS

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A theoretical model is presented for determining the design thickness of flexible pavements for low-volume roads. The structure of the pavements under consideration is single-layered, with either a waterproofed surface such as a membrane, a single bituminous surface treatment, or a double bituminous surface treatment; in other words, pavements of low construction cost. The theoretical model is based on the theory of plasticity, whose application seems to be a compatible choice for cases like these that encounter high values of rut depths, i.e. those exceeding 5-7.5 cm (2-3 in.). The validity of the model is verified through the test results and performance records of single-layered pavements used in a study carried out in Bolivia. Outputs of the theoretical derivations are employed to construct applicable design charts for the thickness values of single-layered pavements having traffic volumes of up to 0.5 x 10^6 operations of a standard axle of 80.1 kN (18 kips). These design charts are compared with the methods of other research agencies, and the existing differences are discussed.

Introduction

In developing countries, low-volume roads are associated with low construction cost pavements, whose structure is single-layered, with either a water proofed surface such as a membrane, a single bituminous surface treatment, or a double bituminous surface treatment. The single layer consists of granular material. These pavements are considered failures when the rut depth exceeds 5-7.5 cm (2-3 in.). An example of such a failure is shown in Fig. 1.

The design thickness of single-layered pavements is of special interest because of the effect on their low cost, on one hand, and on their required performance, on the other hand. This paper investigates design thickness in the light of the test results and performance records obtained from a study which was carried out on single-layered pavements in Santa Cruz, Bolivia, for the Bolivian Public Roads Dept., and which was intended to develop design charts for low-cost pavements for this country.

Basic Assumptions

Design thickness of flexible pavements is usually based on the theory of elasticity. However, for low-cost structures with a permissible rut depth of up to 5-7.5 cm (2-3 in.), the theory of plasticity seems to be a more compatible choice, and is therefore the one chosen for this study. The assumptions behind the theoretical model are delineated in equations 1 - 5.

For a given wheel load and given wheel configurations, a unique relationship exists between the critical bearing capacity of the pavement structure q and the number of operations leading to failure N. This relationship may be expressed as:

$$q = \alpha_p \log \beta_p N \qquad (1)$$

where α_p and β_p are constants depending on the wheel load, the wheel configurations, and the tire pressure.

Equation 1 actually states that for different structures having the same critical bearing capacity, the number of operations leading to failure is also the same, provided that the load's parameters are the same. The critical bearing capacity by itself is, of course, a function of the thickness of the pavement and the properties of the construction material. Thus, calculations of q necessitate a knowledge of the strength parameters both of the subgrade and of the single granular layer.

According to Foster's observations, the insitu CBR values of a single granular layer vary with depth, having high values on its surface and low values at its bottom. Similar results were obtained from existing single-layered pavements in service in Bolivia. Table 1 presents these insitu CBR results.

It should be noted that Foster's finding and the Bolivian results are compatible with the variation of the ratio E_G/E_S with depth, E_G being the resilient modulus of the granular material and E_S the resilient modulus of the subgrade material. This variation of E_G/E_S with depth is taken into consideration when the theory of elasticity is used to determine the thickness of flexible pavements.

Figure 1. Rutting failure in a granular single-layered pavement.

Table 1. In-situ CBR results from Bolivian Study.

Thickness of granular layer (cm)	CBR of subgrade (%)	CBR in the middle of the granular layer(%)	CBR on the surface of the granular layer(%)
15.0 - 17.0	9%	25% - 40%	> 80
14.5 - 16.0	12%	28% - 44%	> 80

Note: 1 cm = 0.39 in.

Summing up, the variation in in-situ CBR with depth is given according to the following expression (see Fig. 2):

$$\log CBR_G = (1 - \frac{y}{H}) \log \frac{CBR_T}{CBR_S} + \log CBR_S \qquad (2)$$

where CBR_G denotes the in-situ CBR of the granular material as a function of the depth y; CBR_T denotes the CBR on the surface of the granular material, i.e. for $y = o$; CBR_S denotes the CBR of the subgrade material, and H denotes the thickness of the single granular layer, being the thickness of the pavement.

Equation 2 is valid only for $o < y < H$. When $y > H$, CBR_G is actually equal to CBR_S.

Finally, use is made of the finding of Wiseman and Zeitlen, expressed by the following equation:

$$S = \lambda \cdot CBR \qquad (3)$$

where S is the shear strength of the material and λ is the material constant.

In order to simplify calculations, the angle of internal friction is taken as zero; thus S is equal to the radius R_f of the Mohr circle at failure, (see Fig. 3). In other words:

$$R_f = \frac{\sigma_x - \sigma_y}{2\cos 2\psi} = \lambda\ CBR_G \qquad (4)$$

Or:

$$\sigma_x = p + R_f \cdot \cos 2\psi \qquad (5a)$$

$$\sigma_y = p - R_f \cdot \cos 2\psi \qquad (5b)$$

$$\tau_{xy} = R_f \cdot \sin 2\psi \qquad (5c)$$

where σ_x, σ_y, and τ_{xy} denote the components of tensor stress in the $x - y$ plane; p denotes the isotropic stress; and ψ denotes the angle between x and the principal stress directions (the positive direction of ψ is counter-clockwise from x to the principal stress direction).

Figure 2. Variation in the CBR values of the granular layer with depth.

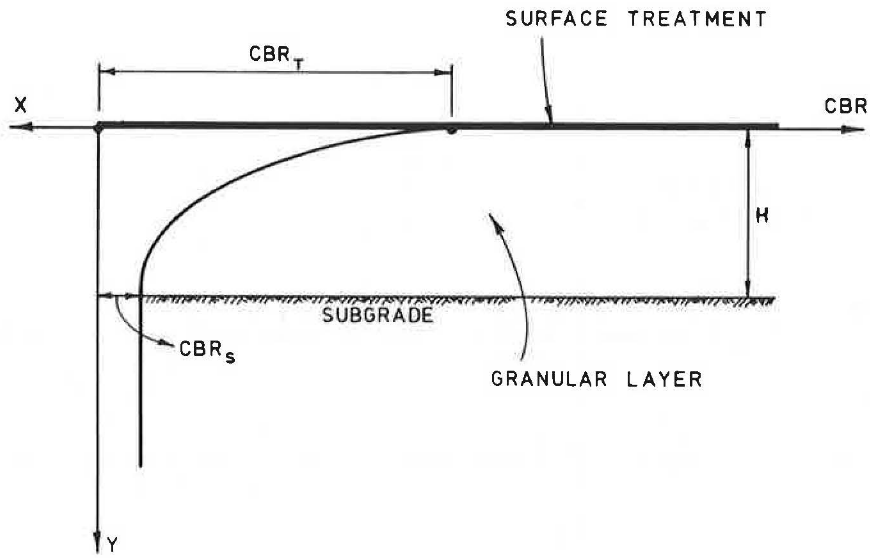
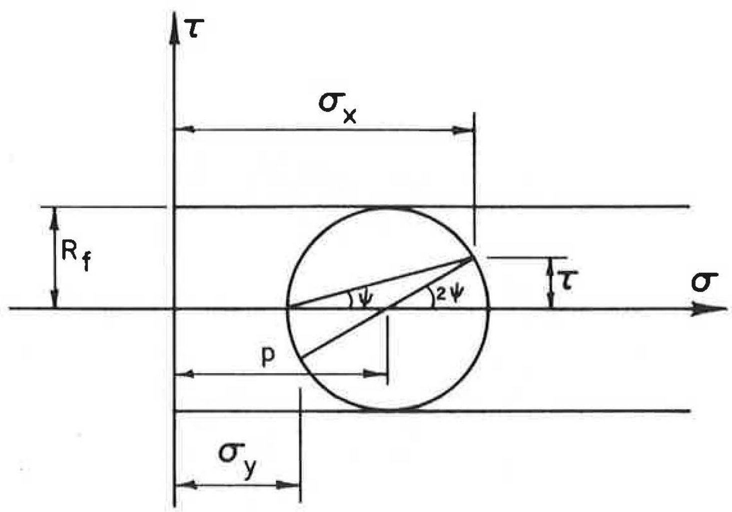


Figure 3. The Mohr circle during yield, at a given ψ.



Development of the Critical Bearing Capacity Equations

According to Livneh and Greenstein, it is possible to adopt the Prandtl failure mechanism - (see Fig. 4a), for calculating the bearing capacity of a single-layered pavement. This failure mechanism consists of (a) triangle OAD, a region of active Rankine shear, with angle OAD = $\pi/2$ ($\psi = \pi/2$); (b) triangle CBO, a region of passive Rankine shear, with angle CBO = $\pi/2$ ($\psi = o$); and (c) sector OAD, a region of radial shear, with a circular arc BA.

To find the critical bearing capacity q, moments are equated about 0. The stresses producing moments about 0 (see Fig.4b) are these:

σ_A - the active stress, which varies with the depth y, from Equation 2;

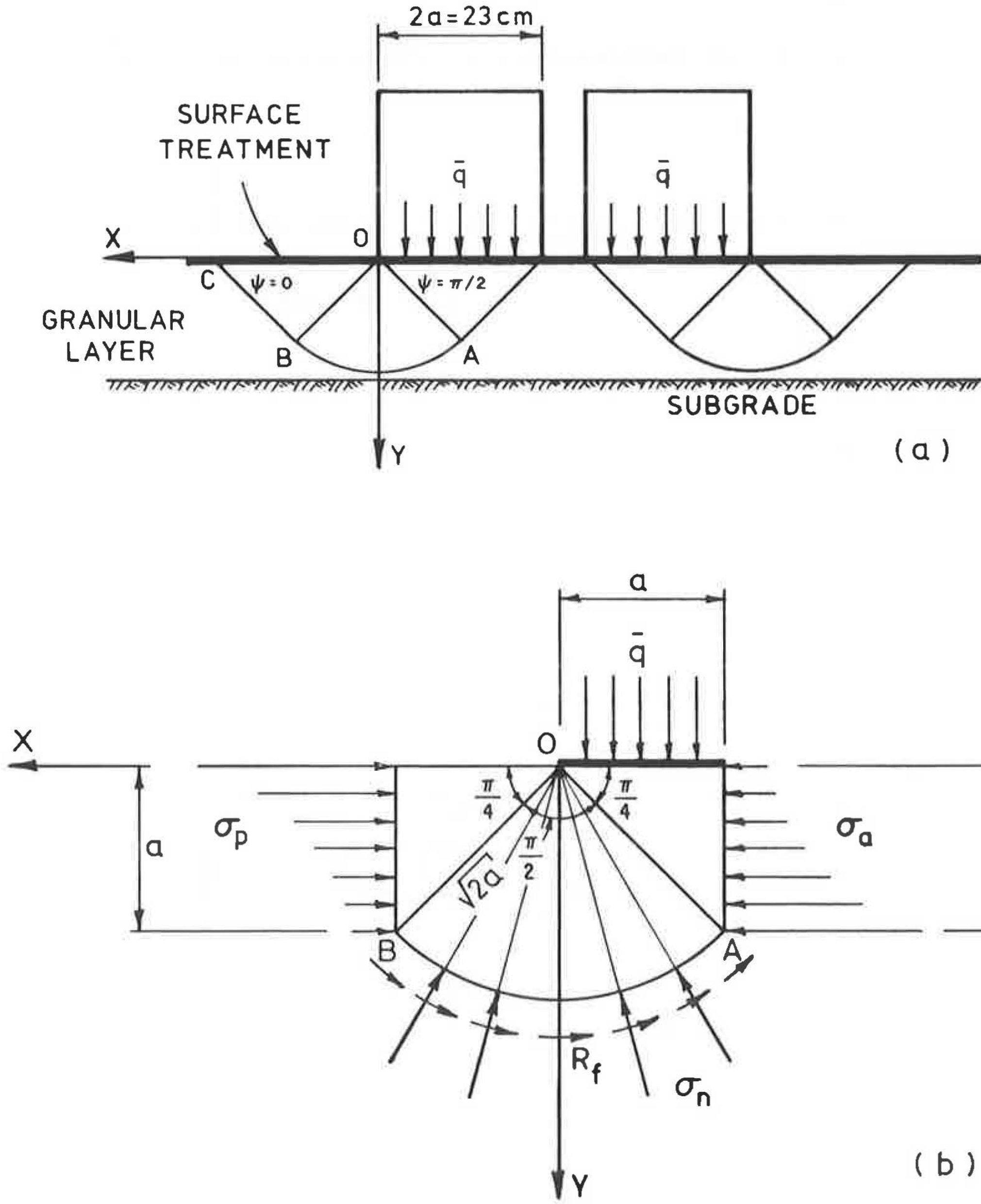
σ_p - the passive stress, which also varies with y;

R_f - the strength factor (i.e. the radius of the Mohr circle at failure), which varies with x and y (the normal stresses σ_n acting along the arc AB do not produce moments about 0). Thus, the average critical bearing capacity $\bar{q}$ acting along x (since q is a function of x, $\bar{q}$ is equal to $[\int_0^{2a} q\cdot dx]/2a$) is

$$\int_0^a \sigma_p y\cdot dy + \int_0^{\pi/2} \cdot R_f \,(a\sqrt{2})^2 \, d\psi = \frac{\bar{q}\, a^2}{2} + \int_0^a \sigma_A\cdot y\cdot dy \qquad (6)$$

where 2a is equal to the width of the contact area of the wheel being 23 cm (9 in.) for the standard axle.

Figure 4. Rupture mechanism for determining bearing capacity



According to Equation 5, σ_A is given by σ_x for $\psi = \pi/2$ and $\bar{q} = \sigma_y$; thus $p = \bar{q} - R_f$ and

$$\sigma_A = \bar{q} - 2R_f \tag{7}$$

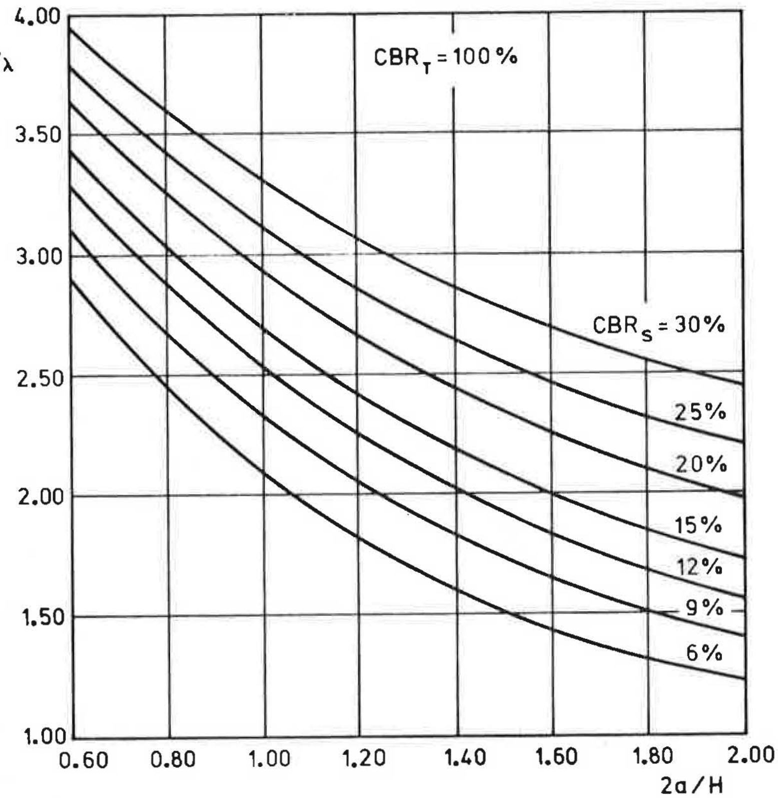
σ_A being a function of y, as mentioned above.

Similarly, σ_p is given by σ_x for $\psi = o$ and $\sigma_y = o$; thus $p = R_f$ and

$$\sigma_p = 2R_f \tag{8}$$

Proper substitutions and integration of Equation 6 with the aid of numerical methods yield the results given in Fig. 5. From Fig. 5, it can be seen that $\bar{q}/\lambda$ is dependent upon (a) the ratio $2a/H$, i.e. the ratio between the width of the contact area and the thickness of the pavement layer; (b) the CBR_S value of the subgrade; and (c) the CBR_T value of the granular material on the surface, which is equal to 100% in Fig. 5. It should be noted that $CBR_T = 100\%$ represents CBR values that are 40 - 60% of the entire granular material.

Figure 5. Critical bearing capacity as a function of 2a/H for different values of CBR_S



Determination of Design Charts

In order to determine design thickness charts, the α_p and β_p parameters should be calculated first. This is done with the aid of the results in Table 2, which were obtained from the Bolivian site study on failed pavements. These had the same values for the CBR_S value of the subgrade.

(N, the number of operations to failure, corresponds to the standard 18 kips axle load; 2a is equal to 23 cm (9 in.); $\bar{q}/\lambda$ is taken from Fig. 5 for the corresponding values of 2a/H; and CBR_S = 9%).

Table 2. Performance data of failed pavements in Santa Cruz, Bolivia, study, with CBR_S = 9%.

H (cm)	2a/H	N x 1000	rut depth (cm)	$\bar{q}/\lambda$
16.5	1.39	20	7.0	1.84
18.5	1.24	50	5.0-6.5	2.03
22.0	1.04	150	5.0-6.5	2.24
25.0	0.92	450	5.0-6.5	2.45

Note: 1 cm = 0.39 in.

The results of Table 2 are plotted in Fig. 6, with $\bar{q}/\lambda$ being a function of N. It is seen from this figure that a linear relationship exists between $\bar{q}/\lambda$ and log N, as postulated in Equation 1. This finding supports the theoretical derivations presented here.

Fig. 6 enables the conversion of the $\bar{q}/\lambda$ which appear in Fig. 5 into N values, thus yielding the design thickness charts presented in Fig. 7. The $\bar{q}/\lambda$ and 2a/H values are also given in this figure

Verification of Design Thickness Charts

The validity of the resulting design charts was verified with the aid of performance data from the Bolivia study of failed pavement, with different types of subgrades. These data are presented in Table 3.

Table 3. Performance data of Bolivian failed pavements with different subgrades

1000·N	H (cm)	CBR_S (%)	rut depth (cm)
20	16.5	9	7.0
50	22.0	6	5.0-8.0
50	18.5	9	5.0-6.5
50	16.5	12	5.5-6.5
100	26.5	6	5.0-6.0
150	22.0	9	5.0-6.5
150	20.0	12	4.5-5.5
200	26.5	6	5.5-7.0
200	20.0	12	5.0-6.0
200	16.5	15	4.0-5.0
350	20.0	15	5.0-6.0
350	21.5	15	5.0-6.5
450	25.0	9	5.0-6.0
450	19.0	15	3.0-4.0

Note: 1 cm = 0.39 in.

Figure 6. Relationship between the critical bearing capacity and the number of operations to failure of the standard axle load (18 kips)

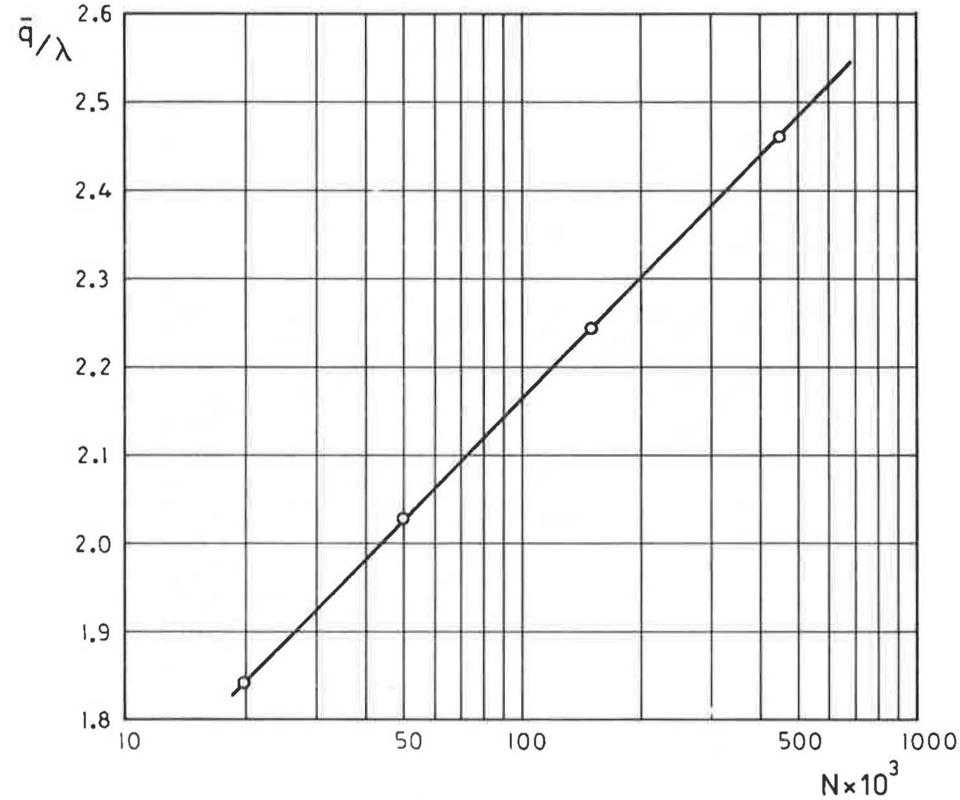
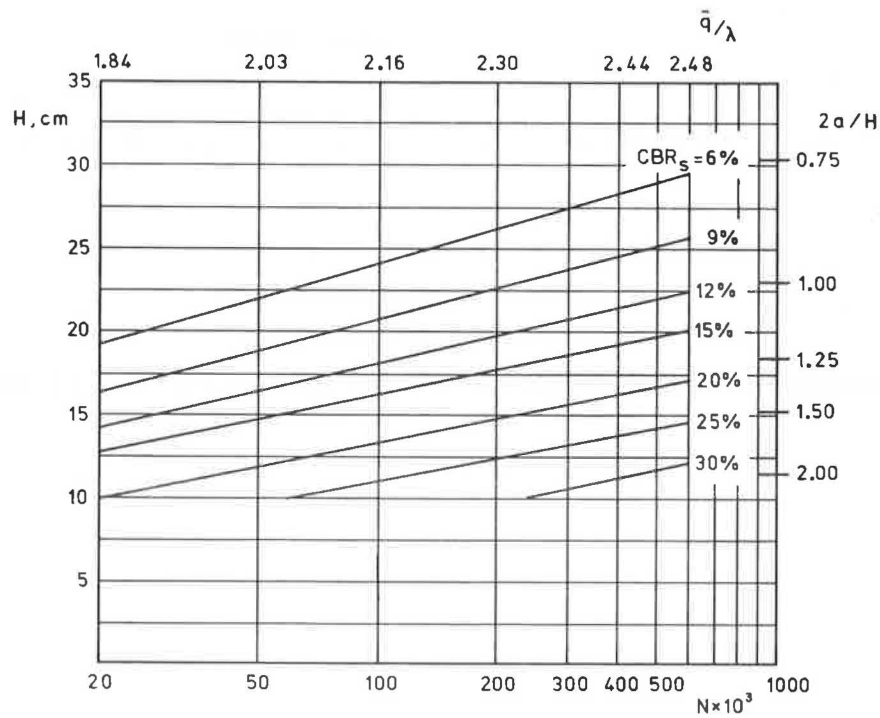


Figure 7. Thickness design charts for a granular single-layered pavement and for the standard axle load (Note: 1 cm = 0.39 in.)



Again in this table, N denotes the number of operations of the standard 80.1 kN (18 kips) axle load per lane leading to failure. The rut depth is measured with a beam 180 cm (5.9 ft) in length.

Fig. 8 shows the performance data together with the design charts obtained from Fig. 7. It can be seen that the suggested design charts are in reasonable agreement with the performance data.

It should be noted that using these performance data for a multiple linear regression analysis yields the following results:

$$H = 9.5 - 22.5 \log CBR_S + 6.6 \log N \qquad (9)$$

(H is given in cm)

The differences in H as obtained from Equation 9 and from Fig. 7 are negligible (less than 0.5%). Thus, the theoretical background presented in this paper strengthens the validity of the results. Yet, because of the limited performance data, the design charts should be used only when $0.9 < H/a < 2.2$.

Comparison With Other Sources

Brabston (6) constructed design charts for a single-layered structure having a single bituminous surface treatment. These charts may be formulized as follows:

$$H = 53.3 - 84.5 \log CBR_S + 8.1 \log 0.4\ N \qquad (10)$$

(H is given in cm and N ranges from 1.7×10^6 to 2×10^7).

It can be shown that using Equation 10 for low values of N may result in considerably small values of H when these are compared with Fig. 7. For example, when N equals 0.5×10^6 and CBR_S equals 9%, H equals 16 cm (6.3 in.); according to Fig. 7, H equals 25 cm (9.8 in.). Moreover, using higher values of CBR_S and lower values of N, Equation 10 may result in negative values of H. Thus, Equation 10 is not applicable for the range of operations under discussion.

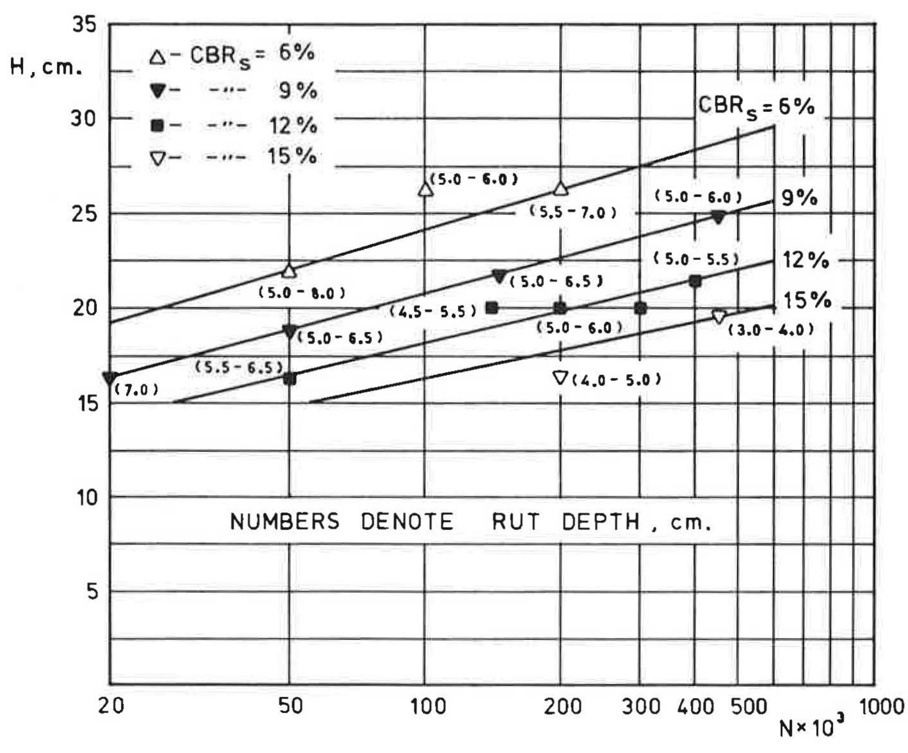
Hamitt (7), states that the thickness of unsurfaced pavements is 75% of the thickness of conconventional structures as obtained by the U.S. Corps of Engineers (8). A comparison of the thicknesses derived from Fig. 7 with those from the U.S. Corps of Engineers' procedure for conventional structures results in the percentages presented in Table 4.

Table 4. Thickness percentages for three CBR_S values

CBR_S (%)	N x 1000			
	20	50	200	400
6	59	64	65	67
9	72	74	78	78
12	74	81	87	87

From Table 4 it can be seen that the thickness percentage is not constant; its higher values associated with higher values for CBR_S and with higher values for N. In any case, the values are comparable with the value of 75%; thus, the procedure proposed in this paper may be considered a refinement of Hamitt (7).

Figure 8. Performance data together with the derived design charts(Note: 1 cm= 0.39 in)



Conclusions

There are two main conclusions to be drawn from the work described in this paper:

1. The theoretical derivations presented in this paper enable a determination of design thickness charts for granular single-layered pavements.
2. The proposed method is a refinement of the procedure proposed by Hamitt (7).

It is suggested that the design charts given in Fig. 7 should not be extrapolated for N or for CBR_S. Also, the minimum thickness of the granular material should not be less than 10 cm (3.9 in.).

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SOME ASPECTS OF PAVEMENT DESIGN AND PERFORMANCE FOR LOW VOLUME ROADS IN NEW ZEALAND

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In defining the role of a pavement, this paper discusses the essential features, methods of design and performance with respect to flexible unsealed and thin surfaced New Zealand roads. The materials which are normally used in these pavements are identified and assessments made on material characteristics for design. By defining the two main modes of pavement deformation, namely shallow shear and deep seated (subgrade) deformation, it has been shown that stabilisation methods can be used to reduce the rate of deformation in both chip sealed and unsealed pavements.
In order to limit the rate of deformation, a Modified Design Approach, which essentially incorporates both a modified subgrade and base layer into the structural pavement, has been recommended and then confirmed as far as practicable using field performance data.
It is concluded that low volume roads can be constructed successfully using thin total pavement depth provided think time and stabilisation techniques are employed with the aid of appropriate design skills.

Roading in NZ

New Zealand is a small country (266 917 sq km in area), has a low population density (3 million), and has a roading network of 92 600 km. As stated by Langbein et. al. (1) New Zealand's economy is based on the export of primary produce, and therefore the transport system must cover the country with a reasonably close network, but with only a moderate density of traffic away from the three main centres of population.

The Roading Network

At present the national network of state highways has of 10 785 km sealed and 770 km unsealed while the local county and municipality roads and streets consist of 35 615 km sealed and 45 432 km unsealed.

The present network is divided into two classes, Class I limits a twin tyred spaced axle to 8.2 tonnes while for Class II the limit is 7.3 tonnes. Traffic flows on many roads are light (less than 100 vehicles per day) and the vehicle type varies considerably from area to area depending on land utilisation and port outlets.

Estimating design loadings is sometimes difficult as a change in use will turn a satisfactory road into one which rapidly deforms under the new traffic conditions.

Climate

Generally the climate in New Zealand can be regarded as temperate, with air temperatures reaching 30°C plus in the summer down to below zero in the winter. Rainfall varies from 300 mm in the Central South Island region to 7000 mm on the southern parts of the West Coast. Frost penetration of pavement layers is confined to central regions of both North and South Islands with a maximum recorded penetration of 400 mm in areas serviced by roads.

Soil Types

Naturally occurring soils vary widely throughout the country and in fact can change over a 10 m length of road. In Canterbury thick layers of gravel form the subgrade while in the Central North Island, volcanic ashes and pumice are the predominant soils. Further north large areas of swamp and very weak clayey soils predominate.

Geologically the South Island is much more mature than the North and consequently soil types are more stable in the South Island.

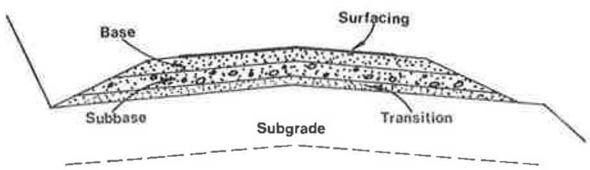
General Roading Type

Roads in New Zealand vary from just tracks suitable for four wheel drive vehicles, to modern motorways.

The unsealed road has generally been developed from cart tracks by grading and the application of maintenance aggregate. Of the sealed roads, 95% are surfaced with chip seals. The typical construction being subgrade, transition layer, subbase, base and surfacing (Figure 1). Many of these pavements so constructed were designed, but in the forties and fifties a "seal as is" policy was implemented in New Zealand to increase the total percentage of sealed roads.

A few concrete roads were built in the late twenties while in the late fifties to early sixties a number of pavement layers were cement stabilised.

Figure 1. A typical New Zealand pavement cross-section.



Surfacing - Thin bituminous chip seal.
Base - Quality unbound granular basecourse.
Subbase - Unbound granular material.
Transition - Graded layer to prevent fines from the subgrade migrating into the overlying layers.
Subgrade - That portion of the road formation, from formation level down one metre.
Basecourse - Unbound aggregate used as a base layer.

The Requirements of a Pavement

For the purposes of this paper, only thin surfaced or unsealed pavements incorporating unbound or bound layers will be considered.

For a pavement to be considered adequately designed it must be capable of retaining an acceptable (to the motorist) surface shape before the design life expires. In New Zealand basically three modes of pavement distress cause loss of shape in a thin surfaced flexible pavement, namely:

1. Failure of the surface allowing water infiltration (eg, by cracking);
2. Shallow shear within the pavement layers;
3. Deep seated (subgrade) deformation.

Assuming an adequate reseal cycle, pavements should be designed to resist the last two deformations for the vehicle loadings appropriate to the road.

The Purposes of Pavement Design

Design is not only an attempt to predict the performance of a system before it is constructed, but includes the best utilisation of available resources to yield an adequate structure. Even though pavements are different from most civil engineering structures in that they deform until the surface ride is no longer acceptable, some attempt at design is essential if premature surface deformation or shape change is to be avoided. In the past, pavement design has concentrated on the determination of thickness, based either on observation of local area performance, or design methods utilising limited laboratory testing of subgrade soils.

Pavement design is best approached by the selection of materials and structure thickness which will resist the modes of deformation. Whether new pavement is being constructed or an existing one upgraded, the design must recognise that deformation will occur. It is the control of the rate at which this occurs that is of prime concern.

Present Pavement Design and Material Use

Because of the limited funds available in New Zealand per kilometre of road, the practice has been to use local materials and minimal structural depths.

Unsealed Roads

Unsealed roads in New Zealand are seldom formally designed, but are maintained regularly by grader and maintenance aggregate added twice yearly at an average annual rate of 0.50 m^3 per kilometre per vehicle per day. Regular grading is essential to limit wheel path rutting, potholing and corrugation formation.

The quality of the running aggregate varies throughout the country such that some road controlling authorities are forced to use material which rapidly breaks down under traffic and the fines so formed are blown away. In other areas good aggregate is used, but it is pushed down into the subgrade over the winter period and lost.

Sealed Roads

Most existing thin surfaced roads have been designed on the basis of subgrade CBR values. Guidelines on pavement design are at present provided by the National Roads Board (2), while specifications provide for the quality control of materials to be used in a pavement. These specifications have been developed to allow the use of local materials especially in subbase layers.

Most pavements built in the last ten years have been designed as flexible using unbound materials. Acceptance of all but the basecourse has been predominantly on the soaked CBR value. The basecourse is accepted on grading control, sand equivalent, 10% fines limit and weathering resistance. In the last two years stabilisation of many local materials has changed the mode of pavement performance. The use of the CBR test as the acceptable criterion is now common for all cement or lime modified materials. This test may not appear the most appropriate but it at least provides some measure of shear resistance of the modified material.

General Pavement Performance

Most thin surfaced pavements have performed adequately to the end of their design life. Many road controlling authorities maintain a reasonable reseal cycle so that surfacing integrity is retained.

From trenching of deformed pavements it has been confirmed that the two main modes of distress are shallow shear in the basecourse and deep seated deformation with the predominant mode being found to be the former. Shallow shear appeared to be associated with low shear strength. This is confirmed by one hundred basecourses sampled from out-of-shape pavements in a local central North Island region where the soaked CBR's ranged from 3 to 30, thus clearly indicating the reason for loss of pavement shape. All the basecourses tested contained appreciable quantities of clay, which was probably produced by hydrothermal and hydrochemical degradation.

The Use of Stabilised Layers

For approximately 20 years, stabilised materials have been used in New Zealand road pavements (3). In the late 1950's and early 1960's considerable lengths of pavement were constructed using cement stabilised bases and subbases. Stabilisation declined in the late 1960's and early 1970's largely because of the lack of a clear concept of mode of action leading to concern at the development of reflective cracking. This decline continued until two years ago when renewed interest was shown especially in lime stabilisation. At the present time stabilisation is playing a developing role in New Zealand's roading construction and rehabilitation programme, and its use is expected to increase.

A stabilised layer can be located at any level in the pavement, or indeed the whole pavement above the natural subgrade may be treated.

Subgrade Treatment

1. By modifying the subgrade soil with small quantities of lime or cement, the sensitivity to ingress of water is reduced.

2. The addition of a stabiliser to the subgrade will increase the modulus (stiffness) of this top layer so formed which, although thin, will greatly reduce the rate of deformation in the untreated subgrade.

Subbase and Base Treatment

1. Both lime and cement improve the load spreading ability of an otherwise unbound material by increasing its modulus and tensile strength. With high lime contents and moderate cement contents, slab action can be developed, resulting in greater load spreadability of the pavement layer, provided intensive internal cracking can be avoided.

2. The addition of a stabiliser will also increase the shear strength of a material sufficiently to enable it to be used at a higher level in the pavement than that for the untreated material.

3. It would appear that lime and probably cement inhibit hydrochemical and hydrothermal degradation of aggregates. This means that presently unusable aggregates, after treatment, can be used in subbases and bases.

In designing any pavement with one or more stabilised layers it is essential that the original purpose of using a stabiliser is understood. It is not necessarily a process which will automatically bring about a pavement thickness reduction when compared with conventional unbound pavements, but is a system by which locally available material of otherwise unacceptable quality can be used in place of a higher grade and consequently more expensive unbound aggregate.

Likewise, it is not economic to over-stabilise a material. For this reason, Dunlop, (4) defined two distinct stabilisation phases namely; modified-soil and cemented-soil.

A modified-soil is one in which small amounts of stabiliser (.5-3.0 percent by mass) are used to correct a grading deficiency, reduce or eliminate plasticity, or provide a weak construction platform. In this definition, for a soil to be termed modified, the tensile strength of a lime treated soil after 14 days curing at 20°C must be less than 80 kPa. The same criterion applies to a cement modified-soil but the curing time should be seven days.

A cemented-soil has a tensile strength of greater than 80 kPa using the same curing conditions specified for the modified-soil.

Identifying Reactive Soils

Before attempting any stabilisation work it is essential to check that the soil is reactive to the proposed stabiliser. Most soils can be stabilised with either lime or cement, but soils containing organic matter do not usually react.

In New Zealand laboratory testing is undertaken before any soil is stabilised. If reactivity only is required a quick response test is specified (5), while for more accurate results detailed testing is required. For modified soils the CBR values are obtained on the untreated and then treated soil after a specified number of days curing and soaking. Increase in bearing of at least 100% indicates reactivity. If cemented soils are required then tensile testing with respect to time is carried out to check strength development.

Selection of Materials for Light Duty Pavements

Wide ranges of material types and design methods are available for thin surfaced flexible pavements. With limited cross-section and hence limited volume of materials per unit length, importation of materials from a significant distance incurs not only a transportation cost penalty, but the cartage of roading materials will often cause a substantial proportion of total pavement wear on the low volume road network. Therefore for overall economy a road controlling authority should aim to use local materials, either untreated or treated.

Untreated, modified and cemented materials each have their advantages and disadvantages. Examination of the disadvantages of each, in the local situation, is a suitable way of deciding on local policy. The principal disadvantages to be considered for each material are:

Totally unbound materials.

1. All subgrade variability must be dealt with by the superimposed pavement layers.

2. Nearly all unbound materials have substantial moisture sensitivity.

3. Easily won local materials can suffer substantial breakdown by hydrothermal and hydrochemical alteration (6).

4. Most naturally occurring materials when being won from a source (eg quarry) contain pockets of contaminants or low grade materials.

Modified-soil.

1. Require specific mixing/blending equipment and techniques needing change in job skills and management.

2. Design method needs modification to effect maximum available economies.

Cemented-soil.

1. All cemented-soil layers are affected by shrinkage and thermal strain changes.

2. The layers are subject to fatigue failure and are vulnerable to over-stressing caused by overloads (7).

3. Crack initiating defects are events which will usually have to be designed for, (8) and will cause up to 40% increase in the tensile strain at the bottom of a cemented layer. When such a defect progresses to full crack development, the vertical strain in the top of the subgrade is increased by a factor of up to 14 in the area immediately adjacent. If water filters down the

crack into the subgrade a concentrated soft spot develops with consequent loss of bearing.

4. Construction variability on the bottom of a cemented-soil layer (the critical zone for tensile based performance) must be carefully appraised and controlled. Both mixing and compaction are difficult at the bottom of an in situ mixed layer; Otte (9) indicates that a reduction of 30% in assigned strength should be used for design purposes.

5. Cemented soils increase in stiffness with time, accentuating the load attracting characteristics and hence likelihood of crack formation unless strength improvement matches the stiffness change.

6. Design method requires substantial sophistication, and sufficient testing and construction control to assure expected properties, if full economy of the process is to be realised.

7. Construction technique adaption is required as for modified-soils.

It can be concluded that both unbound and cemented-soil layers have problems which are not always easy to overcome. Take for instance the development of the up-side down pavement in which a cemented-soil layer is used as a subbase. It appears to be one more of accident than real design in that it was found that cemented bases produced large reflective cracks on the road surface. Therefore to overcome both thermal and shrinkage effects and the danger of reflective cracking the cemented layer has been buried under a granular base.

The writer would question the thousands of dollars which are now being, and have been spent on trying to accommodate an introduced problem layer. Hence the recommended use of a modified-soil with its superior shear strength, modulus and durability when compared to untreated unbound aggregate yet it is unlikely to cause reflective cracking even when used in the base.

Recommended Design for Flexible Pavements

Pavement design is developed in this section as a concept approach utilising locally available materials and its consistency with inservice performance is discussed in the following section.

Design for Unsurfaced Roads

An unsealed road must withstand environmental changes, traffic abrasion, and vehicle loadings. The motorist expects the road to remain free of potholes and corrugations, to be dust free and never become impassable.

The presently accepted New Zealand technique of applying maintenance aggregate once or twice yearly and regularly grading falls well short of the motorist's requirements of an acceptable ride, little dust and in some cases a passable road. Some roading authorities have now decided that an initial outlay of three times the annual maintenance bill per kilometre can stabilise the existing unsealed pavement to a depth of 200 mm with either lime or cement and produce overall savings within four years. This treatment also provides a much better service to the motorist.

For this type of treatment, design requirements are minimal, but the following steps need to be considered:

1. If the road is likely to be subjected to freeze-thaw cycles then stabilised layers should be used with caution. This problem can be overcome by overlaying the stabilised material with a depth of non-frost susceptible either unbound, or lime modified aggregate to a depth equivalent to the maximum frost penetration expected.

2. Check if the soil is reactive to lime; if not use cement.

3. Decide on stabiliser content on the basis that the top surface should not act as a slab. Normally 2-4% stabiliser is required.

4. Plan to construct late spring early summer to allow the stabilised layer time to develop strength before the winter.

5. Ensure that adequate water tables are formed before commencing construction.

6. Shape and compact the surface to a tight finish.

7. After one day roll into the surface a thin layer of clean large-sized running aggregate.

This procedure is applicable for roads carrying less than 500 vehicles per day.

In the first summer deformation in the wheel paths will occur and the dust problem will be similar to that experienced on an unbound unsealed road. Over the first winter lime stabilised material in particular can be re-shaped once the pavement becomes saturated. From this time on dust problems will be minimal and potholes and corrugations are very unlikely to develop.

Design for a Thin Surfaced Road

This section outlines the minimum input data required and then describes with the aid of Figure 3 the use of modified materials (defined by tensile strength not exceeding 80 kPa), in a Modified Design Approach for thin surfaced pavements.

Traffic Analysis. In determining the pavement loading prior to determining the required thickness it is desirable to obtain information on the present traffic volumes, the loading of the vehicles using the road, the expected growth rate and the desired design life. If a completely new road is being constructed then forecasts of traffic volumes need to be made.

Because of convenience and lack of evidence to the contrary, the AASHO road test data for axle load equivalencies should be used, in which all loads are related to an equivalent design axle (EDA) of 8200 kg.

Stage Construction. In obtaining pavement loadings it is desirable to consider the economics of stage construction. Normally a new flexible pavement would be designed for 15-20 years but a shorter design life should be evaluated. Many roading authorities are now considering stage construction in which the design life is reduced for the initial construction stage, and an overlay is programmed to be constructed after approximately ten years depending on traffic loadings.

The design life for a stage should take into account pavement deformation rate, surfacing life and residual life so that the next stage will become necessary only when full economic benefit has been obtained from the initial investment.

Environment. Variations in ambient temperatures do not appreciably change material properties of unbound material except in the upper 100 mm of the base. In this top layer temperature gradients can become quite high thus moving moisture through the unbound layer and changing its shear strength. Another factor which needs to be considered is the possible accumulation of water on the underside of the seal. It would appear from New Zealand experience this only becomes a problem when the basecourse layer is topped with fines whose high surface area available for water retention promotes seal lifting.

Increasing water contents has a detrimental effect on the subgrade whereas freezing and thawing can affect any pavement layer constructed of frost susceptible soils. When designing a pavement the designer must allow for adequate subsurface and surface drainage.

Material Characterisation

Subgrade Soil. Many investigators have demonstrated on full scale test pavements that linear elastic theory can be used to describe the subgrade response provided the modulus of the soil is determined under appropriate conditions. It is preferable that modulus values be obtained from either in situ or repeated loading laboratory tests. If this information is not available then the empirical relationship between modulus and CBR can be used.

$E = 10 \times CBR$ MPa

The other factor which needs defining is the permanent deformation in the subgrade. Claessen et al (10) have suggested the use of the following relationship:

$e = 2.8 \times 10^2 \times N^{-0.25}$

e = permissable compressive strain in subgrade

N = number of strain repetitions.

It must be remembered that strains developed in cohesive soils under repeated loadings depend on soil history, saturation and density. Therefore if a pavement is being placed on a dense over-consolidated soil subgrade then the permissible strain is increased.

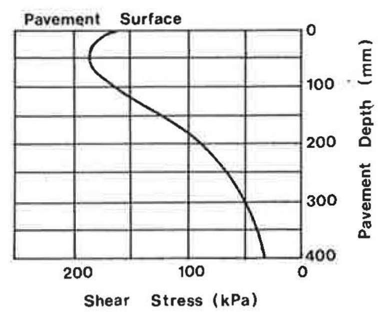
Normally a subgrade soil is subjected to relatively low stresses due to wheel loads, compared with overlying materials, so that shear failure is not possible. In thin pavements with cover thickness of less than 200 mm it is recommended that shear strength of the soil be checked.

Untreated and Modified Granular Layers. From research recently carried out in New Zealand (11) unbound and modified aggregates, after initial traffic compaction, tend to develop interparticle interlock which remains intact indefinitely provided adequate durability and shear strength of the material are retained. Therefore these two factors, durability and shear strength characterise an unbound or modified material.

As degradation occurs by physical, hydrochemical and hydrothermal means, no present known test adequately ranks aggregates for long term durability. In an attempt to overcome this present lack of an adequate durability test it is proposed that any unbound or modified-soil be subjected to a 12 cycle wetting and drying test (5) followed by petrographical analysis of the clay size particles. Ranking of these fines can be aided by the "clay index" system developed by Sameshima et al (6). It is suggested that a clay index of > 4.5 should be regarded as satisfactory. Inservice performance indicates that aggregates modified with cement and lime appear to develop an immunity to degradation.

In determining the suitability of the soil which has been subject to the wetting and drying test a simple shear test will provide adequate results. The tested soil shear strength should be at least twice the expected shear strength obtained in the appropriate pavement layer (Viz Figure 2, developed using linear elastic layered theory).

Figure 2. Maximum Shear Stress versus Depth for a Typical Unbound Pavement



Modified-Soil Subgrade Design. The design which is outlined in Figure 3 (Step 1) assumes that any subgrade with a CBR of five or less should be modified with a stabiliser, fabric or granular fill to produce a working platform with more uniform performance with respect to bearing and to provide a transition layer between the subgrade and overlying layers. In adopting a modified subgrade it is difficult to assess the strengthened layer's full potential. If lime is being used to stabilise a cohesive subgrade soil it can fairly safely be assumed that some tensile strength will be developed in the modified layer. This factor should therefore be used in design together with the natural subgrade and stabilised subgrade CBR values.

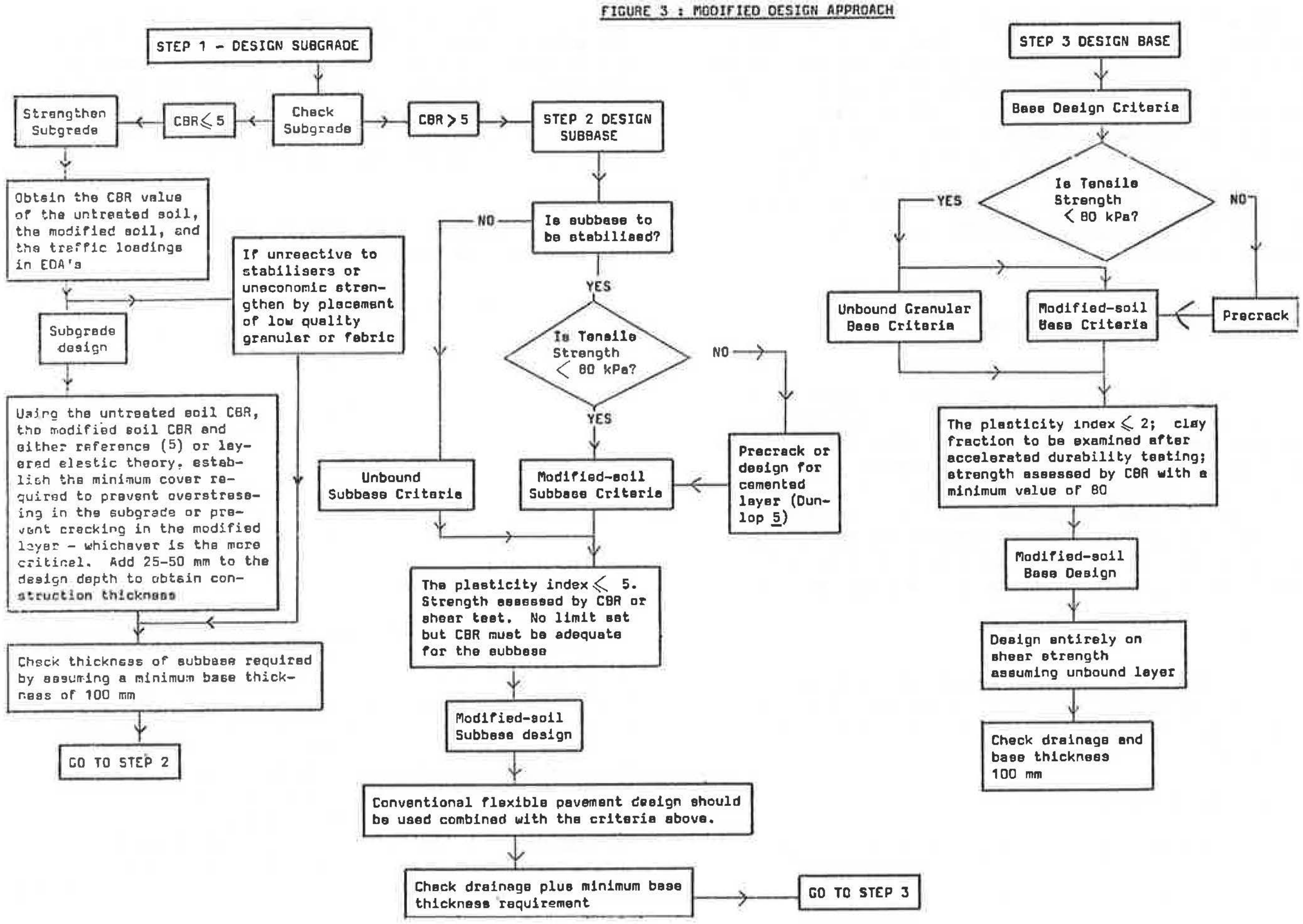
Reference (5) gives details of a simplified modified subgrade design approach, but a linear elastic multilayered program can be used in conjunction with limiting strain in the subgrade to obtain granular cover thickness. From this point it is essential to check that a subbase is required as some lightly trafficked roads need only a base layer.

Subbase Design. Step 2 on Figure 3 specifies the procedure for design of the subbase using unbound, modified-soil or cemented-soil pavement layers. Although unbound and cemented-soil layers are included, as suggested earlier in this paper a modified layer gives the best performance for capital outlay. An unbound subbase layer will be adequate provided durability and shear strength are maintained for the design life of the pavement. Both unbound and modified layers develop particle interlock which aids the load carrying ability of the layer yet does not produce undesirable reflective cracking.

Base Design. Design of the base follows down Step 3 of Figure 3 and is very similar to the subbase design. Again unbound and cemented-soil layers have been included, though the writer considers that precracking of the cemented base to form an essentially modified-soil layer is highly desirable. Precracking is the process by which a roller is used to deliberately crack a stabilised pavement layer into blocks of less than 100 mm square, thus preventing wide spaced reflective cracking.

In designing a base for a thin surfaced road it

FIGURE 3 : MODIFIED DESIGN APPROACH



is essential to determine long term durability and shear strength of the material being used. As suggested in the section on material characterisation, durability can be checked by an accelerated wetting and drying test and possibly limiting the plasticity index to less than 2. Unless large scale shear boxes are available it is suggested that a minimum CBR value of 80 be specified in conjunction with the durability requirements to ensure adequate shear strength in the base layer.

Drainage requirements of the whole pavement must also be checked to ensure that water which penetrates a cracked seal does not filter down into the subgrade and reduce bearing capacity. Also when stabilised subbase layers are used their low permeability with respect to the base may cause pore pressure dissipation problems which need to be overcome by attention to crossfalls, full construction widths and subsoil drainage.

Pavement Performance

The design concept introduced in the last section was based on many in-service observations of pavements and their performance. No design method is complete without feedback from observations of performance of constructed pavements with respect to loading and time.

Even though many "test sections" have in the past relied heavily on surface deflection and shape as performance criteria, a number of the more recent ones have been initiated with good start data. The use of "in-service" data is best demonstrated by examining the results of some test sections on a test track and on a number of roads in New Zealand.

Unsealed Pavements

In a number of recent trial sections on lightly trafficked county roads, attempts have been made to stabilise the top 150-200 mm with quicklime. After stabilising, the layer is compacted, cured and left unsealed.

One particular clay road in the Waipa County near Hamilton was treated this way with no aggregate being added to the under-designed pavement. After its second winter, its appearance is good with no potholing or corrugations. The road surface has cracked into approximately hexagonal paving block sizes and some wheel path rutting has occurred as predicted (Figure 4). While this road only carries 100 vehicles per day the savings in maintenance since completion has been approx. 60%.

In a number of other sections of road which have been lime stabilised, unbound aggregate layers of 50-100 mm have been placed over the stabilised layer to add pavement strength. Within a few months considerable potholing and corrugations developed in all sections indicating the deficiency of using an unbound base layer for an unsealed road.

The latest development has been to lime stabilise an existing unsealed road and one day after construction roll into the surface a one particle size layer of clean 37 mm aggregate. This aggregate provides resistance to traffic wheel abrasion and reduces dust loss in the summer.

Figure 4. Stabilised Unsealed Road Surface

Test Track Results

The circular test track at the University of Canterbury has been used to evaluate twelve different unbound basecourses under a standard 4.1 tonne dual wheel. The difficulty of laying basecourse and the extent of grading change with rolling was observed. It was found that refusal density could not be obtained by compaction plant, but was achieved before the pavement reached 10% of its design life (ie, traffic compaction plays an important role).

An interesting feature of this testing was the virtual lack of basecourse deformation which occurred after refusal density was obtained. This does not correspond with many studies carried out in the laboratory (12), in which samples have indicated considerable creep with accumulated traffic loading. It would appear that the densities achieved in laboratory samples are not as great as in the field, hence the continued recording of creep.

The other important point is the sensitivity of a basecourse to saturation. As the basecourse is compacted, the grading changes and more fines are produced. These fines will provide a large surface area for attraction of moisture hence dictating equilibrium moisture content. If traffic continues to compact an already dense partly saturated (by construction water) basecourse layer, instability has been shown to increase rapidly once saturation exceeds 80%.

On examination of the above results it is evident that there are many problems associated with constructing a basecourse with respect to minimising traffic densification and avoiding 80% saturation. These points tend to reinforce the modified-soil concept in which grading and moisture sensitivity are not important.

Quarry Road Test Sections

Bartley (13) in a report on 13 test sections at Quarry Road near Auckland, New Zealand, has outlined the problems associated with construction and performance of unbound, bitumen bound, foamed bitumen treated, asphaltic concrete and lime treated methods of overlay. It was apparent that materials which did not retain adequate shear strength, deformed by shallow shear. Likewise the low quality aggregates treated with 1% of hydrated lime have performed excellently in service and better than even the unbound high quality basecourses.

Bradley (14) in triaxial tests performed on both the good quality unbound and poor aggregate treated with lime found that the resilient moduli of saturated samples were 400 MPa and 1100 MPa respectively for a deviator stress of 250 kPa and a confining stress of 50 kPa. These laboratory results confirm field performance which indicates the advantages of using a material which can resist shear deformation under heavy wheel loadings.

East Coast Road Test Sections

In a series of eight test sections constructed (in 1969) and monitored in a co-operative project between Waitemata County Council and the Road Research Unit of the National Roads Board, New Zealand; bitumen treated basecourse, cement treated basecourse, local aggregates, lime stabilised subbases and conventional high quality unbound aggregates were used. From results presented in a report on this project by Malcolm et al (15) and subsequent information collected the following conclusions can be reached:

1. The bitumen treated conglomerate used as a base was found in post testing to have a soaked CBR value of 25. This treated material gave very poor performance as it deformed to an unacceptable level after 4×10^4 EDA.

2. The cement stabilised base cracked under imposed traffic loadings within the first six months.

At this stage a chip seal was applied and no further cracking has appeared on the surface. Performance has been excellent with very little visible rutting and no shallow shear failure. The 150 mm cement stabilised conglomerate layer over 50 mm of unbound conglomerate subbase on a measured CBR of 70 has carried 2.5×10^5 EDA without any signs of measurable deformation.

3. Of the unbound basecourses the performance appeared to be related to the layer thickness and the aggregate fines plasticity. Basecourse layers of 150 mm were found to be adequate while the 75 mm layer exhibited early shallow shear deformation. Likewise, the basecourse with a plasticity index of 11 also did not perform well to the extent that this section reached only one quarter of its expected design life.

4. The two sections containing lime stabilised subgrade (150 mm and 125 mm) performed well even with the minimal cover depths of 75 mm and 150 mm respectively. The most interesting point is that these two sections have carried a total of 2.3×10^5 EDA without showing any appreciable subgrade deformation even though water contents were high. These results substantiate the recommended design concept of modifying the subgrade to reduce sensitivity to water and improve load carrying ability.

General Comments on Pavement Performance

Many other case histories could be mentioned (4), but the following concluding comments will reflect the findings:

1. Unbound basecourses which produced soaked

CBR values of less than 50 when tested in the laboratory were providing unsatisfactory performance in service by exhibiting shallow shear deformations.

2. Many unbound aggregates which were apparently satisfactory before being placed in a pavement had deteriorated.

3. Neither lime nor cement stabilised aggregates have shown these changes in durability properties.

4. Stabilised shoulders appear to reduce incidence of edge breaks, shoulder wear and water infiltration under the sealed pavement.

Concluding Comments

This paper has attempted to outline the main deformation modes which cause loss of surface shape in unsealed and thin surfaced roads and then suggests design techniques which will limit the rate at which these deformations occur. In the development of stabilised surfacings it must be appreciated that their use should be confined to areas where frost penetration does not occur. Nevertheless many countries throughout the world do not have a freezing problem and therefore could benefit from the use of stabilised unsealed roads.

For a thin surfaced road the Modified Design Approach outlined in Figure 3 provides a designer with a simple method by which the typical low volume road can be designed. By improving the input data and design sophistication may be more predictable pavements could be built, but for roads carrying low traffic volumes it can be shown that very thin pavement depths are adequate.

Many of the analytical techniques employing high speed computers do not predict the performance of pavements built of unbound or modified materials. While the initial consolidation of a newly constructed subgrade (deep seated deformation) can be predicted the unique characteristics of unbound and modified materials appear to improve the pavement's overall performance over that predicted analytically. This could be attributed to particle interlock which tends to develop within a pavement layer to form a weak slab. Provided physical and chemical degradation do not occur the unbound or modified layer should last indefinitely.

In concluding that deep seated deformation will always occur the Modified Design Approach has suggested that every weak subgrade should be modified before being overlaid with the remaining pavement layers. By modifying the subgrade with lime, the rate at which deep seated deformation develops is reduced. The lime will improve the soil's sensitivity to water infiltration, even out bearing capacities in the subgrade, and prevent clay infiltration into upper unbound pavement layers.

To conclude, it should be emphasised that pavement design can be reduced to fundamental considerations providing the designer thinks about the structural design and then follows a method similar to that described in this paper. Observation of pavement performance is still and will be for many years to come one of the most valuable means by which the local practitioner can gain the necessary design skills. No designer with the aid of the most powerful computer, yet no appreciation of pavement mode of action and its likely performance in service, will produce an adequate pavement design at an economic level.

Also it must be emphasised that correct construction techniques, adequate quality control and good design are equally important as neglect of any phase will cause the performance of the pavement to suffer. Stabilised layers can play a major role in providing cost savings and longer pavement life when compared with conventional unbound construction. With the limited availability of high quality aggregates and the increasing cost of moving them to the construction site, use of modified local materials in the pavement structure must continue to increase.

Acknowledgement

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AN ALTERNATIVE TO THE DESIGN SPEED CONCEPT FOR LOW SPEED ALINEMENT DESIGN

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While the design speed concept originated from considerations of driver speed behavior, it is now treated as an arbitrary means of designing and matching geometric elements. The implicit assumption of a maximum uniform driving speed is examined in terms of Australian research into the relationships between driver speed behavior and alinement design. For alinements based on design speeds of 110 km/h or more, driver behavior appears to be in accord with the design speed concept. However, for alinements with design speeds between 90 and 110 km/h, driver speeds tend to vary according to the standard of individual features, but the speeds adopted on horizontal curves are generally below the curve design speed. For alinements with design speeds of 90 km/h or less, driver speeds vary along the route and are consistently in excess of the design speed. The results of the speed studies have been used to formulate an alternative approach for the alinement design of two-lane rural roads where topographic or financial difficulties preclude the adoption of design speeds greater than 90 km/h. The method is based on the estimation of a desired speed of travel as related to terrain classification and overall alinement standard. This is used to predict the speed behavior of drivers on individual horizontal curves as a function of the curve standard. This method provides quantification, in terms of driver speed behavior, of what represents a sub-standard curve relative to the overall alinement standard.

As indicated by Table 1, extremes in several demographic characteristics place Australia in a unique position with regard to rural roads and rural road transport. Roads and road transport play an important role in Australian economic and social activity, yet much of the primary highway network is carrying traffic volumes typical of what are regarded as low volume roads in many other countries. While a number of the more significant routes have been designated as National Highways and are being progressively upgraded to a high geometric standard, economics demand that much of the primary network remain at a relatively low geometric standard (single carriageway with constrained alinement).

Australian road authorities are confronted with the problem of designing roads for long distance, and

Table 1. Summary Australian population, vehicle and road statistics (1972).

Country	Population (millions)	Population per sq.km	Motor Vehicles per 100 Population	Road Length per 100 Population (km)
Australia	13.1	1.7	40.3	6.6
U.S.A.	208.8	22.3	56.7	2.9
Gt. Britain	54.2	235.6	26.7	0.6
Germany	62.0	248.0	28.4	0.7

often high speed, travel while only modest geometric standards can be afforded. The designers have been forced to look closely at the principles underlying geometric design practices, and this has led to a growing suspicion that the traditional design speed approach may not be appropriate for the design of the lower speed range alinements (1, 2). Accident studies suggested that horizontal curves had a considerable influence on the safety of traffic operations on such roads (3). Other studies revealed that, relative to other alinement properties, road curvature had the greatest influence on driver speed behavior (4).

These considerations caused the Australian Road Research Board (ARRB) to undertake a review of the design speed concept and its application to horizontal alinement design practice, and to carry out research into the relationship between horizontal alinement and driver speed behavior. The present paper summarizes the results of this research and the recommendations that have arisen from it. Full details of the work have been published in the project reports (5,6,7,8,9).

The Geometric Road Design Committee of the National Association of Australian State Road Authorities (NAASRA) played an active advisory and reviewing role during the course of this work and is currently examining the results and recommendations in conjunction with a revision of the NAASRA geometric design policy (10).

Evolution Of The Design Speed Concept

In the 1920s roads were located on long tangent sections as much as possible. The radii of the curves joining these tangents were determined solely by the topography and available funds. Little thought was given to the actual speeds at which vehicles might negotiate the curves, or to consistency in curve design. During the 1930s attention was given to the simple relationship between radius, superelevation, vehicle speed and centripetal force, resulting in a practice of superelevating curves to resist the total centripetal force for an assumed speed. While speed limits were low, the legal limit provided the assumed speed, but as limits were raised it became apparent that some alternative approach to design was required.

Barnett(11) provided the first formal definition of design speed and the design speed concept. Following field trials on road curves by volunteer subjects conducted by the Bureau of Public Roads, Barnett recommended that superelevation be designed to counteract the centripetal force for 0.75 of the assumed design speed, relying on side-friction to supply the remaining horizontal resistance. He defined the assumed design speed as:

'the maximum reasonably uniform speed which would be adopted by the faster driving group of vehicle operators, once clear of urban areas'.

While his design speed approach was developed specifically for determining design values for curve radius and superelevation, Barnett argued that all features of geometric design should be made consistent with the chosen design speed with a view to achieving balanced design.

The American Association of State Highway Officials (AASHO) gave official endorsement to the design speed concept in its 1938 'Policy on Highway Classification' (12). This policy defined design speed as:

'the maximum approximately uniform speed which probably would be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones'

The publication of AASHO's 'Policy on Geometric Design of Rural Highways' (13, 14) saw the design speed concept as it is known today. Here design speed is defined as:

'a speed used for the design and correlation of the physical features of a highway that influence vehicle operation'

and as

'the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern'

While these publications realized 'balanced design' as envisaged by Barnett, with minimum standards for all design elements being related to the chosen design speed, it shifted the design speed concept itself away from the behavioural measure proposed by him. Design speed is no longer the speed adopted by 'the faster driving group of vehicle operators', but has become a design procedural value used for the 'design and correlation' of design elements which is also a 'maximum safe speed'. This shift in interpretation has an important bearing on the subsequent development of this paper.

Australian road authorities have tended to follow American geometric road design practices. NAASRA (10) employs the design speed concept in much the same way as AASHO, with design speed being defined as the speed at which a vehicle can travel:

'without being exposed to hazards arising from curtailed sight distance, inappropriately superelevated curves, severe grades or pavements too narrow to accommodate the design volume'

Critique Of Current Alinement Design Practice

Design Speed Concept

While most Australian road authorities continue to use design speed as the basis for alinement design, there has been a growing suspicion that the concept may have deficiencies if applied in a literal sense. Three related criticisms of the design speed approach were raised by the author in a recent review of the design speed concept and its current application (5).

Type 1 Criticisms. Designing according to the design values permitted by a specified design speed does not necessarily ensure consistent alinement standards.

Design speed, as defined by both AASHO (14) and NAASRA (10) only really has meaning in the presence of physical roadway characteristics which limit the safe speed of travel. This is not the case for level, tangent sections. Even for physical features that limit safe speed of travel, the design speed only specifies minimum values; above minimum values are recommended wherever terrain and economy permit. Thus, a road can be designed with a constant design speed as conceived by the designer, yet have considerable variation in speed standard and, to a driver, appear to have a wide variation in design standard.

Type 2 Criticisms. Designing according to the design values permitted by a specified design speed does not necessarily ensure compatibility between the standards for combinations of design elements.

Minimum values for alinement are based on the safe operations, as defined by design criteria such as side friction factor, for a vehicle travelling at the design speed negotiating such features in isolation. In rolling and mountainous terrain, it is frequently necessary for vertical alinement elements to be combined with horizontal curves. Adequate minimum values for isolated elements do not provide the same level of safety when the elements occur in combination. Consequently design policies and manuals emphasise the importance of avoiding combinations of minimum values.

Statements on avoidance of combinations of minimum values are, in effect, an amendment to the design speed approach to alinement design. However, neither such statements, nor the design speed concept itself, guide the designer as to acceptable or appropriate combinations of values.

Type 3 Criticisms. Free vehicle operating speeds and design speed are not necessarily synonomous.

AASHO (14 p87) argues that, on rural highways, most drivers aim to travel at an 'approximately uniform speed'. (The original design speed concept is, to a large extent, based on such an assumption.) While this may be true for freeway standards, experience suggests that it does not accord with driver behavior on lower standard alinements. A driver adjusts his speed according to his desired speed of travel and the perceived hazard. As discussed above, the speed standard, and hence the perception of hazard presented by the alinement, may vary along a road designed with a constant design speed. The speed adopted by a driver tends to vary accordingly, and may often be in excess of the design speed. The situation is further complicated by differences in perceived hazard for different alinement elements. Entering a horizontal curve at an excessive speed will almost certainly result in a loss of control situation, so drivers adjust their speed accordingly. However, the possibility of curtailed sight distances concealing a hazard is perceived as remote, so drivers do not generally adjust their speed to a level commensurate with sight distance restrictions.

Curve Design Standards

Current minimum curve design standards are based on two criteria:

1. Ensuring that the side friction demand is not excessive for the design speed.
2. Ensuring that the sight distance is adequate for the design speed.

The first criterion has developed from railway engineering practice. It is based on the side force required for a vehicle to traverse a curve at the design speed and at a constant radius equal to the curve radius. The design standards are derived from the equation:

$$e + f = \frac{V^2}{127R} \qquad (1)$$

where: e = curve superelevation (m/m),
f = side friction factor (side force/force normal to the pavement),
V = vehicle speed (or design speed) (km/h),
and R = curve radius (m).

The design value for f is given as a decreasing function of design speed. The relationship between f and design speed is supposedly based on a 'driver comfort' interpretation of early empirical studies (11). However, it is often justified as being representative of the decline in pavement skid resistance with increasing speed.

The sight distance criterion is based on the minimum sight distance requirements for the design speed. The curve radius necessary to meet these requirements can be determined for the possible lateral clearance to line of sight obstructions. For high design speeds, the sight distance criterion tends to be the controlling factor.

The assumptions underlying the side friction criterion have not stood up to experimental investigation. Unlike the railway situation, road vehicles are not constrained to follow a path of fixed radius. Glennon and Weaver (15) examined vehicle trajectories on curves with radii ranging from 250 to 875 m, and found that the minimum radius on the trajectory tended to be tighter than the curve radius. Good and Joubert (16) found that, for substantial deviation angles (> 90 deg) and strong constraints on drivers' lateral positioning, this relationship also applied to curves of lower radius (18 to 116 m). However, for low radius curves with smaller deviation angles, or with room for lateral manoeuvreing, drivers tend to 'cut the corner' such that the vehicle path curvature remains less severe than the road curvature.

From a review of published speed-curve geometry data, the author (17) concluded that drivers do not respond to superelevation, and hence side friction factor, when selecting the speed at which they will traverse a curve. Road curvature appeared to be the dominant factor affecting speed. The strong relationship between speed and curvature is also at variance with the intent of the sight distance criterion. Increasing curve radius to improve sight distance may merely serve to increase operating speeds, so that the sight distance remains inadequate for the speeds that prevail.

ARRB Research Into Driver Speeds On Curves

Objectives And Data Collection

With a view to resolving differences between actual driver-vehicle behavior and the design assumptions, ARRB undertook an empirical study of driver speed behavior on horizontal curves. Speed data were collected at 120 curve sites on two-lane rural highways in three States, and on the approach tangents to the curve sites. The nominal speed standards of the curves ranged from 40 to 120 km/h. Free spot speeds were measured at 20 sites on level tangent sections, with lengths greater than 1.5 km, in the vicinity of curve sites.

Analysis And Results

Desired Speed and Overall Alinement Standard. The speed at which a driver might wish to travel a particular section of road should have a bearing on the speed at which he chooses to negotiate curves contained in that section. This speed was referred to as the 'desired speed' for the road section and was defined as the speed at which drivers choose to travel under free flow conditions when they are not constrained by alinement features.

A subjective assessment was made of each road on which curves were studied to divide it into sections of relatively uniform character, based on such factors as overall alinement standard, topography, cross-section, traffic volumes, adjacent land-use, and proximity to major urban development. The lengths of these sections ranged from 3 to 30 km. The higher value speed distributions measured on each section (measured on the better approach tangents or on long level sections) were regarded as a measure of the desired speed pertaining to the section. When directional differences occurred, separate desired speeds were estimated for each direction of travel.

Scrutiny of the data indicated that the desired speed on particular route lengths was influenced by road function, typical trip purpose and length for traffic on the road, proximity to major urban centres, and, most importantly for design purposes, by the overall standard of alinement as specified by the overall design speed and terrain type. Insufficient data were available to specify desired speeds for all circumstances. Table 2 gives the 85th percentile desired speeds that can be expected for the most common road conditions encountered during the research.

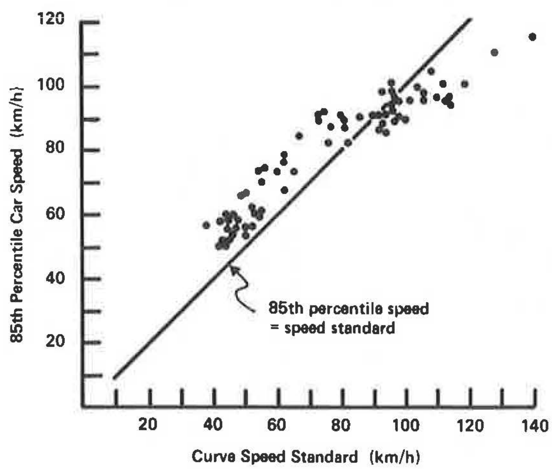
Observed Curve Speed and Speed Standard. The relationship between 85th percentile free speeds for cars on curves and the speed standard of curves is shown in Figure 1. The speed standard of a curve is regarded as the maximum speed at which a vehicle can negotiate the curve without exceeding the NAASRA (10) side friction factor criterion. This will often be in excess of the nominal design speed. For curves with speed standards of 100 km/h or more, 85th percentile free speeds tend to be less than the curve speed standard, while for curves of lower standard the reverse applies.

Table 2. 85th percentile desired speed of travel as a function of overall design speed and terrain type for single-carriageway rural roads with a State Highway classification.

Overall Design Speed (km/h)	Desired Speed (km/h)		
	Flat	Rolling	Mountainous
40 - 50			70*
50 - 70		90	
70 - 90		100	
90 - 120	115	110	
> 120	120		

* Under these conditions, tangent lengths are too short for a meaningful measure of 'desired speed'. The value given represents the typical maximum 85th percentile speeds measured on available tangents.

Figure 1. Relationship between observed 85th percentile car speeds and curve speed standard.



Speeds on Individual Curves. Regression analysis revealed that the observed 85th percentile curve speeds were dominantly influenced by the desired speed pertaining to the road section and the curve radius (expressed as curvature). While available sight distance had a statistically significant effect on curve speeds ($p < .05$), it represented less than one percent of the variability in observed 85th percentile speeds. The other traffic and road geometry parameters considered in the analysis failed to show a statistically significant effect on curve speeds ($p > .05$).

A regression based on desired speed and first and second order terms in curvature was found to provide a good description of the empirical data in terms of statistical significance and even spread of residuals. The resulting regression equation, with all terms significant at $p < .01$, was:

$$V_c(85) = 53.8 + .464\, V_F - 3.26\left(\frac{1}{R}\right) \times 10^3 + 8.5\left(\frac{1}{R}\right)^2 \times 10^4 \quad (2)$$

$$r^2 = .92$$

where: $V_c(85)$ = 85th percentile curve speed (km/h);

V_F = desired speed of the 85th percentile car (km/h);

R = curve radius (m), and

r^2 = proportion of variance of the dependent variable explained by the regression.

Discussion of Results

The results indicate that the interpretation of design speed, as it relates to design standards and driver speed behavior, is very much a function of the overall alinement standard of the road. For roads with design speeds greater than 110 km/h in rolling terrain, or 120 km/h in flat terrain, 85th percentile desired speeds are less than the design speeds, and the original (11) concept of design speed applies.

In rolling terrain, for roads with design speeds of 100 or 110 km/h, speeds will vary according to the speed standards of individual features. However, the 85th percentile speeds on horizontal curves will not generally exceed the speed standard of those curves, though this may not hold for other design features such as crests. This situation is subject to the Type 3 criticisms of the design speed concept. Increases in alinement standards within this range would serve to reduce the basis for such criticisms.

When the design speed is 90 km/h or less, the 85th percentile driver will be traversing all sections of the road with a free speed in excess of the design speed. The Type 3 criticisms of the design speed concept apply over the length of the road, and attempts to overcome such criticisms by increasing curve radii will only serve to increase operating speeds by a commensurate amount.

The three general relationships (desired speed vs overall alinement standard, curve speed vs curve speed standard, and curve speed vs desired speed and curvature) have a circularity which suggests that it mav -.t be feasible to produce a design procedure whereby the higher percentile speeds can be accommodated within the current criteria for safe operations. Increases in overall alinement standard will serve to increase the desired speed of travel which will, in turn, increase the operating speed on individual alinement features.

An Alternative Approach to Low Speed Alinement Design

Rationale

The remainder of this paper describes the development of an alternative approach to constrained alinement design based on predicted 85th percentile speeds. (The 85th percentile approximates a point of inflexion in the normal distribution curve, and, as speeds tend to be normally distributed, is likely to represent the point of diminishing returns when designing according to a percentile speed value.) This approach is most relevent to the design of horizontal alinements where terrain and/or financial constraints necessitate the use of standards corresponding to a design speed of 90 km/h or less. The variation in driven speed along the road is allowed for, and each alinement feature is designed according to the predicted speed of travel for the faster drivers. To this extent, it is a return to the original (11) concept of a design speed, but without the assumption of a uniform speed of travel. A suggested design procedure is outlined in Figure 2, and further details are given in reference (9).

It is suggested that current design standards and procedures be retained for alinements based on design speeds of 100 km/h or greater. On such alinements, driven curve speeds tend to be conservative relative to the design speed standards. This is in keeping with the view that the objective of high standard alinement is to provide a high level of comfort and convenience for the widest possible range of road users.

Curve Speed Prediction

The ARRB research showed that 85th percentile curve speed is determined largely by the desired speed of travel pertaining to the route section and the curve radius. While the regression equation (eqn 2) was appealing for its simplicity, and was very successful in terms of explaining the variability in observed curve speeds, it tends to produce anomalous results in the extremes of the data range.

The data were subsequently partitioned into four groups according to the desired speed value, and separate speed vs curvature regressions applied to each group. Higher order curvature terms failed to produce statistically significant improvements for the grouped data regressions, so four linear speed-curvature equations resulted. The regression coefficients were then iterated or extrapolated against desired speed value to produce the family of curve speed prediction relationships shown plotted against radius in Figure 3. The original data were used to check the validity of these relationships and, with observed desired speed rounded to the nearest 5 km/h, the family of relationships explained a greater proportion of curve speed variability than did eqn 2.

Each relationship shown in Figure 3 can be interpreted as the 85th percentile speed vs curve radius relationship pertaining to a length of road with a relatively uniform alinement standard giving rise to the desired speed shown. This gives the speed which should be used for the design of a curve of specified radius.

Side Friction Factor Design Criterion

A review of recent literature relating to driver behavior on curves (8) found that drivers do not adjust their speed on curves according to the f value utilized, and that the values actually utilized are often well in excess of the assumed design values, particularly on low standard curves. Furthermore, because of the variations in vehicle path radius, the actual tire/pavement friction force can vary appreciably from the f value given by the circular path formula (eqn 1). Despite these criticisms, designers still consider that f is both a necessary and valid design criterion. The difference between f and the actual friction available is the most important factor affecting safe vehicle operations on curves, and, as maintenance of pavement skid resistance is outside the realm of geometric design, the designer must concentrate on the friction demand contribution to the difference.

If f is to remain as a fundamental curve design criterion, the driver behavior literature suggests that it should be based on a different conceptual framework from that currently employed. In particular, f demand should be seen as an outcome of driver behavior, rather than as a representation of it (18). The objective of alinement design for the f criterion is, then, to ensure that the design driver does not exceed the design values of f. For such an approach, design values must be based on a realistic assessment of the behavior and comfort tolerance of modern drivers, and the pavement skid resistance that can be anticipated.

The design f values shown in Figure 4 are based on an assessment of the limits acceptable to the 85th percentile drivers observed during the ARRB research (8). As they have been derived from the circular path formula (eqn 1), they are appropriate for use in this formula. The values shown for speeds greater than 90 km/h are in excess of those likely to be required by the 85th percentile driver, in keeping with the concept that high speed alinements should provide a high degree of comfort and safety for all road users. The range of Side Force Coefficients measured on curves during routine pavement friction surveys by the Victorian Country Roads Board is shown for comparison.

Horizontal Alinement Standards

Minimum Curve Radii. NAASRA (10) specifies maximum superelevation rates of .06 in easy terrain and .10 in difficult terrain, with an absolute maximum value of .12 permitted in mountainous terrain. The curve speed prediction relationships in Figure 3 and the maximum design f values in Figure 4 can be used to compute the minimum curve radii corresponding to these superelevation rates for each desired speed. These are given in Table 3, together with the corresponding predicted curve speeds.

Above Minimum Radius Curves. In keeping with normal design practice, superelevation rates can be reduced on above minimum radius curves. The reduction in superelevation must be balanced against the desirability of reducing the expected side friction factor to a value below the design maximum. The superelevation rates suggested in Figure 5 are based on equalizing these two reductions when the required superelevation rate is less than .10.

Individual Curve Speed Standards. The speed standard of an individual curve is defined as the maximum speed at which it can be traversed without exceeding the design f criterion. For above minimum radius curves, this will generally be greater than the predicted speed. Figure 6 shows the relationship between curve speed standard, radius and superelevation.

Estimated Desired Speed (or Speed Environment). The speed at which a driver will travel a particular road section when unconstrained by traffic or alinement elements has an important bearing on curve speed selection. This speed is largely determined by the impression the driver gains of the overall alinement standard. This parameter has been referred to as a 'desired speed' when used in the context of driver behavior. However, the term 'speed environment' (after Armstrong, 1) has come into usage when the parameter is regarded as a property of the alinement design.

Table 2 gives typical desired speed values as a function of terrain type and the conventional design speed values. Table 4 presents this information, with some interpolation and extrapolation, in terms of the proposed alternative approach to alinement design. At the concept level, the 'speed environment value' would serve the same function as the current use of a 'ruling design speed'.

Figure 2. Suggested alinement design procedure.

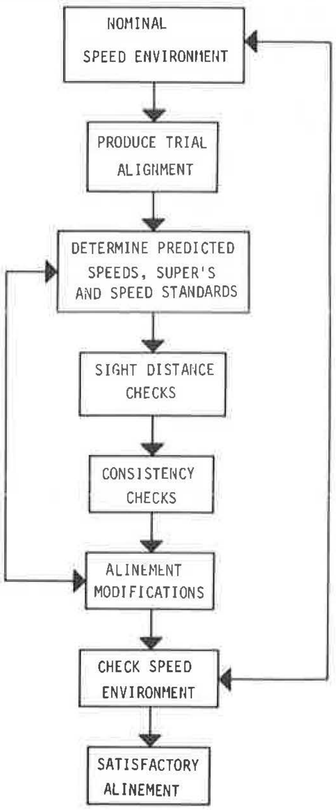


Figure 3. Curve speed prediction relationships.

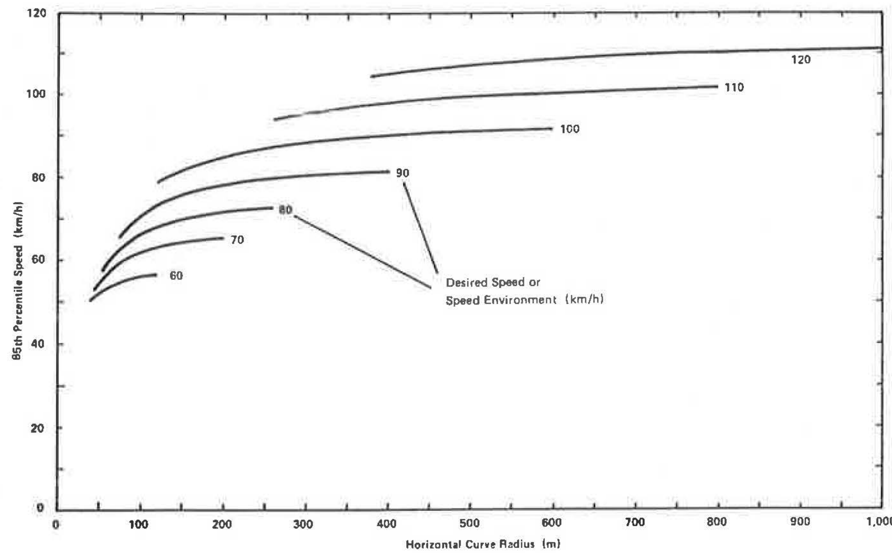


Table 3. Minimum curve radius and corresponding curve speed pertaining to desired speed or speed environment values.

Desired Speed or Speed Environment (km/h)	e = 0.12		e = 0.10		e = 0.10	
	Min. Radius (m)	Curve Speed (km/h)	Min. Radius (m)	Curve Speed (km/h)	Min. Radius (m)	Curve Speed (km/h)
60	45	50	50	50	55	55
70	50	55	60	55	70	60
80	65	60	70	60	85	65
90	85	70	95	70	120	75
100	140	80	160	85	210	85
110			300	95	400	100

Figure 4. Proposed maximum design f values for use with predicted curve speeds compared with the range of Side Force Coefficients measured on curves by the Victorian Country Roads Board.

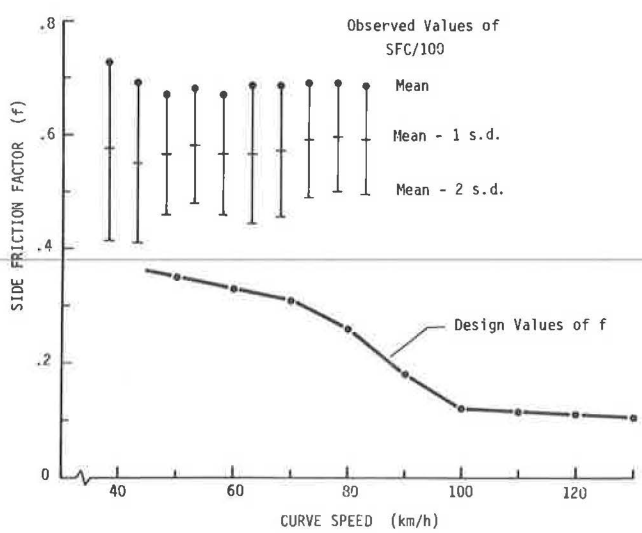


Figure 5. Recommended design superelevation rates.

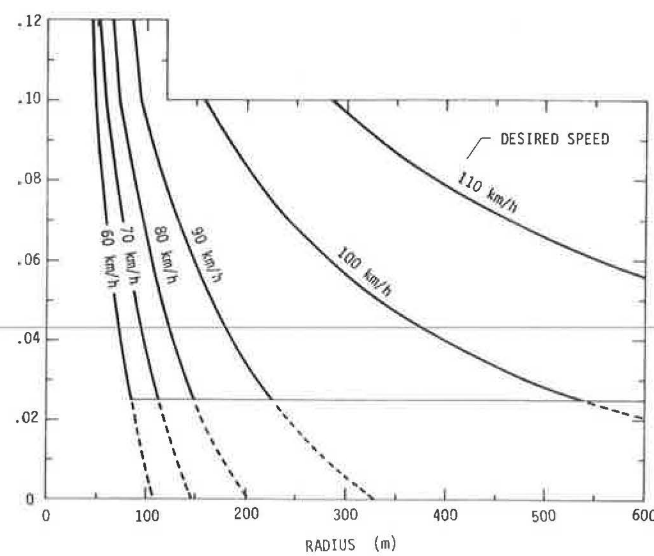


Figure 6. Curve speed standard related to radius and superelevation

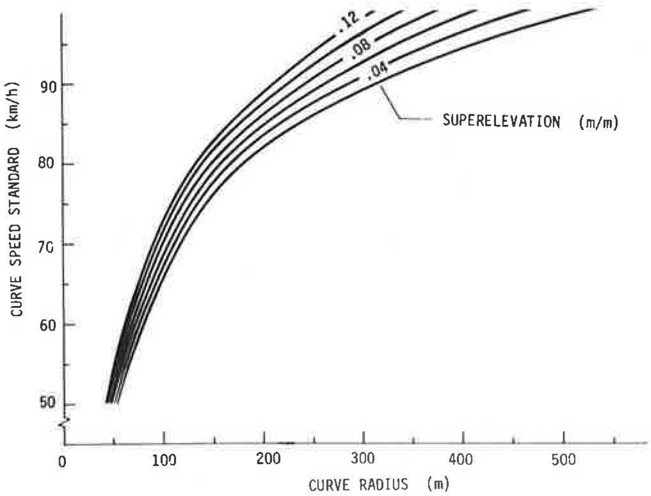


Table 4. The speed environment of two-lane rural highways as a function of horizontal alinement standard and terrain classification.

Typical Min. Curve Speed Standards (km/h)	Speed Environment (km/h)		
	Flat	Rolling	Mountainous
50			65
60		85	75
70		90	80
80		95	
90		100	
100		105	
110	115	110	
120	120	110	
>120	120		

Sight Distance Considerations

The ARRB speed studies revealed that, in terms of current NAASRA standards, the speeds at which drivers operate are often excessive for the sight distance available. While this applies to all forms of sight distance restriction, it is particularly true for sight distances restricted by crest vertical curves located on tangent sections.

Despite this anomoly, Australian designers consider that, in terms of operational experience, the current balance between horizontal and vertical alinement standards appear reasonable and should be retained. In Australian experience, serious accidents on crests are more often related to illegal overtaking manoeuvres than to stopping distance criteria, and this problem would not be alleviated by minor adjustments to minimum sight distance standards. As well as leading to additional construction costs, the lengthening of crests that would be associated with an increase in stopping distance standards would serve to reduce the total length of road with sight distance adequate for overtaking. It would also increase the difficulty of meeting the basic rule of good practice for combined alinement design that crest vertical curves should be contained within horizontal curves.

Other research at ARRB (19) suggests that modern vehicle/tire/pavement combinations are capable of achieving much higher deceleration rates than are assumed for the derivation of current stopping distance standards. With this research as justification, the design values for deceleration can be increased to maintain the current balance between horizontal and vertical alinement when a predicted speed is used as the basis for design.

Even with this amendment, it was evident that minimum horizontal sight distances appropriate to actual speeds were not being provided on many existing low radius curves which were studied, and that, when in cut, it would not be feasible to provide the necessary lateral clearance. However, as 'ran off road' is the main cause of accidents on such curves (3), the sight distance restrictions did not appear to be contributing directly. On tightly constrained alinements, drivers will be in an alerted condition, and a 2.5 sec reation time may not be required. An absolute minimum reaction time of 1.5 sec has been suggested when designing for predicted speed in constrained situations, which would accommodate most of the conditions encountered during the research. (The reduced reaction time value should not be used on isolated alinement features where the driver might well be in a relaxed state.)

Designing For Driver Expectancies

The importance of desired speed (or speed environment) for driver curve speed selection indicates that, above all else, the driver expects consistency in alinement standard. Based both on the research findings and the operational experience of major Australian road authorities, a number of rules of good practice can be formulated to ensure that the requisite degree of consistency is provided.

Section Speeds and Curve Standards. On a well designed section of road in an area with generally uniform topographical character, a driver develops a speed expectancy as quantified by the speed environment or desired speed concept. This expectancy should be reinforced by designing curves to an approximately uniform standard. Desirably, the speed standard of curves within the section should not differ by more than 10 km/h, and a 10 km/h variation in predicted curve speeds should be treated as the absolute maximum variation in curve standard. If this latter criterion cannot be met, the designer should seek to change the speed environment pertaining to the lower standard curves.

Isolated Curves. The predicted speed for curves occurring at the ends of long straights should desirably be not more than 10 km/h, and definitely not more than 15 km/h, below the speed environment pertaining to the road section. 'Long straights' is a relative term which relates to the overall alinement standard, and probably ranges from about .25 km for low standard alinements in difficult terrain to 3 km for high standard alinements in easier terrain.

Changing the Speed Environment. When a change in topographical character or some other constraint necessitates a change in alinement standard, this change should be made clear to the driver. This is best achieved by a sequence of horizontal curves, each having a predicted speed consistent with the design f criterion. When going from a high to a low standard, the predicted speed on sequential curves should not differ by more than 10 km/h, and the speed environment relevent to each curve can be taken as the predicted speed of the previous curve.

Discussion

Conservative Design Criteria

At a superficial level, it might appear that the proposed standards and procedures would result in alinements which are less safe than those based on current standards, due to the increse in design friction factor values and reduced sight distance standards. This is not the case, as the proposed values are derived from actual driver behavior on existing alinements designed according to current standards and procedures. The reality of driver behavior (for Australian drivers at least) is such that, on low standard alinements, many drivers operate with smaller safety margins than those traditionally assumed, and this fact should be recognized in design.

Traditional standards based on the design speed concept have attempted to build safety into design through the employment of very conservative design criteria. However, the ARRB research has demonstrated that for the lower range of speed standards, drivers compensate for the conservative criteria by travelling at speeds greater than the nominated design speed. The proposed alternative approach, in effect, matches the conservatism of the design criteria to the conservatism which drivers subjectively apply in actual situations.

Attempts at deriving standards from conservative criteria which are not consistent with driver behavior have led to some marked anomalies, particularly with regard to sight distance requirements. For example, AASHO (14)justifies the use of a conservative f criterion for low design speeds on the grounds that 'drivers tend to overdrive low design speed highways'. However, the minimum sight distance standards are derived from an assumed 'average running speed' which is less than the nominated design speed. A more recent amendment (20) bases desirable stopping sight distance on the nominated design speed, but this is still likely to be below the speed at which low standard alinements are 'overdriven'. The proposed alternative design standards are internally consistent, as well as being consistent with real world behavior.

Acceptance and Application

While application of the proposed procedure is recommended as a viable means of achieving effictive low speed alinement designs, it is recognized that it may not be acceptable in some authorities. Through several decades of usage, the design speed concept has become an integral part of the thinking, procedures and practices employed in most authorities for road planning, location and design. There are instances where the nomination of a ruling design speed and its associated standards has legislative significance. Removal of the design speed concept would, therefore, present problems in communication and the need for considerable retraining.

The main advantages of the proposed procedure are in its emphasis on producing alinements which are consistent with driver expectancies. Even if designs continue to be based on traditional standards, it is strongly urged that methods such as those outlined in this paper be used to check the consistency and acceptability, in terms of driver behavior, of low speed alinement designs.

Based on Australian experience, the inadequacies of the traditional design speed approach do not generally result in deficiencies in designs produced by central design offices in major road authorities. Here, the accumulated experience and expertise are applied at various parts of the design process to ensure that such deficiencies do not occur, and this probably has a greater bearing on the final product than either the design speed concept or the design standards. It is the designer of lesser experience working in isolation from centres of expertise who is most dependent on formalized design procedures. Low standard roads, for which current procedures appear deficient, are likely to be designed in this latter situation, and this is where the proposed alternative approach is likely to be of greatest use.

Continuing Research

With Australia's particular interest in low volume roads, ARRB is continuing to undertake research in this area which will lead to a more comprehensive consideration of the factors influencing the speed environment of a road. Attention is also being given to the relationship between traffic operations and alinement design standards. Indications are that, in undulating terrain and for design speed standards less than 90 km/h, improving the alinement speed standard produces only marginal improvements in traffic operations. Where traffic operations are a problem on a low speed alinement, the introduction of auxilliary lanes at various locations is probably the most cost-effective means of providing an improvement.

Conclusions

The original design speed concept was a driver behavioral approach to alinement design, with design speed being regarded as an upper estimate of a relatively uniform travel speed. While driver speed behavior no longer conforms to the assumptions implicit in the original concept, design speed is still used as a 'design procedure' directed at providing consistent and co-ordinated alinement.

For road alinements based on a design speed of 120 km/h or greater, drivers tend to adopt a relatively uniform speed which is less than the design speed. This is in keeping with the original concept, and for roads of this type, the traditional design speed approach provides a valid and rational method of alinement design.

On alinements with design speeds between 100 and 120 km/h, free operating speeds vary along the road according to the speed standard of the horizontal alinement, and, while seldom exceeding the speed standard on individual horizontal curves, they will often be in excess of the nominal design speed. While this speed variation does not appear to give rise to operational problems, it should be recognised by designers.

For the range of alinement standards below 100 km/h, driver behavior is completely at variance with the assumptions underlying the design speed approach. Free operating speeds not only vary along the road, but tend to be continually in excess of the design speed. Alinements can be designed according to a consistent design speed, yet, to a driver may appear to have markedly varying standards. Attempts to introduce additional safety through the application of additional conservatism in design criteria will only serve to increase the discrepancy between actual speeds and the assumed design speed.

An alternative method has been proposed for low speed alinement design which should overcome the deficiencies in the current design speed approach. As the method is based on observations of actual driver behavior, compatibility between the design criteria and assumptions and driver speed behavior and expectancies is ensured.

Even if the current design speed approach to alinement design is retained, it is strongly urged that quantified consistency checks, such as those presented in this paper, be included as part of the design procedure.

Acknowledgement

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OPEN GRADED EMULSION MIXES FOR USE AS ROAD SURFACES

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This paper describes the development of structural layer coefficients for open graded asphalt emulsion surfacing layers which can be used with the AASHTO pavement design method, as modified by the U. S. Forest Service, Region 6. A field survey conducted to document the history, performance and material characteristics of in-service pavements is described and this information, together with a survey of experienced users, is used to determine the most important factors affecting the design thickness for the pavements. Traffic, base type, temperature and drainage design all are important to determine the pavement thickness. Two procedures used for the development of structural layer coefficients are described. The first procedure uses the information from the performance survey and the AASHTO pavement design method to determine appropriate layer coefficients for three in-service roads. The second procedure is a fundamental approach based on layered elastic theory and using information from the field survey. This procedure is useful for the analysis of the pavement in the fully cured or partially cured state. Although a conservative approach was used for both procedures, the developed layer coefficients are greater than the values used in the current design procedure. This indicates that modification of the current values is appropriate. A tabled presentation of layer coefficients is proposed which considers only those factors found to be important for the determination of design thickness.

Open graded asphalt emulsion mixes (OGAEM) are mixtures of open graded aggregates and emulsified asphalt. An open graded aggregate is an aggregate with a low percentage of fine particles. An OGAEM is characterized by high void contents on the order of 20 to 30 percent, and typically less than 10 percent of the aggregate material passes through a No. 10 screen (1). Three typical gradation specifications used in the Pacific Northwest for OGAEM are presented in Table 1, along with a U. S. Forest Service gradation specification for a dense graded asphalt mix.

Pavements constructed with OGAEM are cold-mixed and cold-laid with conventional paving equipment. Using OGAEM with conventional equipment generally results in less pollution and lower construction costs. Construction costs are reduced because of the elimination of the operations of heating and drying the aggregate, aggregate screening and maintaining the asphalt temperature. Eliminating the aggregate dryer eliminates both a primary source of air pollution and a fire hazard and results in an energy savings by reducing fuel costs. Using damp aggregates with lower percentages of fine particles reduces the nuisance of dust (1). Asphalt emulsions are less polluting and less hazardous than solvent solutions of asphalt.

An OGAEM project constructed by the Douglas County Road Department of the State of Oregon in 1966 was one of the first projects in the Pacific Northwest. The success of this project and others prompted the U.S. Forest Service Region 6 to construct an OGAEM project in the Ochoco National Forest. Since this time, the U.S. Forest Service has become one of the largest users of OGAEM materials.

In recent years, however, the U.S. Forest Service has not been entirely satisfied with the performance of OGAEM pavements (2). As a result of apparent problems, the U.S. Forest Service, Region 6, contracted with Oregon State University to develop an improved procedure for designing pavements using OGAEM materials through proper selection of layer coefficients (a-values). The U.S. Forest Service currently uses a modified AASHTO design procedure (3). The factors considered in the determination of the structural layer coefficients for OGAEM include traffic, asphalt type, aggregate plasticity index, aggregate quality and to a limited extent "curing conditions, traffic control, compaction requirements, stockpile or aggregate uniformity requirements, etc." (3). The U.S. Forest Service recognized the limitations of the current design procedure for OGAEM materials and wished to develop a procedure to establish layer coefficients which gives consideration to laboratory stiffness tests of these materials. This report is a description of recent efforts to develop improved layer coefficients. The purpose of the report is to describe the development of a method of establishing layer coefficients, which considers laboratory test results and those factors which affect the thickness design of OGAEM pavements.

Performance of Open Graded Cold Mixes

At the time the project was initiated, there was considerable confusion as to whether or not OGAEM were performing in an acceptable manner. Some engineers indicated all cold mix jobs were 'falling apart while others indicated they provided acceptable, if not excellent, performance. Further, no clear-cut evidence was available as to which factors most affect the performance of these type mixes. To clarify this confusion, extensive field and questionnaire surveys were conducted.

Field Performance Survey

Fourteen projects throughout Oregon and Washington were selected for survey. They included variations in geographic region, traffic, climate, and responsible agency. Because the primary purpose of this survey was to observe different types of distress and to provide information for development of layer coefficients, random sampling techniques were not considered necessary. For each project ride quality, pavement condition and drainage conditions were rated. A standard survey form was developed to assure complete collection of the data and facilitate the compilation.

Each project was driven at normal speeds in a passenger car. Each evaluator independently rated the riding quality and overall evaluation (ranging from very poor to very good). Roughness or patching due to slope failures was not considered in the evaluation process. For each project the following pavement conditions were rated: percent cracking (alligator and longitudinal), depth and amount of rutting, degree of ravelling (presence of potholes would be severe), percent maintenance patch, percent surface seal retained, surface texture, drainage conditions, and amount of asphalt observed by visual inspection.

The actual thickness of emulsion mix was measured at each stop. A rut-depth meter was used to determine the depth of rutting. Where possible, the types and causes of distress, as well as the reasons for no distress, were documented. A summary of selected data from the survey is given in Table 2. The ride quality ratings of the surveyed pavements ranged from 4.2 to 9.2 with an average value of approximately 7.6 (scale of 0 to 10 where 10 is excellent). The average overall rating averaged approximately 7.9 out of 10.

The types of distress observed in the open graded mixes included distortion in the form of rutting, alligator cracking, ravelling, and poor ride quality. Rutting was observed up to depths of 1.3 cm (1/2 in.) with an average value of 1 cm (3/8 in.). The only section which exhibited considerable cracking was a thin 5 cm (2 in.) section. Other projects exhibited only small amounts of local cracking which could often be attributed to drainage problems. Ravelling of the surface treatment was observed on several of the projects. The poor ride quality observed for some projects was attributed to built-in roughness, or cracking caused by thin sections or drainage problems.

For each project surveyed, construction information, traffic data, materials type and materials properties were also obtained. The average age of the projects surveyed was five years and the most recently constructed project surveyed was two years old. There has been mention of projects constructed in Region 6 which have apparently failed in less time. None of these jobs were included in this survey.

Ten cm (4 in.) diameter cores of the OGAEM were obtained at all project sites in order to determine the resilient modulus, gradation, and residual asphalt content and properties (4). The resilient modulus tests were conducted using a diametral repeated load test system (5). The results of the tests indicate the average modulus to be on the order of 1380 to 2760 MPa (200,000 to 400,000 psi).

Based on the variable performance observed in the field survey, it was concluded that several factors other than traffic affect the performance of open graded emulsion mixes. These factors include environment, quality control, subgrade and base type, and drainage. Weather or climate have the greatest effect on emulsion mixes during the curing stage of the mix. Curing studies (6) conducted at Oregon State University and reported in the literature have shown that cooler temperatures tend to retard curing. However, the temperature dependency of the open graded emulsion mixes results in high modulus values at cool temperatures (6). This fact may account for the observed success of the roads constructed in cooler climates. Rain falling on an unbroken mix also can cause problems by washing the emulsion out of the mix (2). This is typically only a problem if a heavy rainfall occurs during the laydown process or before the emulsion breaks.

The need for improved quality control was observed on several of the projects. Non-compliance to specifications was observed for thickness, aggregate gradation and emulsion content. The open graded jobs are usually subject to less quality control than a typical hot mix job.

Materials varied for the projects surveyed. Cleanliness of the aggregate was the most noticeable difference, with the cleaner aggregates exhibiting better coating. Variations between design and extracted emulsion contents were as much as 2 1/2 percent, indicating the level of quality control experienced with these types of projects. The effect of emulsion type was not treated in this study. All were manufactured by Chevron U.S.A. and all were either CMS-2 or CMS-2h.

Subgrade and base type also appear to greatly influence the performance of open graded emulsion mixes. Both Lewis River and Merrill Lake (Section 1) roads have been subjected to considerable traffic; however, both projects show very little distress. This could be attributed to the very good subgrade and base layers (average R values of 64 for Lewis River and 69 for Section 1 of Merrill Lake) that were observed by the rating party (4).

Distress was also observed wherever drainage problems (standing water or springs) existed. Thus it is apparent that removal of water from beneath the pavement is a necessary part of design for open graded emulsion mixes, just as it is with the design of conventional hot mixes.

The survey was not of sufficient size to rank the importance of the factors which affect performance. This was accomplished by interviews with experienced users of OGAEM materials.

Factors Affecting Performance

To assess the relative importance of the factors which apparently affect performance of OGAEM materials, a questionnaire was distributed to experienced users to rank the importance of the factors affecting performance and to evaluate how the various factors might be accounted for through pavement design.

The literature review and field survey yielded twenty-four factors which apparently affected performance. These factors were listed in two questionnaire forms and sent to user agencies, contractors and researchers to rank the relative significance of each factor and to indicate how these factors could be accommodated through improved design.

The initial questionnaire resulted in twenty-three factors verified as affecting performance. The factors found to affect open graded emulsion mixes more than hot mix include seal coat application, curing temperatures, rainfall during curing, and humidity. Free draining characteristics and ravelling problems of open graded mixes generally require a seal coat to be used. The latter three factors listed are environmental factors which demonstrate the different nature of the two materials. Hot mixes "set" upon cooling, whereas emulsion mixes cure with time.

The mixer performance is also more important for a hot mix material than an emulsion mix, because a portion of the mixing of the emulsion mix actually occurs during the laydown operations. This does not occur to the same extent for hot mixes, so mixer performance is of greater importance. A point stressed by several contractors and user agencies is the importance of aggregate gradation. A "dirty" aggregate with a high fines content has a larger surface area. This can cause problems with coating or premature breaking. The need for additional quality control of both materials and construction procedure factors was expressed by a majority of the respondents.

The results of the follow-up questionnaire demonstrate the necessity of good quality control and specifications for open graded emulsion mixes. Table 3 summarizes the areas in which improvements might be required if an unfavorable condition is encountered. Nearly every factor which affects performance is subject to improvement by quality control or specifications. The factors that could require design thickness modifications include the amount and type of traffic loads, curing temperatures, base type, and drainage design. These factors are not subject to quality control to the same extent as other factors. The designer should be able to predict these factors; thus these factors can and should be considered in a thickness design procedure.

The design thickness of a pavement layer is dependent on the number of load repetitions, the strength of the pavement material, and the strength of the pavement support. The expected amount and type of traffic can be converted to equivalent axle loads (4) to represent the number of load repetitions. The strength of an emulsion mix paving material is a function of temperature and degree of curing (6). The climate of a road location can be used by the designer to estimate the temperature effects on curing and modulus. The strength of the pavement support is related directly to the type of base material. Subgrade and base materials are usually weakened by the presence of water, so drainage provisions are an important consideration when the base strength is estimated.

Development of Improved Layer Coefficients For OGAEM

Two techniques were employed in the development of improved layer coefficients for use by the Forest Service. The first method is based on observations of in-service roads and use of the AASHTO procedure (Chapter 50) to estimate layer coefficients (3). This method is particularly useful for the estimation of minimum values for layer coefficients. The second method of layer coefficient development is based on layered elastic theory and improved laboratory characterization of materials to calculate layer thicknesses to preclude fatigue and rutting (4). These thicknesses are compared with those for dense graded asphalt concrete to establish layer equivalency factors and using these factors, layer coefficients are developed.

In-Service Roads

To evaluate the layer coefficients of OGAEM materials, three projects evaluated during the performance survey were selected for further analysis: Merrill Lake Road and Lewis River Road in the Gifford Pinchot National Forest, and Burns-Izee Road in the Ochoco National Forest.

In each project test pits were excavated for collection of layer samples, measurements of in-place densities and moisture contents, and observations of the pavement condition. Measurements of densities and moisture contents were made using nuclear testing equipment.

The material samples from all projects were analyzed to determine gradation and stabilometer resistance values (R values). CBR values were also measured for the subgrade samples obtained from the Gifford Pinchot National Forest. The resilient modulus values for the surface OGAEM materials were measured (using a diametral testing device) as part of the performance survey. The resilient modulus values for the other material layers were measured using conventional triaxial testing procedures (8).

This materials information is used along with the Forest Service thickness design procedure (Chapter 50) to estimate the layer coefficient of each surfacing layer. This procedure is useful for bracketing the design layer coefficient values. When a pavement in good condition is analyzed, the layer coefficient value determined will be less than the design value because the pavement system will support additional traffic before failure occurs. Any additional traffic will result in an increase in the weighted structural number of the pavement and a subsequent increase in the layer coefficient. In the case of a failed pavement, the determined value of the layer coefficient will exceed the appropriate design coefficient. The value determined in this case is the layer coefficient that would result if the pavement system had survived. The analysis has been conducted for the nine test pits excavated in the three selected roads. Using the measured thicknesses, the estimate of traffic, a regional factor of 2.0, and the soil support values, it was possible to determine an "a-value" for each surfacing layer.

Table 4 lists the values used for the determination of the layer coefficients of the OGAEM surfacing layers and the results. For Lewis River Road an a-value greater than 0.25 would be appropriate. An a-value between 0.33 and 0.60 is bracketed by Merrill Lake Road. The Burns-Izee Road analysis resulted in a minimum a-value of 0.39. Each of the minimum values is larger than the most optimistic value that could be derived using the current method in Chapter 50 (3).

The method used to back calculate the a-values of in-service roads is subject to errors from testing, correlations and traffic determinations. However, a conservative approach has been used so that the determined a-values are minimum values. The sample size of this investigation is not large enough for a precise determination of the range of a-values. This method is being applied to additional roads in order to refine the results. Projects approaching the point of failure are particularly useful for the method. Excavating more pits per project would reduce errors caused by materials variability and thus improve results. Accurate traffic estimates are important for this method. If the traffic history of a project is unknown, the project is not suitable for analysis.

Layered System Elastic Theory

The second approach used to develop layer coefficients is based on the concept of providing sufficient thickness to limit strain. Typically the strain limitations used are the horizontal tensile strain at the bottom of the surface layer (related to fatigue) and the vertical compressive strain at the top of the subgrade layer (related to rutting). The strains at these two points can be determined using layered system analysis techniques and laboratory test results of materials strength characteristics.

Fatigue cracking and permanent deformation in the form of rutting are the failure modes which are typically considered for this design approach (8,9). Criteria for these two modes of failure were developed as a part of this project (8). Based on the field analysis, it was determined that the failure criteria presented in Figures 1 and 2 are reasonable for OGAEM. These relationships were originally developed by Chevron, U.S.A. for dense graded emulsion mixes but seem applicable for the open graded mixes, too (8).

Layer coefficients are determined using the concept of layer equivalencies. For equivalent conditions the design thickness of hot mix and OGAEM materials are determined. The ratio of design thickness is used along with the known layer coefficient for hot mix asphalt concrete (Chapter 50), to determine the layer coefficient of the OGAEM material, i.e.

$$a_{OGAEM} = \frac{d_{Hot\ Mix}}{d_{OGAEM}} (a_{Hot\ Mix}) \qquad (1)$$

where: a = layer coefficient
d = design thickness.

The method of design thickness determination using fatigue criteria is illustrated in Figures 3 and 4. The critical horizontal tensile strains based on fatigue criteria models are determined for both materials (Figure 3). The design thicknesses which limit the strains to the appropriate critical values are then determined (Figure 4). Determination of design thickness using subgrade strain criteria is similar with the exception that an identical subgrade failure criteria model is used for both types of mixes.

The layer equivalency determinations assume resilient modulus values of 2760 MPa (400,000 psi) for hot mix and 1380 MPa (200,000 psi) for OGAEM. (4). For this study three different subgrades were analyzed. The assumed modulus values of the "good", "fair" and "poor" subgrades were 205 MPa (30,000 psi), 70 MPa (10,000 psi), and 20 MPa (3,000 psi). The base modulus has been assigned modulus values equal to 1.5 times the modulus of the subgrade. A 30.5 cm (12 in.) base layer is assumed for each case and an additional case of a 60 cm (24 in.) base layer is analyzed for the "poor" subgrade.

For a given design life, design thicknesses for hot mix and OGAEM materials were determined for each subgrade condition using the fatigue criteria (Figure 1) and the appropriate strain vs. thickness relationships. The average value of the thickness ratios is 1.27, with the values ranging from 1.23 to 1.29.

Thickness ratios have also been determined for both types of mixes using the subgrade failure criteria (Figure 2). The average value of the thickness ratios is 1.22, and the values vary between 1.21 and 1.25.

The thickness ratios tend to vary slightly for the different subgrade and base strengths, with slightly lower ratios determined for the better subgrades. The variation is minimal and consideration of this factor will not be included in this portion of the development. The maximum ratio values are the controlling values considered for the development. The thickness ratios represent layer equivalencies where the ratio value times a given thickness of hot mix asphalt concrete determines an equivalent thickness of an OGAEM layer. The fatigue criteria ratio determinations result in the larger average ratio value of 1.27. The layer coefficients for hot mix materials given in Table 5 are modified using this ratio, i.e.

$$a_{OGAEM} = \frac{a_{Hot\ Mix}}{1.27} \qquad (2)$$

to obtain the values shown for OGAEM, also listed in Table 5.

A comparison of these values with the values bracketed by the in-service roads shows good agreement. The layer coefficient bracketed by the Merrill Lake test pits is between 0.33 and 0.60. The layer coefficient determined using the layer equivalency concept is 0.31. The appropriate layer coefficient for Lewis River Road would be 0.25 and the in-service road analysis determined 0.25 to be a minimum layer coefficient. For Burns-Izee test pit number two, the minimum layer coefficient of 0.39 is considerably higher than the layer coefficient of 0.28 as developed using the equivalency concept. The Burns-Izee test pit number one minimum layer coefficient of 0.28 is a more reasonable value and corresponds well with the layer coefficient determined using the equivalency concept.

It is important to realize how the assumed modulus of the OGAEM surfacing layer affects the layer equivalency values for hot mix as determined using this procedure. The values presented in Table 5 assume a modulus value of 1380 MPa (200,000 psi) for the OGAEM and 2760 MPa (400,000 psi) for hot mix asphalt concrete. The OGAEM moduli correspond to the average moduli measured for the cores from the three roads used for the in-service road development of layer coefficients. Table 6 demonstrates the variation of the average layer equivalency value as the OGAEM modulus is varied between 690 and 2760 MPa (100,000 and 400,000 psi). The table shows that the layer equivalencies decrease as the modulus increases. Thus if a laboratory investigation indicates that the modulus of the OGAEM is expected to be relatively high, then a larger design layer coefficient closer to that of hot mix is appropriate.

The approach used to develop the layer coefficients is conservative because conservative failure criteria have been used, and a conservative value has been assumed for the modulus of the OGAEM. Despite this, the layer equivalencies are somewhat less than those currently used by the U.S. Forest Service, Region 6, as shown in Table 7. When comparing these values with what other agencies in the Pacific Northwest are using, one cannot directly compare hot mix layer equivalencies. This is because the a-values used for hot mix are considerably different, the Forest Service using a-values ranging from 0.30 - 0.42 and the other agencies using 0.28. Cold mixes must therefore be compared directly by a-values or by using a base layer equivalency. If we compare a-values, we find the high side of the Forest Service values approximately the same as the other agencies, values of 0.24 - 0.30 vs. 0.25 - 0.28. The Forest Service discounts these values for marginal quality aggregate, soft asphalt, and poor quality control. The design a-value can be as low as 0.18.

Early Cure Considerations

The criteria given are also useful for checking the adequacy of a design thickness before a pavement is fully cured. An estimation of the traffic expected during the early cure period (normally 3 months or less) is used to determine the allowable strain values from the failure criteria. The actual strain values during the early cure period are determined using layered theory for the design thickness and an appropriate early cure modulus. This strain value is checked to assure that it is less than the allowable strain. In the situation where the strains are greater than the allowable strains, the designer can increase the design thickness or require limitations of traffic during the early cure period. For details on the procedure for calculating additional thickness refer to references 4 and 7.

Conclusions and Recommendations

Conclusions

Thickness Design. OGAEM pavements have been used successfully in the Pacific Northwest by several agencies. The U.S. Forest Service has experienced success with OGAEM pavements used for light, medium and heavy traffic volumes in various climatic regions. The current Forest Service design procedure is based on the AASHTO design procedure which uses structural layer coefficients as a measure of the capabilities of materials in a pavement system. The factors considered in the determination of a layer coefficient principally include traffic, asphalt type, aggregate plasticity index, aggregate quality, and to a limited extent "curing conditions, traffic control, compaction requirements, stockpile or aggregate uniformity requirements, etc." (3).

This study has determined that the factors which affect design thickness include the amount and type of traffic, curing temperatures, base type, and drainage design. The other factors considered by the Forest Service such as asphalt type, aggregate quality, traffic control, compaction requirements, etc. apparently have relatively less effect on design thickness. These factors are important. However, they are more appropriately considered in the specifications for OGAEM materials and subjected to quality control.

A proposed table for the layer coefficient determinations considering these factors is given in Table 8. The layer coefficients presented in the table are estimates based on the layer equivalency development (Table 6). Extrapolation to other temperature conditions is not given at this time, but should be considered as a future study. If the majority of the traffic is seasonal, an appropriate temperature is the average temperature for the period of use. The surfacing structural number (SN_B) is the design input used to represent the base strength (good, fair, poor). A SN_B of 1.0 would represent the situation where one has a good subgrade while an SN_B of 3.0 would represent a poor subgrade.

The layer coefficients presented in Table 8 are best estimates based on data presented in Tables 5 6. Further in-service road testing is necessary to establish these coefficients for modular values outside the range of 1380 to 2070 MPa (200,000 to 300,000 psi). The in-service road investigations can also be used to define the failure criteria models. If the failure criteria are adequately defined, the missing values in the table can be filled in, and an early cure layer coefficient table similar to Table 8 could then be developed. This table would permit the determination of the thickness requirements of uncured mixes for the traffic expected during the early life of the pavement. The development of this table has been described in the preceding section.

A minimum thickness of OGAEM material is apparently required because of construction variability. The minimum recommended thickness over a good base, based on the performance survey, is 6.35 cm (2.5 in.) It should be emphasized that this applies to situations where there is a good quality base (R = 78, CBR = 80+). If the base material is of lower quality a greater minimum thickness would be necessary.

Specifications. The majority of the factors which affect the performance of OGAEM materials must be controlled with adequate specifications and quality control. The current Forest Service and other agency specifications all cover the factors affecting OGAEM performance as determined by the survey. Only minor variations in the specifications for the various agencies are noted. The performance survey did document non-compliance with specifications for items such as aggregate gradation, emulsion content, and layer thickness. It is therefore apparent that good quality control is not always practiced for the OGAEM material. Improvements in this area would result in more uniform projects which should improve performance.

Recommendations

Recommendations are listed throughout this report. In summary, these recommendations include the following:

1. Modify the existing layer coefficients used for OGAEM materials by the U.S. Forest Service to larger values. The in-service road analysis and the theoretical layer equivalency development, both demonstrate that the current values used are low.
2. Replace existing procedures for layer coefficients with a table similar to Table 8 in which only those factors which most affect thickness design are considered in the layer coefficient determination. Factors which affect the performance of OGAEM materials that can be controlled with specifications should be subject to quality control rather than thickness design considerations.
3. Improve quality control of OGAEM projects to assure more uniform results on the projects. Education of the project inspectors is appropriate to explain the different nature of asphalt emulsions as compared to other asphalts.
4. Continue in-service road analysis to develop layer coefficient values and refine the failure criteria models. Only those projects for which accurate traffic histories can be obtained should be analyzed. Roads for which the terminal serviceability index is approaching 2.0 (failure) are the most useful roads because the coefficients determined will be close to the design layer coefficients.
5. Analyze projects after construction to determine the materials variability experienced and the deviation from the expected results. This information is useful to the designer for comparison of the lab test results with the results obtained in the field, and to the construction supervisor for determining if changes in quality control procedures or specifications are appropriate. The establishment of materials variability will involve sampling several sections within a project. This procedure is also useful for definition of the stiffness development relationship for field conditions if the sampling is also done at time intervals.
6. Improve construction records to better document the history of each project. A documentation process accessible to the designers would allow

analysis of new processes and materials.

7. Improve traffic information collection to benefit the road system managers and the designers. Knowing the traffic history allows the road manager to predict the remaining service of a project and allows the designer to improve the design process.

8. Periodically inventory the pavement system for roadway management and for pavement design. The inventory would allow the history of project performances to be recorded, thus providing the information required to evaluate construction procedures or materials. A standardized inventory process should be developed to measure such things as ride quality, surface deflections, and distress. This information would not only be useful to the designer but also be useful in the development of maintenance management systems for the Forest Service.

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Table 1. Typical gradation specifications for open graded mixes.

Sieve Size	Percent Passing			
	USFS (Open)	Douglas County (Open)	FHWA (Open)	USFS (Dense)
25 mm	100	100	100	100
12 mm	45 - 70	-	-	60 - 85
9 mm	-	55 - 75	-	-
4.7 mm	0 - 20	25 - 45	-	35 - 60
2.0 mm	0 - 6	0 - 7	0 - 7	20 - 40
0.42 mm	-	-	-	8 - 22
0.074 mm	0 - 2	-	0 - 2	2 - 8

Note: 1 inch = 254 mm

Table 2. Summary of Riding Quality and Overall Rating of Projects Surveyed

Project	Ride Quality	Overall Evaluation
Lewis River #1	7.6	9.0
Lewis River #2	6.8	8.6
Canyon Creek #1	6.6	6.3
Canyon Creek #2	4.6	3.8
Merrill Lake #1	7.5	7.9
Merrill Lake #2	4.2	3.9
Ringo Butte	9.0	8.7
Hermiston	7.9	9.2
Charlois	7.9	9.1
Tipton	8.5	8.3
Burns Izee #1	8.9	8.0
Burns Izee #2	8.9	9.0
Silvies Van	9.2	8.5
Logan Valley	8.2	8.1
Smith River	8.0	8.5
Cow Creek	9.1	9.0
Indian Caves	6.4	8.2
Umpqua Community College	8.2	8.9
AVERAGE	7.6	7.9

Table 3. Improvement areas for the ranked factors which affect performance of OGAEM materials.

Rank	Factor	Method of Treatment			
		Quality Control	Mix Design	Pavement Design	Specifi-cations
1	Emulsion Grade and Type	X	X		X
2	Aggregate Gradation	X			X
3	Emulsion Content	X	X		X
4	Drainage Design			X	X
5	Emulsion Content Compliance	X			
6	Aggregate Gradation Compliance	X			
7	Curing Temperatures			X	X
8	Aggregate Water Content at Mixing	X	X		
9	Rainfall During Curing	X			X
10	Compaction	X			X
11	Base Type			X	X
12	Aggregate Quality	X			X
13	Thickness Compliance	X			X
14	Aggregate Uniformity	X			X
15	Amount of Traffic			X	
16	Seal Coat Application	X			X
17	Humidity During Curing				
18	Oversized Vehicle Loads			X	
19	Stockpiling Methods	X			X
20	Traffic Control	X			X
21	Mixer Performance	X			X
22	Lift Thickness	X			X
23	Laydown Machine	X			X

Table 4. Determination of layer coefficients of OGAEM surfacing layers from in-service roads.

	Lewis River		Burns Izee		Merrill Lake				
	1	2	1	2	1	2	3	4	5
TRAFFIC HISTORY									
Applied 80 KN EAL	2,910,000	2,700,000	215,000	215,000	24,000	24,000	24,000	24,000	24,000
SURFACING (OGAEM)									
Thickness (cm)	20[a]	23[b]	17	11	4	5	10	10	13
Res. Modulus (MPa)	496	1800	1620	2379	1393	1393	1393	1393	1393
BASE[c]									
Thickness (cm)	-	43	13	20	27	42	48	35	46
R Value	-	60	68	75	76	65	77	55	55
Layer Coef., a_2[d]	-	0.04	0.055	0.085	0.09	0.05	0.10	0.03	0.03
Soil Support[d]	-	6.5	6.9	7.8	8.0	6.8	8.2	6.3	6.3
Weighted Structural No.[e]	-	3.1	1.9	1.7	1.0	1.2	0.9	1.3	1.3
SUBGRADE									
R Value	76	60	71	19	72	60	46	51	68
Soil Support[d]	8.0	6.5	7.2	5.4	7.4	6.5	6.1	6.2	6.9
Weighted Structural No.	2.6	3.1	1.8	2.4	1.1	1.3	1.4	1.4	1.2
Req. Structural No. of Surface	2.0	1.6	1.9	1.7	1.0	1.2	0.9	1.3	1.3
Min. Layer Coef. of Surfacing	0.25	0.18	0.28	0.39	0.67	0.60	0.23	0.33	0.26
Performance	Good	Good	Good	Excellent	Failed	Failed	Excellent	Cracking	Excellent

[a] Supported by 10 cm Pulvermix layer, M_R = 1169 MPa, a_2 = 15
[b] Supported by 8 cm Pulvermix layer, M_R = 6860 Mpa, a_2 = 0.50
[c] Base and Subbase Combined
[d] Determined using R-Value Test Result
[e] Assumed regional factor of 2.0
Note: 1 in = 2.54 cm, 1 psi = 8.895 kPa
1 K = 4.448 KN

Table 5. Layer equivalency development of OGAEM layer coefficients.

Total 80 KN Equivalent Axles	Hot Mix Layer Coefficient[a]	OGAEM Layer Coefficient[b]
Less than 10,000	0.42	0.33
10,000 - 60,000	0.40	0.31
60,000 - 120,000	0.38	0.30
120,000 - 350,000	0.36	0.28
350,000 - 1,000,000	0.34	0.27
1,000,000 - 3,000,000	0.32	0.25
More than 3,000,000	0.30	0.24

Note: 1 K = 4.448 KN

[a] After reference 3
[b] Hot mix layer coefficient divided by 1.27

Table 6. Layer equivalency[a] variations with variation in the assumed modulus of the OGAEM layer (4).

(a) Average layer equivalency determined using the fatigue criteria model.

Subgrade Type	OGAEM Modulus, MPa			
	690	1380	2020	2760
Good	1.33	1.23	1.11	1.00
Fair	1.60	1.28	1.09	1.00
Poor	1.68	1.27	1.08	1.00
Poor, with thick base	1.77	1.28	1.12	1.00

(b) Average layer equivalency determined using the rutting criteria model.

Subgrade Type	OGAEM Modulus, MPa			
	690	1380	2020	2760
Good	1.44	1.21	1.09	1.00
Fair	1.48	1.22	1.08	1.00
Poor	1.56	1.25	1.10	1.00
Poor, with thick base	1.50	1.23	1.09	1.00

Note: 1 psi = 6.895×10^{-3} MPa

[a] Layer Equivalency =

$$\frac{\text{Design Thickness of OGAEM}}{\text{Design Thickness of Hot Mix Asphalt Concrete}}$$

Table 7. Comparison of layer coefficients between cold mix and hot mix (4)

(a) U.S. Forest Service

80 KN Axles	Cold Mix	Hot Mix	Hot Mix Equivalency	Base Layer Equivalency
< 10,000	.24 - .30	.42	1.40-1.75	1.72-2.16
10-60,000	.22 - .28	.40	1.43-1.82	1.57-2.00
60-120,000	.20 - .26	.38	1.46-1.90	1.43-1.86
120-350,000	.18 - .24	.36	1.50-2.00	1.29-1.72

(b) Other agencies

Agency	Cold Mix	Hot Mix	Hot Mix Equivalency	Base Layer Equivalency
Oregon DOT (34)	0.25	0.28	1.0 - 1.1	1.8
Washington Highway Department (38)	0.25	0.28	1.1	1.8
FHWA Office of Federal Projects (36)	0.28	0.28	1.0	2.0

Table 8. Proposed layer coefficient table for open graded emulsion mixes, at average annual temperatures of 5 - 13° C.

Surfacing Structural Number (SN_B)		1 (good subgrade)	2 (fair subgrade)	3 (poor subgrade)
Traffic 80 KN EAL	Ultimate Resilient Modulus MPa @ 23°C	Layer Coefficients		
10^4	690	0.29	0.26	0.25
	1380	0.34	0.33	0.33
	2070	0.38	0.38	0.38
10^5	690	0.26	0.24	0.23
	1380	0.31	0.30	0.30
	2070	0.34	0.35	0.35
10^6	690	0.22	0.20	0.19
	1380	0.26	0.25	0.25
	2070	0.29	0.29	0.29

Note: 1°F = 1.8°C + 32
1 Kip = 4.448 KN
1 psi = 6.895×10^{-3} MPa

Figure 1. Fatigue criteria for emulsion mixes (4)

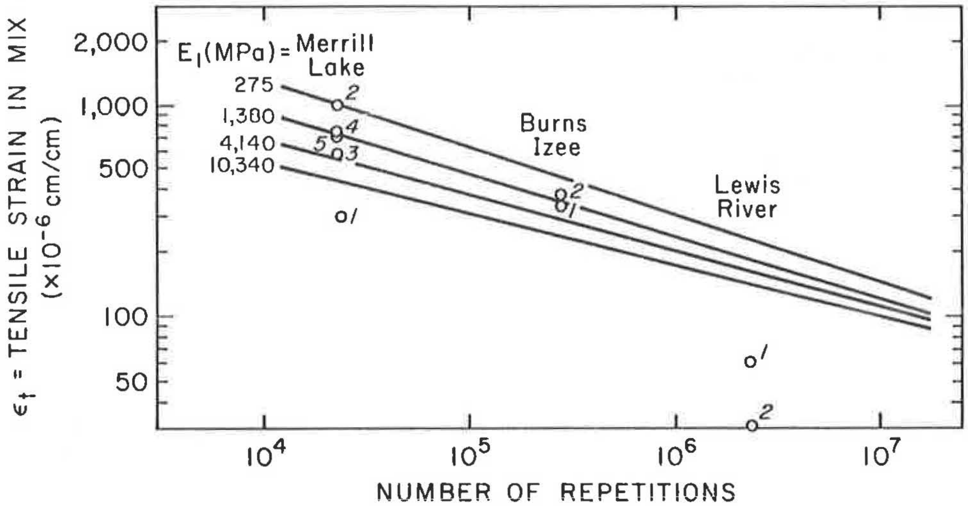


Figure 2. Subgrade strain criteria (4)

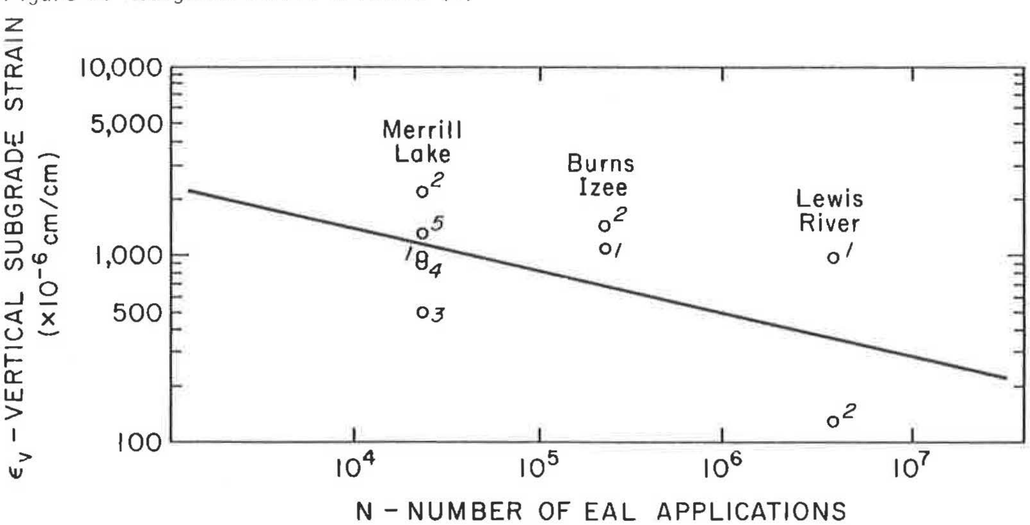


Figure 3. Determination of critical strain level based on design life

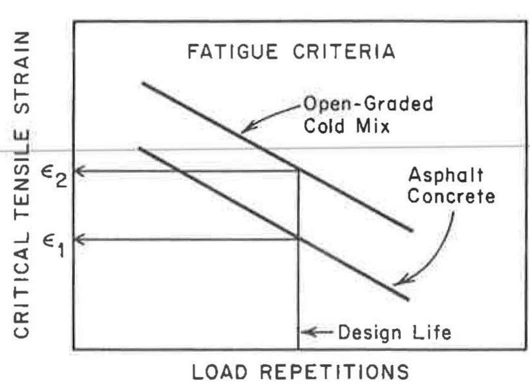
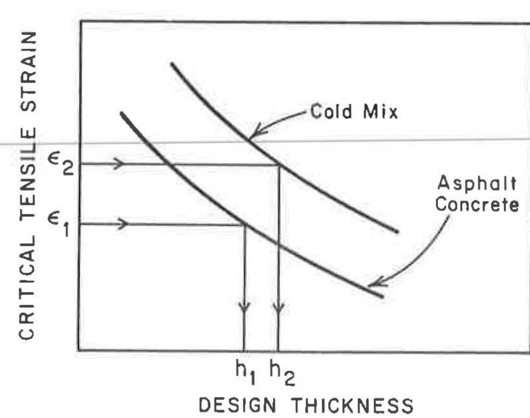


Figure 4. Determination of design thickness based on critical strain level



LOW TRAFFIC PORTLAND CEMENT CONCRETE PAVEMENT ON THE TURTLE MOUNTAIN INDIAN RESERVATION NORTH DAKOTA

DeWayne E. Storley, Department of the Interior,
Bureau of Indian Affairs

The Turtle Mountain Indian Reservation is situated at Belcourt, North Dakota, area near the Canadian border. The population density averages 55 people per mile of road. This presented problems in maintaining a gravel road, and we therefore began looking for hard all weather surfacing. We had two things to consider when we evaluated surface types. (1) We must utilize local aggregate which consisted of 65% fine aggregate and 35% coarse aggregate and also meet the quality requirements for surfacing aggregate. (2) The maintenance cost over the life of the surfacing would have to be low due to rising labor and material costs. The final decision was a 140 mm (5½ inch) non-reinforced, slip formed concrete pavement, 6.7 m (22 feet) wide. The present gravel surfaced roads were trimmed a depth of 102 mm (4 inches) by an autograder. The grade line was checked so excess trim would be avoided and yet fill areas could be avoided. This trimming resulted in the concrete being placed directly on the clay subgrade material. The trim material was used for shoulder material against the concrete pavement. The joint spacing varied from 4.27 m (14 feet) to 5.48 m (18 feet) on an 24.38 m (80 foot) cycle. The 24.38 m (80 foot) joint was formed with a premolded joint material with all intermediate joints being sawed. In three years we have completed concrete pavement on 56.3 kilometer (35 miles) of road and the maintenance costs have been only for snow removal and shoulder repairs.

By 1975 the population in the rural portions of the Turtle Mountain Indian Reservation was averaging 55 people per mile of road. Traffic consisted mainly of cars, some farm to market trucks and school buses. These had caused problems in maintenance and dust control on the gravel surfaced roads. In evaluating hard all weather surfacing we took the following two items into consideration.

(1) We must utilize local aggregate, which, when crushed consisted of 65% fine aggregate and 35% coarse aggregate.

(2) The maintenance cost over the life of the surfacing must be kept low to produce the lowest annual cost to the user in view of increasing costs for labor and materials.

The final decision was for a 140 mm (5½ inch) non-reinforced, slip formed concrete pavement 6.7 m (22 feet) wide with a 0.9 m (3 foot) unpaved shoulder on each side.

The aggregate to be used in the concrete mix came from a glacial deposit. The specifications called for crushing all material under 254 mm (10 inches) in diameter which resulted in a gradation of 65% passing the 9.5 mm (3/8 inch) sieve. Therefore our design mix consisted of about two-thirds fine aggregate and one-third coarse aggregate which was just about the opposite of our normal requirements.

The local aggregate also had shale and this problem in the coarse aggregate had to be watched closely. Our specifications for the coarse aggregate was modified to increase the maximum shale content from 0.7% to 1.5%. Excess shale was removed after crushing by washing in a sand screw and passing through a jig. We found that after crushing if the coarse aggregate was spread out in a shallow layer, rather than placed in a conical stockpile, the shale would breakdown faster and could be separated more readily during the washing operation. The shale content of the fine aggregate was not a problem.

The aggregate specifications were as follows:

Fine Aggregate

Sieve Size	Percent passing
9.5 mm (3/8 inch)	100
4.75 mm (No. 4)	95-100
1.19 mm (No. 16)	45-80
0.29 mm (No. 50)	10-30
0.15 mm (No. 100	2-5

Coarse Aggregate

Sieve Size	Percent passing
38 mm (1½ inch)	100
25.4 mm (1 inch)	50-95
19 mm (3/4 inch)	35-80
9.5 mm (3/8 inch)	10-40
4.75 mm (No. 4)	0-15
0.074 mm (No. 200)	0-3

The Physical Properties of the fine aggregate conformed to the requirements of AASHTO M6 with the following modification:

Physical Properties	Max. % by wt.
1. Clay lumps, not more than	0.5

2. Coal, Lignite and Scoria not more than 1.00
3. Material passing 0.074 mm (No. 200) sieve not more than 3.00
4. Other deleterious substances (such as shale, alkali, mica, coat grains, soft and flaky particles.) not more than 3.00
   Sum of materials listed 1, 2 and 4, not more than 4.00

The physical properties of the coarse aggregate conformed to the requirements of AASHTO M80 with the following modifications:

Physical Properties	Max. % by wt.
(a) Shale	1.5
(b) Soft Iron Oxide Particles (paint rock and ochre)	2.0
(c) Coal and Lignite	0.3
(d) Total Spall Materials (includes items a, b and c above, plus other iron oxide particles, unsound cherts, clayey, limestone particles, and other materials having similar characteristics	3.7
(e) Soft Particles (exclusive of items a, b, c and d above)	1.3
(f) Clay Balls and Lumps	0.3
(g) Sum of Materials listed under items d, e and f above (For item d, use % in total sample retained on the 4.75 mm (No. 4) sieve)	4.0
(h) Slate	3.0
(i) Thin or Elongated Pieces (maximum thickness less than 1/4 the maximum width, or maximum length more than 3 times the maximum width)	15.0
(j) Material Passing 0.074 mm (No. 200) sieve	1.0

The final concrete mix contained 234.5 Kg. (517 lbs) of type II cement per cubic yard with 0.25 Kg. (0.55 lbs) of water per pound of cement, 884.5 Kg. (1950 lbs) of fine aggregate, 442.3 Kg. (975 lbs) of coarse aggregate, 0.16 Kg. (5.5 ounces) of protex. The 28 day cylinder breaks were generally 20.68 MPa (3000 psi) or better and the beam breaks produced flexure strength of 3.10 MPa (450 psi) or better. Slump was kept at 38 mm to 51 mm (1½-2 inches).

The transverse contraction joint spacing was randomized at 4.27 m (14 feet), 4.57 m (15 feet), 4.87 m (16 feet), 5.18 m (17 feet), and 5.48 m (18 feet) with an 24.38 m (80 feet) cycle. Every fifth joint was formed with a premolded strip to induce initial cracking. In order to better control the cracking during hot weather, additional joints were formed with a premolded strip. The premolded strips had to be placed with care so that they were vertical and even with the top of the surface to prevent spalling at the joint. Transverse joints were sawed 6.4 mm (1/4 inch) wide, 28.6 mm (1-1/8 inch) deep and filled with hot poured rubber asphalt sealer. The centerline longitudinal joint had 762 mm (30 inch), 13 mm (½ inch) diameter deformed bars on 762 mm (30 inch) centers and sawed 9.5 mm (3/8 inch) wide, 34.9 mm (1-3/8 inch) deep and filled with hot poured rubber asphalt sealer.

Costs

The cost per square yard for the 140 mm (5½ inch) slab was as follows:

Year	Square Meters	Cost Per Square Meter
1975	91,445(112,895 sq yd)	7.28 (5.90 per sq yd)
1976	99,702(123,039 sq yd)	6.74 (5.46 per sq yd)
1977&78	184,441(227,705 sq yd)	7.89 (6.39 per sq yd)

Procedure

The roads to be paved on the reservation had been graded several years previously. A centerline profile was taken and a new grade line laid, which required trimming an average of 102 mm (4 inches) from the present surface. The new grade line was checked during the trimming operation so excess trim could be avoided and not allow new fill areas. The trimming was accomplished with an Autofine grader which deposited the material along both sides of the road. This material was placed along the finished slab and compacted to form 0.9 m (3 foot) wide shoulder.

Trimming an average of 102 mm (4 inches) from the roadway surface resulted in the concrete being placed directly on the clay subgrade.

The trimmed subgrade without new fill area provided a very uniformly compacted roadbed upon which the concrete surfacing was placed.

The concrete was mixed at a central plant and hauled in tandem axle, end dump trucks and dumped on the roadbed in front of the slip form paver. The vibrators on the slip form paver were slowed as much as possible to reduce the floating of shale particles to the surface.

The slip form paver was followed by the finishing cart equipped with an astro grass drag to produce a rough driving surface. The finishing cart also carried the angle iron template which cut the fresh concrete for inserting the premolded strip. The cure cart followed the finishing cart and sprayed white membrane curing compound.

In a good days operation the contractor could achieve up to 1.6 kilometers (1.0 miles).

Results

We now have 56.3 kilometers (35 miles) of concrete pavement in place on rural roads constructed from 1975 through 1978. The riding quality of the slab is good to excellent. The transverse joint spacing has controlled the cracking and we have very little random cracking other than what occurred during construction. The joints formed with the premolded strips are being watched and if spalling occurs they will be sawed and resealed.

We have not had a popout problem due to the shale content. We attribute this to the light use of the vibrators and not floating the shale particles to the surface.

The 6.7 m (22 foot) wide slab seemed narrow at first, but we now feel this may have some merit in keeping the speed down and the drivers more alert. As of this date we have not had a head on collision or other accident which could be attributed to the slab being too narrow. The 0.9 m (3 foot) unpaved shoulder provides additional width in case of emergencies and adds to the safety of the road.

The maintenance costs to date have been only for snow plowing and shoulder repair.

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Richard Komstad, Superintendent, Davis Construction
Kenneth Frigstad, Vice-Pres, NoDak Contracting Corp.
Neil W. Chauncey, Asst. Area Road Engineer, BIA
Sherman E. Oland, Highway Engineer, BIA

PORTLAND CEMENT CONCRETE OVERLAYS OF EXISTING ASPHALTIC CONCRETE SECONDARY ROADS IN IOWA

Carl F. Schnoor, P.E., Boone County, Iowa Engineer
E.J.Renier, P.E., Senior Regional Engineer-Paving, Portland Cement Association, Minneapolis, Minnesota

Forty-two kilometers (22 mi.) of existing asphaltic concrete low-volume roads were resurfaced with portland cement concrete in five counties of Iowa during 1977. In two counties, complete removal of the old asphalt surface was required prior to repaving with portland cement concrete. In the other three, the old asphalt surface was retained as a base for the new pavement. This paper discusses procedures developed to establish and control grade and portland cement concrete overlay thicknesses in the cases where the old asphalt was retained. On one project grade was established and minimum thickness retained by use of a computer. Economics of design and construction procedures were determined by county engineers. Projects were approved by the Iowa Department of Transportation prior to construction. Thickness monitoring and required equipment modification was accomplished by contractor development and cooperation. The resulting pavements show that portland cement concrete overlays can be successfully constructed over existing asphaltic concrete roads on low-volume secondary systems with a minimum of surface preparation and can contribute a long-term economical solution to the ever-increasing cost of maintenance.

By September of 1978 county engineers in Iowa had constructed over 6,700 km (4,171 mi.) of concrete pavement on the Secondary Roads System of the State. This construction has taken place throughout the State as shown in Figure 1. Earlier papers (1, 2) have been written regarding the performance and related maintenance costs of this system.

During the past 10 years, several county engineers in Iowa have begun to reanalyze the economics of resurfacing procedures used on their asphalt-paved secondary roads in an attempt to decrease maintenance costs and lengthen the required maintenance cycle. Their analysis has resulted in the construction of portland cement concrete overlays over old asphalt county roads (3, p.15) in a number of counties.

Figure 2 was produced from original data obtained in 1972 by W.G.Bester, Portland Cement Association Paving Engineer in Iowa at that time. It shows the effect that an increase in portland cement concrete mileage had on maintenance costs on the low-volume system in one Iowa County.

In 1977, 34.4 km (22 mi.) of asphalt roadway was resurfaced with portland cement concrete in 5 counties. In general, these asphalt roads were some 20 years or more in age and had been resurfaced or seal-coated one or more times during the interim. This paper directs itself to the techniques used for construction of the 5 overlay projects built in 1977.

Two fundamental construction procedures were employed. County engineers in Clinton and Cedar Counties opted for complete removal of the old asphalt surface prior to resurfacing with portland cement concrete because the existing surface exhibited extensive deterioration and distortion. In both cases, after asphalt removal, the existing rolled-stone base was fine-graded prior to PCC paving. The old asphalt material was salvaged and used to upgrade the shoulders. Following removal of the old asphalt, grading and paving procedures followed Iowa's standard slip-form paving requirements for the County Road System. In both cases, the minimum thickness for the portland cement concrete

Figure 1. Distribution by county of portland cement concrete pavement on the county road system in Iowa.

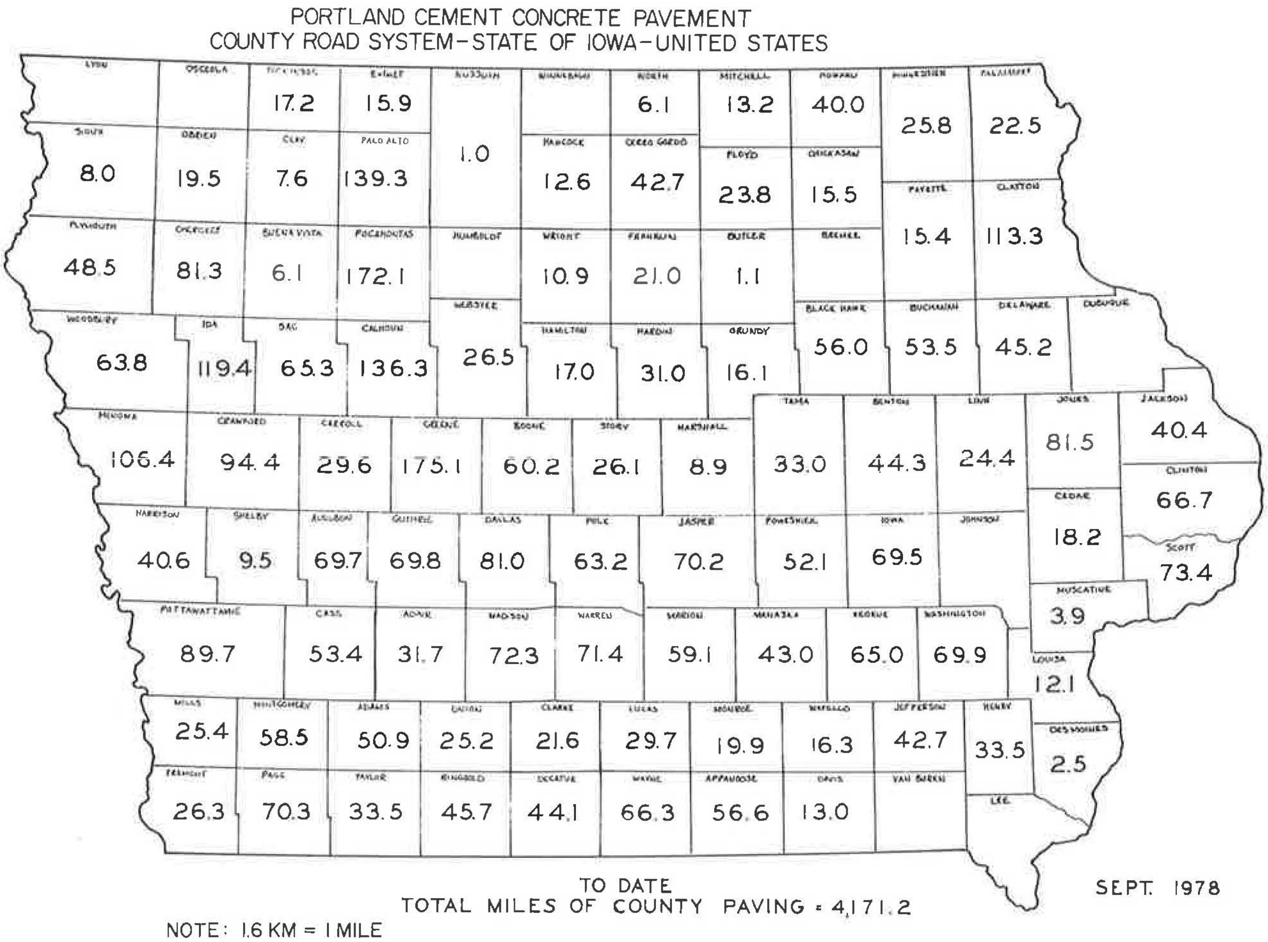
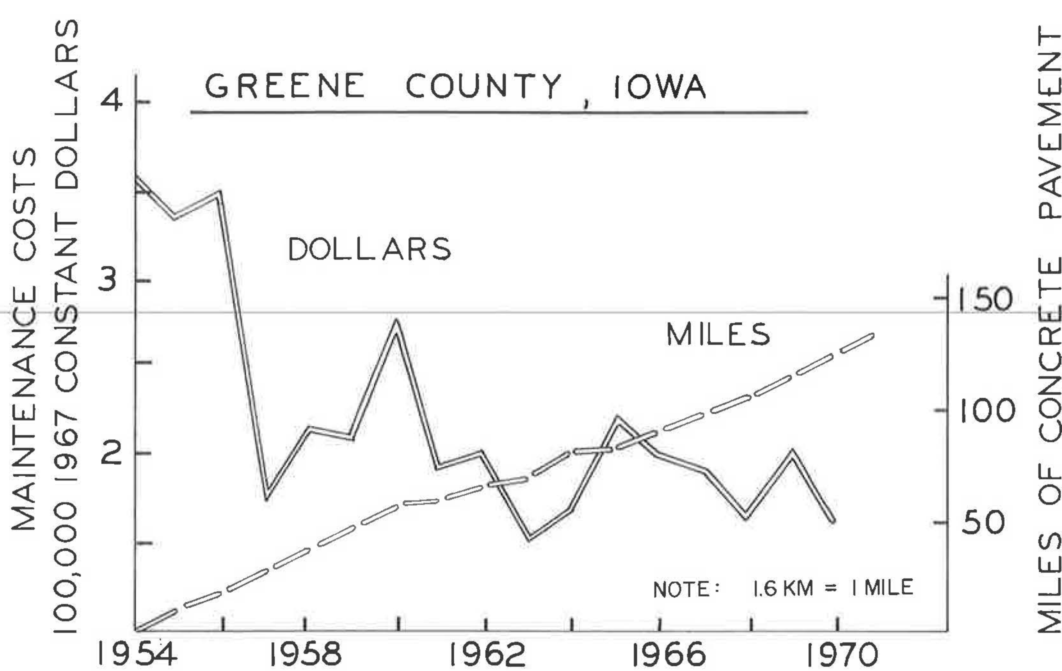


Figure 2. A reduction in maintenance expenditures occurred as the miles of portland cement concrete increased in Greene County, Iowa.



pavement was specified at 152 mm (6 in.).

In Dallas County, Washington County, and Boone County, the county engineers retained the existing asphalt surface as a base under the new portland cement concrete pavement. In all cases, the resulting concrete paving has been excellent. Resurfacing with portland cement concrete directly over the old asphalt required different construction techniques in each case - with each county engineer determining the best procedure to follow for the overlay construction.

Dallas County

In May of 1977, a 1.6 km (1 mi.) project in Dallas County was awarded to the Hallett Construction Co. for the resurfacing of a 76 mm (3 in.) asphaltic concrete mat, which had been placed over 203 mm (8 in.) of rolled-stone base in the mid-1950's. The original pavement had been built 6.7 m (22 ft.) wide and geometric standards at that time allowed the use of narrow shoulders. Because of the narrow shoulders, the contractor was required to widen them prior to the construction of a pad-line for the slip-form paver. The general condition of the asphaltic concrete pavement is shown in Figure 3.

Figure 3. Close-up of the surface,showing general condition of the roadway at the time the overlay was placed.

The roadway to be resurfaced with portland cement concrete had an extremely high crown. Approximately 25.4 mm (1 in.) of this crown was removed with a Gallion road planer as shown in Figure 4.

The Gallion planer cut to a width of 762 mm (30 in.) at centerline and an additional pass was then made on each side of the centerline cut to feather the surface into the existing profile. Grade pins were then set at 15.2 m (50 ft.) intervals on centerline and grade established by stringline. This stringline controlled a CMI tripod auto grader, which in turn established the pad-line for the slip-form paver on

Figure 4. A Gallion road planer was used to reduce the crown 2.54 cm (1 in.).

each side of the road.

Specifications required the placement of a nominal depth - 152 mm (6 in.) portland cement concrete resurfacing. In addition, it was required that a minimum thickness of 127 mm (5 in.) and a maximum thickness of 178 mm (7 in.) be retained.

At bridges and intersections, approximately 30.5 m (100 ft.) of the old pavement was completely removed and the new grade established by stringline to meet existing bridge decks or intersecting pavement sections.

The concrete for this project was produced in a central-mix plant and transported to the job-site in dump trucks. It was then deposited directly on the asphaltic concrete surface. The slip-form paving operation was no different than that used to produce the 6,700 km (4,171 mi.) of portland cement concrete roads already in existence in the State. No new or sophisticated equipment was added. A 152 mm (6 in.) thick Iowa Department of Transportation Type B mix was used for the plain concrete pavement which was constructed 6.7 m (22 ft.) wide with transverse joints sawed on 6.1 m (20 ft.) centers.

The only steel used in the project were 762 mm (30 in.) long 13 mm (1/2 in.) diameter deformed tie-bars, which were placed at 0.9 m (3 ft.) centers at the centerline longitudinal joint. This project was used as a detour during construction of a nearby state highway and an inspection made following this additional loading showed it to be in excellent condition. The successful bid on this project was submitted by the Hallett Construction Co. at \$6.42 per m^2 (\$5.37 per sq.yd.).

Washington County

In August of 1977, Washington County, Iowa resurfaced 14.5 km (9 mi.) of existing asphaltic concrete pavement with a nominal 165 mm (6.5 in.) thick portland cement concrete overlay. The original pavement was built in 1958 and consisted of 102 mm (4 in.) of sand base topped with 152 mm (6 in.) of rolled-stone base and 64 mm (2½ in.) of Type B Iowa specification "Asphaltic Concrete Surfacing".

In designing the new grade, the county engineer cross-sectioned the existing roadway every 30.5 m (100 ft.)and entered the data into a 9830 Hewlett Packard computer. Minimum thicknesses of 152 mm (6 in.) at

Figure 5. The computer printout provided centerline elevation from which required grade was established every 30.5 m (100 ft.). It also computed cross-section area every 30.5 m (100 ft.) and totalled required project quantities.

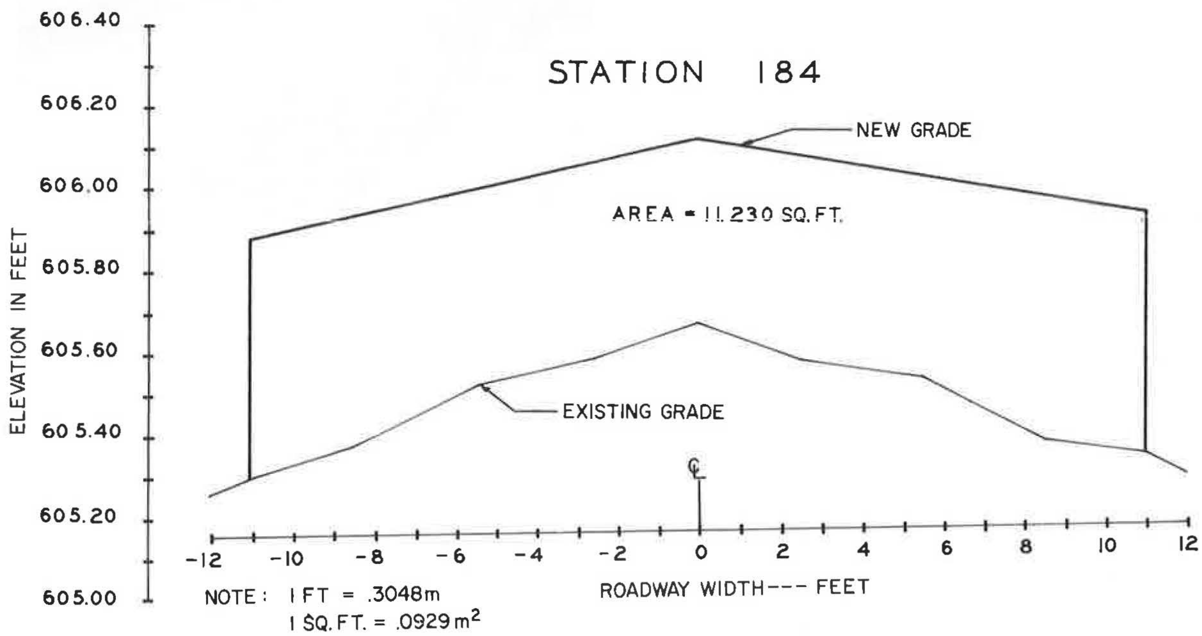


Figure 6. Depicts the entire pavement train including the process of concrete delivery. Note stringline placement.

the outside edge of the roadway and 127 mm (5 in.) at the centerline were required. The existing asphaltic concrete pavement was 7.3 m (24 ft.) in width. The new portland cement concrete overlay was constructed at 6.7 m (22 ft.) wide. With these design restrictions, the average section was built 152 mm (6.5 in.) thick at the outside edge and had a 64 mm (2½ in.)crown.

The computer then plotted the design cross-section every 30.5 m (100 ft.) and established the final grade from which the slip-form paver would operate. A sample computer plot is shown in Figure 5.

The Fred Carlson Co. of Decorah, Iowa was low bidder on the project at \$6.60 per m^2 (\$5.52 per sq.yd.).

The first construction procedure consisted of brooming the existing surface and establishing the computer-determined final grade by stringline. The stringline was established on the right-hand side of the roadway in the direction of paving. After the stringline was in place concrete was delivered to a Rex slip-form paver from a central-mix plant by both dump trucks and agitator trucks and was deposited directly on the old surface in front of the paver. The only piece of slip-form equipment required was the paver itself. No fine-grading was necessary. Figure 6 shows the paving train.

The low-slump portland cement concrete did not slide on the existing asphalt pavement in front of the paver and tearing did not occur in the new pavement surface behind the paver. An inspector operating from a bridge towed by the paver checked pavement thickness at each edge and at the centerline of the plastic portland cement concrete overlay a minimum of once per station. A depth probe, consisting of a slender steel rod operating in a hollow steel tube with a washer-type device to hold the tube at the surface of the slab, was used to check depth. See Figure 7.

Figure 7. A depth probe device was used for measuring the thickness of the resurfacing. The washer-type device is shown at the surface of the plastic concrete with the steel rod inserted into the concrete.

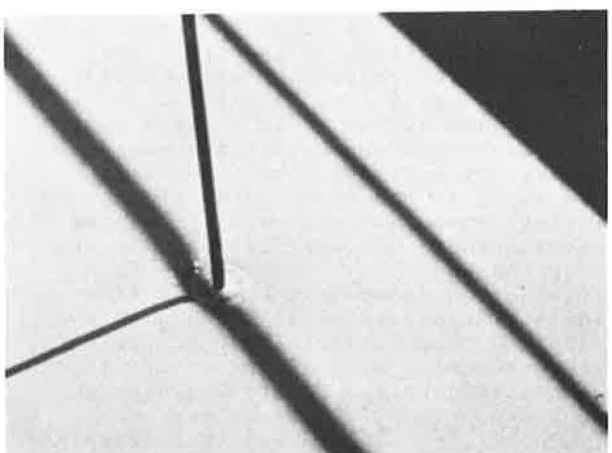
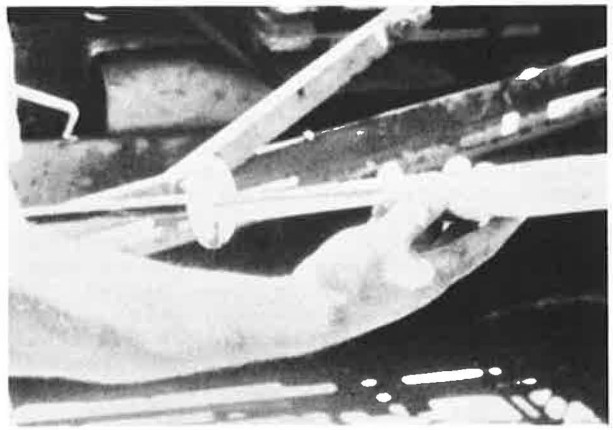


Figure 8. The inspector measured the depth of penetration on the probe with a ruler.



The portion of the rod that penetrated into the plastic concrete was then very easily checked for length as shown in Figure 8 to assure that proper thickness was being obtained. This simple, but accurate device, provided the contractor with continual information regarding overlay thickness while the concrete was still in a plastic state.

The Rex paver towed an astro-turf drag which produced the microtexture in the new surface. The paver also towed a combination bridge from which a transverse combed texture was applied following the astro-turf drag. This last piece of equipment also incorporated the curing operation. Curing was accomplished by application of white membrane curing compound. This operation is shown in Figure 9.

Figure 9. Curing compound was also applied at the edge of the slab. The two pieces of equipment -- Rex slip-form paver, plus combination bridge and equipment -- provided combing and curing.

The new pavement was opened to local farm traffic as soon as flexural test beam breaks indicated that specified strengths had been reached. Both transverse and longitudinal joints were sawed. Deformed 13 mm (1/2 in.) diameter tie-bars 762 mm (30 in.) long were placed at .9 m (3 ft.) centers across the centerline. Very little edge slump occurred as the water content was kept to a minimum in the mix design in recognition of the fact that none of the internal concrete water would be absorbed by the grade.

The completed portland cement concrete pavement was produced in a minimum of time -- over 1,737 m (5,700 ft.) was placed the day following a down-pour, which brought all other construction in the area to a complete halt -- as the procedures used minimized handwork and finishing problems. The resulting pavement had a present serviceability index in excess of 4.5. Crushed rock shoulders were added by county crews prior to opening the roadway to through-traffic.

Figure 10. General condition of the existing asphaltic concrete pavement at the time of overlay with portland cement concrete.

Boone County

In Boone County, Iowa many miles of asphaltic concrete pavement, about 20 years of age, were structurally failing and in need of more than a "cover up" in order to provide additional satisfactory service. The county engineer and Iowa Department of Transportation design personnel determined that a minimum of 127 mm (5 in.) of asphaltic concrete was needed as an overlay to the existing asphalt pavement to provide sufficient additional strength for continued use.

Accordingly, in 1976, Boone County awarded a resurfacing contract requiring the placement of a 127 mm (5 in.) asphaltic concrete overlay over a portion of this mileage. The resulting cost in 1976 was approximately $62,000/1.6 km (1 mi.).

Experience had shown that in Boone County this type of construction would require a seal coat in 5 to 7 years and an additional plant-mixed asphalt overlay of 25.4 mm (1 in.) to 51 mm (2 in.) at approximately 10 years. Therefore, prior to awarding additional contracts for the 127 mm (5 in.) asphaltic overlay design the county engineer conducted an economic analysis. He found that during the period 1976-77, asphaltic concrete surfacing had increased approximately 10% in cost, which brought his estimate for future 127 mm (5 in.) asphaltic overlays to $68,200/1.6 km (1 mi.). The same estimate showed that a portland cement concrete overlay could be placed over the existing asphaltic concrete for less and at the same time greatly reduce expensive future maintenance; thereby, reducing the annual cost of the highway to the taxpayer. Accordingly, in July of 1977, bids were taken for a portland cement concrete design and award made to the Hallett Construction Co. of Iowa who submitted the low bid of $5.13 per .866 m^2 (1 sq.yd.) or $66,212/1.6 km (1 mi.) for 6.4 km (4 mi.) of 152 mm (6 in.) nominal thickness resurfacing of the existing asphaltic concrete pavement with portland cement concrete.

The existing asphaltic concrete pavement was constructed in 1957 and consisted of a 102 mm (4 in.) thick soil aggregate subbase over which a 115 mm (4.5 in.) thick bituminous-treated aggregate base and a 64 mm (2½ in.) thick bituminous concrete surface had been placed. In general, the existing asphaltic concrete pavement was failing structurally. Its' general condition is shown in Figure 10.

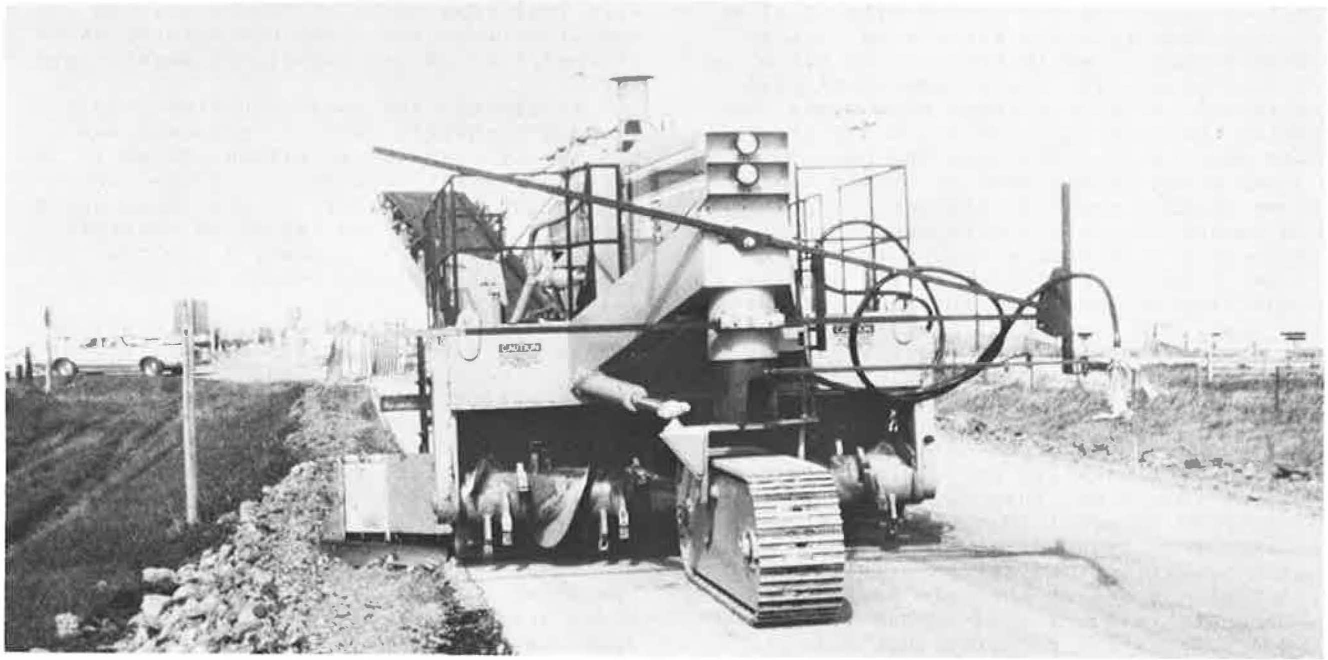
The original pavement had been built 6.7 m (22 ft.) wide and the new portland cement concrete overlay was designed 6.7 m (22 ft.) wide. The crown of the old pavement varied from 76 mm (3 in.) to 115 mm (4.5 in.).

As in-field examination of the old pavement was conducted it became apparent that the profile had deformed less at the centerline than in any other area, and therefore, a decision was made to establish grade control on this project at the centerline of the existing pavement. Specifications required that a nominal thickness of 152 mm (6 in.) and a minimum thickness of 127 mm (5 in.) of portland cement concrete be maintained. In addition, the portland cement concrete overlay would have a maximum 70.4 mm (2-3/4 in.) crown and a minimum 127 mm (5 in.) depth at the centerline. This meant that the greatest thickness of portland cement concrete would occur in the wheel paths of the old pavement; thus, automatically providing greater structural capacity where it was needed the most.

Using the above criteria, the final grade elevation was established with a stringline .6 m (2 ft.) above the existing centerline grade by the contractor. It was then adjusted by the county engineer to remove any obvious discrepancies.

Specifications required that grade for pad-lines for the slip-form paver be electronically established by transferring

Figure 11. Subgrader is transferring the grade from centerline stringline electronically to a cutting bar attached to the wing of the subgrader, which cuts the slip-form paver pad-line at .755 m (2 ft., 5-3/4 in.) below the centerline stringline elevation.



the final profile grade from the centerline stringline to the pad-lines. The contractor accomplished this by use of a half-width CMI sub-grader as shown in Figure 11.

To assure that minimum slab thicknesses as established by specifications were met the contractor constructed a scratch templet, which took its grade from the freshly-cut pad-line. This templet had spring-loaded markers which touched the existing surface at any location where the portland cement concrete would be less in thickness than the minimum required 127 mm (5 in.) depth. Any spot where the templet touched was marked with paint, as shown in Figure 12, and these high spots were then ground to required elevation by the use of a Gallion road grinder.

Concrete for the project was produced in an 6.1 m^3 (8 cu.yd.) central-mix plant on-site. The concrete used met Iowa Department of Transportation B-4 mix requirements and consisted of 233.6 kg (493 lb.) of cement per .765 m^3 (1 cu.yd.) and equal parts of coarse aggregate and sand. Concrete was transported to the site in dump trucks and deposited directly on the old pavement in front of the slip-form paver. Once the concrete was placed in front of the slip-form paver, the remainder of the paving operation was similar to all slip-form projects and the finished pavement was similar to the other portland cement concrete pavements in the County. This project included a bid price for a 152 mm (6 in.) thick nominal slab on a .836 m^2 (sq.yd.) basis and also a .765 m^3 (cu.yd.) bid price based on the

Figure 12. A contractor-constructed spring-loaded templet was used to detect high spots in the pavement surface which would require additional grinding.

theoretical 152 mm (6 in.) nominal thickness.

As in the other projects, the portland cement concrete overlay thickness was constantly checked with a depth probe to assure that minimum thicknesses were obtained. 762 mm (30 in.) long, 13 mm (1/2 in.) diameter deformed bars were placed transverse to the centerline on

0.9 m (3 ft.) centers.

The transverse joints on this project were sawed at different spacing on each mile for the purpose of research. In the first mile, the joints were sawn 12.2 m (40 ft.) apart; in the second mile, 9.14 m (30 ft.) apart; in the third mile, 7.6 m (25 ft.) apart, and in the fourth, 6.1 m (20 ft.) apart. The coarse aggregate used was gravel. On-site surveys made since completion indicate that the 6.1 m (20 ft.) joint spacing is performing the best.

The Boone County project showed that a 152 mm (6 in.) nominal thickness of portland cement concrete overlay could be placed at a cost that was less than a 127 mm (5 in.) asphaltic concrete overlay. In addition to the lower initial construction cost, the history of portland cement concrete pavement on the Secondary System in Iowa indicates a reduced maintenance expenditure per mile as the concrete mileage has increased.

It should further be said that the above projects are all performing excellently at this time. They have all been constructed to specification and meet Iowa Department of Transportation's "Supplemental Specifications for Resurfacing with Portland Cement Concrete Over Asphaltic Concrete Pavement", dated May 10,1977. This specification requires that all portland cement concrete be built with slip-form placing equipment and that the path area over which the slip-form paving machine travels must be constructed to line and grade that provides for placement of the designed thickness of pavement by an electronically-controlled machine. The authors suggest that consideration be given to allowing an alternate method of control, which would allow electronic sensing devices to control slab thickness without construction of pad-lines.

Development of procedures whereby the crown would be allowed to vary could produce a more uniform thickness of slab and should result in material savings. Crown for the new portland cement concrete overlay is specified on the plans and the contractor is required to clean any loose or foreign materials from the existing asphaltic concrete surface prior to the placement of the new portland cement concrete overlay. Where it is necessary to completely remove the existing asphaltic concrete due to poor subgrade support, consideration should be given to thickening the portland cement concrete overlay section through that area. On the Boone County project the portland cement concrete overlay was thickened a uniform 64 mm (2½ in.) through such areas and a sawed transverse joint was placed at the transition point. The simple, but very practical depth probe discussed earlier was used on all projects and assures that the required minimum thickness is obtained.

It is advisable that cut and fill stakes be placed on each side of the roadway at 15.2 m (50 ft.) intervals so that centerline grade can be very simply reestablished if lost. It is required that any subgrade over which the new portland cement concrete overlay will be placed, which is not asphaltic pavement, must be uniformly moistened prior to concrete placement.

All of the portland cement concrete overlays used concrete mixed in accordance with Iowa Department of Transportation specifications for Class B concrete, which allowed full use of locally available aggregates.

In Clinton and Cedar Counties, where the old asphaltic concrete pavement was completely removed, a uniform 152 mm (6 in.) portland cement concrete thickness was required for the overlay. In the three cases -- where the existing asphaltic concrete roadway was used as a base, a nominal thickness of portland cement concrete was specified.

The use of the old asphaltic concrete roadway as a base provides several advantages to the contractor. Bad weather has little effect on construction and paving can start again immediately after an extensive rainfall. In addition, the old asphaltic concrete roadway provided an excellent haul road for materials and a supply route to the slip-form paver. No rutting was encountered ahead of the paver.

In all cases, close control of the amount of water in the portland cement concrete mixes resulted in little or no edge slump to the new portland cement concrete overlay.

The authors feel that the projects reviewed in this paper have developed very practical and usable designs and construction techniques that can be applied in any case where a portland cement concrete overlay is specified over an old asphaltic concrete pavement in order to prolong its life with a minimum of future maintenance.

As additional projects of this type are undertaken, more sophisticated procedures for establishing the grade for the portland cement concrete overlay from the old grade will no doubt be developed. It is suggested that a ski arrangement, which electronically senses grade at the centerline and both edges of the old pavement, be considered.

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OPTIMAL TIMING FOR PAVING LOW-VOLUME GRAVEL ROADS

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This paper examines the economics of upgrading low volume gravel roads with particular emphasis upon construction postponement. The concept of break-even analysis is re-examined and a case presented for consideration of construction deferment in light of the opportunity cost of capital. This consideration is particularly important for developing countries where capital is scarce and the opportunity cost high. Simplified expressions are developed to determine both the break-even year and the optimal year in which to pave a given gravel road. Their application is illustrated by means of a numerical example.

Gravel surfaced roads are generally adequate for most situations of low volume traffic. However, as traffic increases, the maintenance and vehicle operating costs also increase, making it necessary to consider paving the gravel surface. The economic viability of such an action is easily established whenever the reduction in maintenance and vehicle operating costs is substantial in comparison to the construction cost. The concept of break-even analysis (1,2,3) has often been applied in such cases to determine the cut-off volume above which it is economically feasible to pave the road. This cut-off volume, generally referred to as the break-even volume represents the minimum volume above which the net present value of paving is in excess of zero.

If, on the other hand, there are alternative opportunities competing for the same capital, then the break-even criterion is clearly not the most efficient way to allocate scarce resources. Under such circumstances, it is economically more advantageous to postpone construction beyond the break-even volume until such time when the net present value is maximized. If the opportunity cost of capital is at least as high as the discount rate, and the annual net benefits increase monotonically with time, then the net present value is maximized by paving the road in the year in which the first year benefits are equal to the opportunity cost of capital invested. The volume of traffic in this year has been referred to as the "optimal volume" and the concept as the "first-year-benefits" criterion (4).

Therefore, in most instances, when the base year volume is low but increasing with time, it is possible to invest the capital elsewhere in the economy to obtain returns that are in excess of the benefits foregone during the years that construction is deferred. To pave the road a year before the optimal volume would mean foregoing excess benefits that could be obtained from an alternative investment in the economy, while paving the road a year after, would mean losing excess benefits that could have accrued had the road been paved a year earlier. This situation is illustrated diagrammatically in Figure 1. The top portion of the figure shows the stream of annual net benefits arising from paving the road in the year, say n^1. The net benefits of paving the road in any other year would be the same as shown, except for the years before paving when the net benefits are zero. For example, the net benefits corresponding to construction in the break-even year (n_{be}) and the optimal year (n_{opt}) would correspond to the lines $on_{be}pq$ and $on_{opt}q$, respectively. The lower portion of the figure shows the net present value of the entire project, given the year of construction. The points n_{be} and n_{opt} correspond to zero and maximum net present values, respectively.

Design Life, Costs and Benefits

Design Life

In order to make a valid economic assessment of a project, it is necessary to know the time horizon over which the evaluation is to be made. For most road projects, this has been the design life of the project, taken to vary from 15 to 40 years. However, while it is true that a newly constructed facility will have a fixed physical life, it is conceivable, and often the case, that major rehabilitation will be done at the end of this period and in subsequent periods to perpetuate the useful life of that facility. For road projects, this is often done in the form of major reconstruction and overlays, whenever the serviceability of the pavement has fallen below an acceptable level. Under this assumption, benefits may be assumed to accrue indefinitely, for all practical purposes. The physical life of a pavement is a function of the design standard and the traffic volume that it is subjected to. In this paper, no attempt is made to relate these factors, but instead, the frequency of major rehabilitations is assumed to be planned in advance.

Costs and Benefits

The primary costs associated with road projects consist of the construction, routine maintenance, planned rehabilitation and vehicle operating costs.

Construction costs are assumed to occur only in the first year of construction while routine maintenance costs at the end of each year. Maintenance costs are assumed to remain constant over time. Major rehabilitation costs are planned to occur every N years, where N is presumably close to the physical life of the pavement.

Vehicle operating costs are a function of the vehicle types and their relative mix in the traffic. Therefore, a weighted average unit cost per vehicle-mile will be used, derived as follows:

$$c = \sum_{i=1}^{k} c_i \, p_i \qquad (1)$$

where,

- c = average cost per veh-km (0.6 veh-mile), for all vehicle types,
- c_i = unit operating cost of the i^{th} vehicle type, per km (0.6 mile),
- p_i = fraction of total traffic that is of type i,
- k = number of different vehicle types in the traffic mix.

The weighted average cost will be assumed to remain constant over time, but have different values on paved and gravel surfaces, respectively. It follows, therefore, that the annual growth rate of traffic must be the same for all vehicle types.

The benefits from paving a gravel road comprise largely of the reduction in the total vehicle operating and routine maintenance costs. Net benefits are then obtained after allowing for the construction and rehabilitation costs.

As in all other economic evaluations of this nature, the stream of costs and benefits must be adjusted to reflect the temporal value of money, by applying an appropriate discount rate. The choice of an appropriate discount rate is a continuing topic of debate (5,6,7). For our purpose, the opportunity cost of capital will be used as the discount rate also.

Analytical Framework

Definition of Variables

Let the variables relevant to a gravel road being considered for paving be defined as follows:

- Q_0 : volume of traffic in the base year, in vehicles per day
- m, m': uniform equivalent annual routine maintenance costs per km, before and after paving, respectively
- c, c': weighted average operating costs per veh-km on gravel and paved surfaces, respectively
- C: fixed construction costs per km of paving
- N: frequency of major rehabilitation, in years
- R: cost of rehabilitation, in dollars per km
- i: opportunity cost of capital
- r: traffic growth rate per annum (generally less than i)

Let n denote the year in which the gravel road is paved. The present value (P.V.) of various cost elements per km of roadway, are obtained as follows-keeping in mind that the road is kept in service indefinitely through periodic rehabilitation:

P.V. of construction costs

$$= \frac{1}{(1+i)^n} \, C \qquad (2)$$

P.V. of rehabilitation costs

$$= \frac{1}{(1+i)^n} \cdot \frac{R}{(1+i)^N-1} \qquad (3)$$

P.V. of savings in routine maintenance costs

$$= \frac{1}{(1+i)^n} \cdot \frac{(m-m')}{i} \qquad (4)$$

P.V. of savings in vehicle operating costs

$$= \frac{1}{(1+i)^n} \cdot Q_n \cdot 365\,(c-c') \cdot \frac{1+r}{i-r}\,; \quad r<i \qquad (5)$$

where,

$Q_n = Q_0\,(1+r)^n$ = volume of traffic in year n

$\frac{1+r}{i-r}$ = the discount factor for present value of a geometric series with growth rate r and discount rate i, over an indefinite period (for r<i).

The net present value (NPV) of paving is then obtained as:

$$NPV = \frac{1}{(1+i)^n}\left[Q_n \cdot 365\,(c-c')\,\frac{1+r}{i-r} + \frac{(m-m')}{i} - C - \frac{R}{(1+i)^N-1}\right] \qquad (6)$$

Break-Even Analysis

If we now define n as the break-even year, then the break-even volume, Q_{be}, may be obtained by setting the NPV equal to zero in equation (6). Hence,

$$Q_{be} \cdot 365\,(c-c')\,\frac{1+r}{i-r} + \frac{(m-m')}{i} = C + \frac{R}{(1+i)^N-1}$$

$$Q_{be} = \frac{C + \frac{R}{(1+i)^N-1} - \frac{(m-m')}{i}}{365\,(c-c')\,\frac{1+r}{i-r}} \qquad (7)$$

Provided the base year volume, Q_0, is less than Q_{be}, the break-even year, n_{be}, may then be obtained from,

$$Q_{be} = Q_0 \ (1+r)^{n_{be}}$$

$$n_{be} = \frac{Log_e(Q_{be}/Q_0)}{Log_e \ (1+r)} \quad \text{for } Q_0 < Q_{be}, \text{ otherwise zero} \tag{8}$$

Optimal Year of Paving

The optimal year for paving the gravel road is obtained by maximizing equation (6) with respect to n. However, as seen in Figure 1, with the discount rate equal to the opportunity cost of capital, the net present value is maximum when the undiscounted net benefits in the year of paving are equal to zero. For such a maximum to exist, it is necessary that the net benefits increase monotonically with time. This is ensured as long as the savings in the vehicle operating costs and the traffic growth rate are both non-negative.

The net benefits in the first year after paving are given as:

$$Q_n \cdot 365 \ (c-c') + (m-m') - Ci - \frac{Ri}{(1+i)^N-1} \tag{9}$$

To obtain the optimal volume (Q_{opt}), we substitute Q_n by Q_{opt} in the above expression and set it equal to zero.

$$Q_{opt} \cdot 365(c-c') + (m-m') - Ci - \frac{Ri}{(1+i)^N-1} = 0$$

from which,

$$Q_{opt} = \frac{C_i + \frac{Ri}{(1+i)^N-1} - (m-m')}{365 \ (c-c')} \tag{10}$$

Again, if Q_0 is less than Q_{opt}, we obtain the optimal year of paving, n_{opt}, as:

$$n_{opt} = \frac{Log_e \ (Q_{opt}/Q_0)}{Log_e \ (1+r)} \quad \text{for } Q_0 < Q_{opt}, \text{ otherwise zero} \tag{11}$$

The break-even and the optimal years obtained from equations (8) and (11) are rounded to the nearest whole numbers, in line with the usual assumption of year-end cost outlays.

Numerical Example

In this section a numerical example is considered to illustrate the procedure. Although the following values are realistic, any different set of data would indicate the same direction in results as shown by this example.

Q_0 = variable
m-m' = $500 per km
c-c' = $.022 per veh-km
C = $46,500 per km
N = variable
R = $25,000 per km
i = 10 percent per annum
r = 6 percent per annum

Equations (7) and (10) are used first to determine the break-even and the optimal volumes, respectively. These are shown in Table 1 for three different values of N, the frequency of periodic rehabilitation. With base-year volume known, the break-even and the optimal years for paving the road are computed using equations (8) and (11). Table 2 shows these values for base-year volumes ranging from 150 to 400 vehicles per day. The year of paving is taken as zero whenever the base-year volume is in excess of the break-even and optimal volumes, respectively.

It is clear from these results that the difference in the break-even and the optimal years can be large depending upon the base-year volume. This difference reflects the period during which the economy can benefit more from investing the capital elsewhere. Table 3 shows the net benefit of construction deferment computed as the difference in the net present values of paving the road in the years suggested by the optimal and the break-even criteria. As indicated in Table 3 the benefits tend to decline as the base year volume increases. In those cases where the base year volume is much higher than the optimal volume, any delay in paving will lead only to a decrease in the net present value. However, the type of highway considered in this paper is low-volume gravel roads where the base year volume will not generally exceed 400-500 vehicles per day.

Conclusions

This paper has presented an approach to determine optimal timing for paving low-volume gravel roads based on explicit consideration of the opportunity cost of capital. As there are many competing uses for capital at any given time, selecting one use of capital implies the cost of foregoing the opportunity to earn a return with it elsewhere. This situation is particularly important for developing countries where capital is scarce and the opportunity cost is high.

This paper has indicated that the often used break-even criterion is not a desirable approach in determining the year of paving, and that it is economically advantageous to postpone construction beyond the break-even volume until such time when the net present value is maximized. The net present value is maximum when the undiscounted net benefits in the year of paving is equal to zero. By ignoring the effect of the cost and timing of periodic rehabilitation, the optimal time of paving can be approximated by the "First Year Benefit Rule" which states that the time to pave is when the benefits during the first year (savings in maintenance and vehicle operating costs) expressed as a percentage of the construction cost is greater than the opportunity cost of capital. The results of a numerical example considered in this paper indicated that a considerable net savings can be realized by defering paving beyond the break-even volume, particularly in those cases where the base year daily volume is relatively low.

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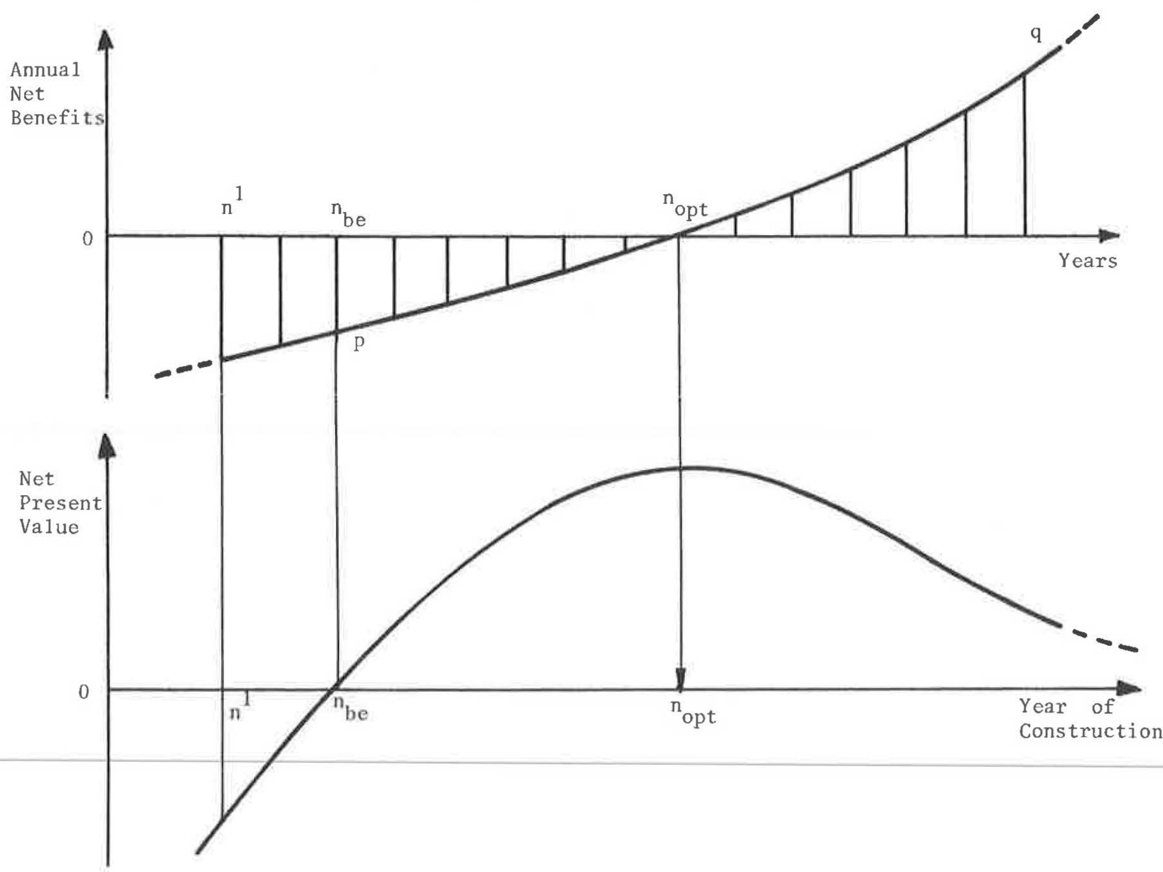

Figure 1. Net Present Value as a Function of the Year of Construction

Table 1. Break-even and optimal volumes (vehicles per day).

Rehabilitation Frequency (N) in Years	10	15	20
Break-Even Volume (Q_{be})	269	232	216
Optimal Volume (Q_{opt})	712	615	571

Table 2. Break-even and optimal year of paving

Base-Year Volume (Q_0)	Rehabilitation Frequency (N) in years 10	15	20
150	10/27*	8/24	6/23
200	5/22	3/19	1/18
300	0/15**	0/12	0/11
400	0/10	0/7	0/6

*Break-even/Optimal year of paving.
**Where base-year volume is in excess of the break-even volume, the break-even year is negative hence taken to be zero.

Table 3. Present value of construction deferment to the optimal year (dollars per km of road)

Base-Year Volume (Q_0)	Rehabilitation Frequency (N) in years 10	15	20
150	7,195	8,274	8,447
200	11,587	13,326	13,605
300	15,928	11,499	8,538
400	8,434	6,073	3,444

A PROGRAM OF BRIDGE INVENTORY, INSPECTION AND RATING FOR A LOCAL ROAD SYSTEM

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This paper is an analytical description of the process developed by the State of Illinois, Department of Transportation, in cooperation with local highway authorities, to not only assure compliance with the Federal-Aid Highway Acts but to formulate a plan to collect accurate data on all bridges with a clear span of 20'-0" or more on local systems in order to demonstrate structural condition and needs. The program coordinates, computerizes and stores, with easy retrieval, the information from efforts of Local and State authorities with a common goal to clarify the highway and structure demand for modern transportation arteries. Positive identification and documentation of obsolescence and deterioration of local bridges has resulted in a monetary response from the legislature which is progressively making shorter and safer routes for today's traffic volume and weights.

Foreword

Under the terms of the Federal Aid Highway Act of 1968, Illinois, like the other 49 states, is charged with the responsibility of inventorying and rating all structures with spans more than 20' on its Federal-aid highway systems. The 1978 Surface Transportation Assistance Act expands this to include off-system bridges also.

In compliance with the requirements of the Acts, and with respect to the pending need for more detailed information on structures in the State of Illinois, the Illinois Department of Transportation undertook to prepare a program designed to inventory, inspect, and rate all structures in the State with spans of more than 20', regardless of the categorical designation of the system on which they were located. Illinois undertook the broader scoped program over and above that required by the earlier Federal legislation because of the obvious need in the State for more complete and up-to-date data on all its structures.

Illinois has approximately 25,000 structures on its highway systems, of which some 15,000 are located on the local highway systems and are under the jurisdiction of local highway authorities. Illinois has 102 counties, 1,269 municipalities, and 1,476 road districts, all of which have jurisdiction over some roads and highways. Also, private concerns have jurisdiction over some structures.

Within the State organization, the Department of Transportation has jurisdiction over the structures on the State and Federal-aid primary, Federal-aid Interstate and some Federal-aid Urban highway systems. Other governmental agencies, such as the Department of Conservation, have jurisdiction over highways and structures located within conservation areas and State Parks.

Development of Inventory and Rating System

To make a program of this magnitude successful, it was necessary to effect a cooperative effort between all local and State highway authorities having jurisdiction over bridges within the state.

In view of the many agencies involved, it was obvious from the start of the program that success was dependent upon coordination. In an effort to achieve that coordination toward a common goal, the project was undertaken with cooperative planning between the state organizations and the various policy committees representing the local highway agencies. A task force was established within the Department of Transportation to establish recommended policies and procedures to implement the program. This task force was made up of representatives of several Department subdivisions and the Bureaus of Planning, Maintenance, Design, and Local Roads.

The task force established the policies and procedures necessary to achieve the goals of the program and to evaluate the data currently available from existing road inventory records and documents. The task force also established what additional data was needed to determine the highway needs in the State of Illinois. Coordination was maintained with policy committees composed of County Superintendents of Highways, City Engineers, and Consulting Engineers, representing local highway agencies. This procedure was initiated to assure cooperation from the local highway agencies and to insure that the needs of the local agencies were met.

After initial study, the task force resolved that, since the two programs were parallel, all necessary forms and procedures should be compatible with both the Federal Bridge Inspection Program and the Transportation Needs Study. The Transportation Needs Study is the documentation used by the State Legislature in drafting legislation to meet the highway needs of the State.

The task force was also charged with the responsibility of drafting a program which would provide comprehensive information for use by all state and local agencies in determining the needs and priorities for improvements to the entire highway system within the state. The following objectives were established by the task force:

1. The program should provide information on the statewide, regional, or local basis, detailing the needs and the improvements required on highway systems, and to estimate costs of such improvements for use in drafting possible legislation, and for budgeting purposes for state and local governmental agencies.
2. The program should provide a mechanism to handle the current needs and the projected needs on the various systems in the state, and a dual system for determining priorities for replacement based on existing revenues at all levels in government.
3. The program should provide the capacity to determine the priorities for the improvement or replacement of structures which are insufficient for safe highway travel.
4. The program should develope data which could be updated periodically to provide a "continuing needs" study for state and local governments.
5. The program should provide the mechanisms for establishing safe load-carrying capacities and the posting of these capacities for the safety of the motoring public.

The task force undertook the preparation of a basic program to implement those objectives, beginning with the determination of information required in the Needs Study for inclusion in the Department of Transportation's computer data bank to satisfy both Federal requirements and that study.

A structure numbering system was established on the basis of the road inventory file data available. The system used a seven-digit number. The first three digits identified alphabetically the county in which the structure was located. Consequently, Adams County was assigned county number 001, and the last county in alphabetical order, Woodford County, was assigned county number 102. The remaining four digits of the structure number were utilized in groups to reflect maintenance responsibility of the structure. The four digit numbers from 0001 through 2999 reflected structures maintained by the state, the number series from 3000 through 5999 reflected structures maintained by the counties or townships, 6000 through 9899 reflected city maintenance responsibility, and structures numbered from 9900 through 9999 indicated maintenance responsibilities of other governmental or private agencies. Water Districts and Railroads, for instance, fell into this category. After the maintenance responsibility for the structures was ascertained, the structure numbers were assigned in conjunction with the route upon which they were located, and the log mile of the structure along the route. The numbers were then assigned in sequence in the ranges of numbers denoting proper maintenance responsibility beginning with the lowest number in each range. For example, if a Federal Aid Secondary route had 6 structures located on it and all were the maintenance responsibility of the county, the 6 structure numbers would be in sequence, with the lowest number being located along the lowest log mile (or mile post) on the route within that county. Each local agency having jurisdiction over structures then prepared a number map, accurately locating those structures under its jurisdiction.

Local agencies were given the option to assign bridge numbers, and some counties adopted unique structure numbering systems, adhering to the 3000 to 5999 rule. For example, several counties elected to assign numbers 3000 through 3299 to the structures on the county highway maintenance system. Then each road district within the county was assigned a block of numbers. The first road district could be assigned numbers 3,400 through 3,499. The second road district could be assigned numbers 3,500 through 3,599. This system could be continued to provide a group of numbers for each road district, thus permitting the local agency to identify county or road district maintenance responsibility from the structure number.

Municipalities were asked to establish numbering systems within the 6,000 to 9,899 numerical confines. To avoid duplication, a block of numbers was assigned to each municipality in each county. For example: One municipality could utilize the numbers 6,000 through 6,099 the second municipality could use 6,100 through 6,199, etc. Care was also taken to assign an ample block of numbers to each municipality to allow for future expansion of the system.

Municipalities identified their bridges by number on official city maps provided by the Department. These maps were incorporated into the program.

Other governmental agencies (for example, water districts) or private agencies having maintenance responsibility for structures were asked to assign numbers to their structures in the 9900 to 9999 series, to avoid duplication. These agencies were asked to indicate their structure numbers on a map. These annotated maps are available to local agencies and the general public upon request.

With the use of the structure-numbering system and structure-number maps, it is possible to locate every bridge in the State as defined by AASHTO (20' between spring lines). The assigned structure number will appear on all future name plates. In all cases, assigned numbers shall be painted in a conspicuous place on the bridge.

After establishing the procedure for numbering the structures, the task force began preparing the necessary inventory forms. As a base for the information, the task force started with Plate 14-1 (FHWA structure appraisal sheet, Figure 6) required by the Federal Bridge Inspection Program and supplemented the requirements of that document to cover the additional data needed for use in the Transportation Needs Study. To facilitate the efficient handling of the information to be collected, the task force elected to divide and document the data on two forms "Structure Inventory Sheet" (Figure 1) and "Structure Appraisal Sheet" (Figure 2). The Structure Inventory Sheet contains the first 57 items on Plate 14-1, plus the items added to the inventory to satisfy the Transportation Needs Study, such as items A, B, or C. A fundamental requirement in the designing of the form was that it be readily adaptable to the collection of the data in the field, as well as in a format acceptable for keypunching into the computer.

The Structure Inventory Sheet was designed to identify the structure by number and location; to indicate the variety of information to be collected by item number; description of the item; a new-data column for data collected in the field; and an old-data column showing data obtained from existing records. The description column is utilized by field personnel in the collection of the structure information. The new-data column permits coding of the data by field personnel and is used for keypunching the data into the data bank. The old-data column indicates information currently available. Field personnel are to code the information obtained in the field directly on the structure inventory sheet and verify or correct the old-data information on the sheet to reflect current conditions.

Several of the supplemental items added to the Structure Inventory Sheet provide enough additional data to complete the overall inventory for the structure. For example, Item 36B, construction section, includes sufficient code space for entering the original construction section, thus supplying ready identification and reference to older filing systems.

Item 37 indicates the microfilm number identifying the microfilm roll upon which the original design plans for the structure are located. Other supplemental code items have been added to the form to provide a more complete and usable compilation of data for each structure. Two copies of the Structure Inventory Sheet were generated from the data available in the State's data bank for each structure and were forwarded to the agency responsible for the maintenance of the structure with the necessary instructions for the proper coding of the items.

The second form used to complete the appraisal portion of the bridge program, a "Structure Appraisal Sheet" (Figure 2), incorporated items 58 through 84 of Plate 14-1 (Figure 6) of the Federal Bridge Inspection Program for use by field personnel in making the necessary appraisal codings. The information from the completed form is adaptable to keypunching. An additional item was added to the structure appraisal form as item No. 85; the date of the inspection, to insure record continuity.

The Structure Appraisal Sheet heading indicates the computer number for use in the State's data bank, as well as the seven-digit structure number. The code items and numbers are consistent with Plate 14-1. The form provides a brief description of the item for use by field personnel, plus space to denote a brief written description of materials and conditions encountered in the field. The form also provides a coding bank for the various items for field coding and use in keypunching the data. Coding instruction sheets were prepared for each item included on the Structure Inventory Sheet and Structural Appraisal Sheet. A typical structure coding sheet is shown in (Figure 3). All coding instruction sheets contain complete coding instructions for each item.

In the illustrative typical coding form (Item 43) shown in (Figure 3), the coding sheets are divided into four major sections. The first section is the description of the item to be coded. The second portion in the coding sheet indicates the purpose for which the information is to be used. The third portion of the coding sheet indicates the procedure to be used in obtaining the information to be coded. The fourth portion of the coding sheet denotes the code numbers and their respective meanings.

These coding instruction sheets are also included as guides for use by field personnel in the proper coding of the respective items. The coding instruction sheets were developed from Plate 14-1 and supplemented by additional instructions and directions to incorporate the proper coding required for the Transportation Needs Study.

In order to assist the many agencies in maintaining adequate records of their structures, a Bridge Record Card (Figure 4) was developed for their use. The Card provides for a description of the structure, hydraulic data, posting and inspection data, and information regarding repairs made to the structure which permits the agency to maintain an up-to-date record.

Since the Federal Acts require a bi-annual inspection of all structures on the Federal-aid highway system, a Bridge Inspection Report Form (Figure 5) was developed by local agencies. The form is designed to cover the critical members of the various structure types, with space provided to denote condition and needed maintenance or repairs. The inspection report can be supplemented by drawings, pictures, etc., as the inspector deems necessary.

Rating for Load Carrying Capacity

Two basic procedures were established for determining the Operating and Inventory ratings and determining the safe load capacity of each structure. The AASHTO Manual of Maintenance Inspection of Bridges was the authority for determining the ratings for each structure. The local highway authority has the option of using either of the following procedures.

The first procedure provides for the use of private consulting engineering firms or the use of local agency staffs to compute the necessary ratings and make recommendations for posting the structures. Many local agencies in the State of Illinois have used Federal highway safety funds to aid in the financing of such ratings. Present statutory requirements in the State of Illinois permit only registered structural engineers to make ratings on structures and determine safe load capacities. Therefore, only consulting firms or local agencies that have qualified personnel on staff can follow this procedure.

Another statutory restriction in the State of Illinois is:

The Department upon request from any local authority shall, or upon its own initiative, may conduct an investigation of any bridge or other elevated structure constituting a part of a highway, and if it finds the substructure cannot with safety to itself withstand the weight of vehicles otherwise permitted under the Statutes, the Department shall determine and declare the maximum weight of the vehicle which the structure can withstand, and shall cause or permit suitable signs stating maximum weight to be erected and maintained before each end of such structure."

The statute also provides penalties for violation of such load restrictions. Based on the above statutory restriction, a registered structural engineer working in the private sector who performs ratings and safe load capacity determinations of any structure on the highway system must obtain concurrence from the Department prior to posting such load restrictions on the structure.

The second procedure provides for the Department to determine the Operating and Inventory ratings and to determine the safe load capacity of the structures when requested to do so by the local agencies. This procedure was established because many local agencies have limited funds available for the repair and maintenance of their structures. Therefore, to provide for a cooperative effort between local agency personnel and Department personnel in obtaining the necessary field and plan data for calculating the rating of the structures and making recommendations for posting, Department assistance was deemed necessary. Specifically:

1. The local highway authority initiates a request for the Department to perform the necessary ratings and provide the posting recommendations for structures.
2. The local highway authority is responsible for obtaining copies of the original "as built" plans for the structures, if they are available. If "as built" plans are not available for a structure, the local agency is responsible for obtaining the field measurements necessary for determination of the structural rating by the Department. Photographs are also requested.
3. When the "as built" plans or the necessary

field measurements have been obtained by the local highway authority, the Department will schedule a member of its field inspection team to inspect the local structures with the local agency representative. The field inspector may also take supplementary photographs of the structure showing damage to deteriorated areas or any unique area of the structure which he feels should be clarified to assist the rater in calculating the ratings for the structure. The Department's field inspector will make appropriate notations or recommendations in the field on the "as built" plans or drawings to reflect the current condition of the structure.

4. After the field inspection is completed by the Department's inspector, the plans and related information are used by the Department to determine the ratings and posting recommendations for each structure.

5. The completed ratings, based on the operating rating, (which is 75 percent of the yield strength in accordance with the AASHTO Manual for Maintenance Inspection of Bridges) and the Inventory Rating (which is based on 55 percent of yield strength of the material) are forwarded to the local highway authority with the Department's recommendation for posting of the structure if the structure will not carry maximum legal loads. If it is determined that the structure is capable of carrying maximum legal loads, the Department will recommend to the local agency that no restriction be placed on the structure.

It became apparent from the beginning of the program that many of the structures on our local highway systems were built in the late 1800's or the early 1900's, and "as built" plans were not available. In order to facilitate collection of field data for rating and for use in providing uniform data to the rater, the Department devised a series of standard drawings for the various types of bridges commonly found on our local highway systems. The drawings include examples of truss type structures, single span I-beam structures, continuous I-beam structures, and timber structures, together with standards depicting the various types of substructures commonly found. A sample of the standard drawing used for continuous I-beam structures is shown (Figure 7).

The drawings are designed for a fill-in-the-blank type collection of data for simplicity in obtaining the field measurements. Using the standard drawings, the inspector in the field is able to review the drawings when he has completed his measurement of a structure and fill in any blanks to complete the report.

The standard drawings, with required entries, should also be supplemented with notes on the drawings and/or photographs showing any special problem areas, such as damage from vehicular traffic, areas of deterioration, etc. The dimensions on the standard drawings, plus the supplemental information on the condition of the structure will aid the rater in calculating the operating and inventory ratings for the structure as well as the recommendation for posting. Supplemental sketches or drawings can be attached to the standard drawings to show any unique design which may differ from the standard drawings.

Figure 8 shows a drawing of a substructure. The substructure drawings are used to provide more complete information to the rater. These drawings show the various common types of substructures, such as pilings, and dimensions of the various substructure members such as pile sizes, pile spacing, or dimensional factors of a solid concrete type substructure element. These drawings are to be supplemented with notations indicating the condition of the various elements by the field inspector to the rater.

Local agencies are asked to prepare a small sketch of the structure for which more than one type of superstructure or substructure element has been used to help the rater to relate the element to its proper function. The overall sketch also is to show the back-to-back length of the structure which can be used to confirm measurements for the individual spans. The local agency inspector is responsible for completing the forms and documenting basic measurements of a structure prior to the field inspection of the strucure by the Department. When the Department has completed the rating and has forwarded the rating and recommendation for posting to the local agency, the standard drawings and photographs taken during the field inspection are returned to the local agency for its records and future use.

To aid local highway authorities in determining the cost of the proposed improvements or replacement of structures, cost graphs reflecting the latest figures available to the Department are prepared. These cost graphs are updated annually. The graphs are guides for estimating structure costs only and do not include any earthwork, excavation, removal of existing structure, etc., since these items vary considerably from location to location. The graphs are compiled for several types of structures such as precast concrete, prestressed concrete, wide flange structures with concrete decks, etc. Each graph also has a weight average cost of various structures which can be used if the type of structure is undecided at the time the estimating data is needed. A similar graph is also provided for estimating the square foot cost for widening an existing structure. A list of standard pay items of the various types of work and the most recent unit cost of these items, compiled from repair work done on the State system, is also provided. For example, concrete removal in small quantities from zero to 10 cu. yds. is estimated for what can be expected as a unit bid price per cu. yd. These costs cover items encountered in repair or rehabilitation of structures.

The responsibility of each of the agencies involved in completing the structure inventory and the rating of the structure was established. In general, the collection of field data on the structures is the responsibility of the local highway authority. Guidance and assistance in the field collection of data is provided by Department personnel. When the field data has been collected on the appropriate structure inventory and appraisal forms, the information is forwarded to the Department for inclusion in the structure data bank.

After the inventory and appraisal data have been included in the structure data bank, the computer generates a printout of the complete information on the structure. Two copies of this print-out are returned to the local highway authority for its files. For future updates in the structure data bank the necessary revisions are made on the two copies of the computer printout. One copy is forwarded to the Department and the second copy is retained in the agency files. The data bank information is then updated and new printouts are generated. Two copies of the corrected printouts are returned to the local agency. This system, if properly implemented, will keep the structure data bank and local files current.

If a local agency replaces a structure, it is necessary to complete a new Structure Inventory Sheet and a Structural Appraisal Sheet, with the pertinent information on the new structure, and to assign a new number to the structure.

The new forms are submitted to the Department of Transportation, along with a copy of the old form for the replaced structure, including the notation that the structure has been removed and replaced by the new structure and indicating the new bridge number that will replace the old. The Department enters the new structure information into the data bank and deletes all reference to the old bridge. This method assures currency in the structure data bank.

It should be noted, however, that the only time a new structure number is assigned is when the bridge is completely new. If alterations or repairs to an existing structure are made, the old structure number is retained, and the items are updated on the old structure inventory to reflect the current structure condition.

Summary

The Department is very gratified by the acceptance of the overall program by the local agencies and their cooperative participation in the program, which has made it a substantial success. A plan of this nature can be successful only with the complete cooperation of all agencies involved. Since this program was undertaken, continuing interest is being shown by local agencies. This, we feel, is the result of planning to meet the needs not only of the Federal Highway Bridge Inspection Program and the Needs Study for the Legislature, but also the needs of the local agencies in their complex planning and budgeting processes.

Benefits have already been derived from the program. For example, the local highway authorities have recognized the inadequacy of the design requirements presently existing for bridges on local agency highway systems. As a result, the County Superintendents of Highways Policy Committee has adopted a new policy calling for all new structures to be designed for a minimum HS20 loading in lieu of the original HS15 loading used on many minor roads.

The information obtained in the bridge inspection program and the ever increasing loads carried over the local structures by farm-to-market and other heavy vehicles have made the need for higher design criteria obvious. Another benefit which has already been derived from the program is a suggested priority listing for repair and replacement of all structures on the local agency highway system for use by those agencies in planning and budgeting their highway needs and improvements. This advice is available to all local highway authorities who have completed and forwarded the inventory and appraisal of all structures on their respective systems.

Conclusion

Since the conception of this program, the local highway officials have become deeply concerned about the condition of their highway structures. As a result, the State has witnessed a large increase in the number of structure replacements and repair projects. Many local highway authorities have readjusted their priorities for expenditures of highway funds and allotted greater portions of those funds for repair and replacement of structures, yet providing that routine maintenance which is vital to road systems.

It is estimated that it will take from 4 to 5 years to incorporate all of the structures in Illinois into the program. However, as more and more structure inventories and appraisals are completed, the need for more highway dollars is increasingly evident. By continually updating the data bank with new information when repairs or replacements are accomplished, the Department can provide ample documentation to the State Legislature to justify legislation to meet the higher costs and urgent needs of the local highway system.

Illinois, like many other states, in past years did not have the mechanism to identify the overall needs on the local or State highway systems. The bridge inspection inventory and rating program will provide a continually updated survey of all structures and the current needs can be ascertained at any time. A program similar to the Structure Inventory, Appraisal and Rating Program for structures in Illinois has been established as a Road Inventory System in the State to provide a continually updated inventory of the road system. The two programs will clarify the overall automobile and truck transportation needs in the State.

Figure 1.

FORM DB-500 STRUCTURE INVENTORY SHEET

STRUCTURE NUMBER ________
DATE OF DATA ________

	FILE CARD	COMP. NUMB.	INV. CO.	KEY ROUTE ON STRUCTURE: TYP.	NUMBER	SUF.	SPUR	APPURT. NUMB.	STATION	KEY ROUTE UNDER STRUCTURE: TYP.	NUMBER	SUF.	SPUR	APPURT. NUMB.	STATION
DATA →	0 3 1 3														
COL. →	1 2 3 4	5 6 7 8 9	10 11 12	13	14 15 16 17	18	19	20 21 22 23 24	25 26 27 28 29	30	31 32 33 34	35	36	37 38 39 40 41	42 43 44 45 46

ITEM	DESCRIPTION	DATA	COL.
	FILE/CARD NUMBER	0 3 0 1	1- 4
	COMPUTER NUMBER		5- 9
2.	HIGHWAY DISTRICT		10
3.	STRUCTURE COUNTY		11-13
4.	MUNICIPALITY		14-17
5A.	LOG ROUTE ON KIND/DESIG.		18-19
	NUMBER/DIR.		20-24
5B.	LOG ROUTE UNDER KIND/DESIG.		25-26
	NUMBER/DIR.		27-31
5C.	PRINCIPAL ROUTE ON OR UNDER		32
6.	FEATURE CROSSED		33-52
7.	FACILITY CARRIED		53-72
8A.	STRUCTURE NUMBER		73-76
	FILE/CARD NUMBER	0 3 0 2	1- 4
	COMPUTER NUMBER		5- 9
8B.	OVERHEAD STRUCTURE NUMBER		10-13
9.	LOCATION		14-33
10.	NAME OF BRIDGE		34-53
	ROUTE ON - ITEMS 11A - 15A		
11A.	MILEPOINT		54-58
12A.	DOD ROAD SECTION NUMBER		59-63
13A.	DOD BRIDGE LETTER		64-65
14A.	DOD MILEPOINT		66-69
15A.	DOD SECTION LENGTH		70-72
	ROUTE UNDER - ITEMS 11B - 15B		
	FILE/CARD NUMBER	0 3 0 3	1- 4
	COMPUTER NUMBER		5- 9
11B.	MILEPOINT		10-14
12B.	DOD ROAD SECTION NUMBER		15-19
13B.	DOD BRIDGE LETTER		20-21
14B.	DOD MILEPOINT		22-25
15B.	DOD SECTION LENGTH		26-28
16.	STATE PLANE COORD SOURCE/ZONE		29-30
17A.	EAST-WEST COORDINATE		31-36
17B.	NORTH COORDINATE		37-43
18.	PHYSICAL VULNERABILITY		44
19A.	BY-PASS LENGTH ON		45-46
20.	TOLL FACILITY		47
21.	MAINTENANCE RESPONSIBILITY		48-49
22.	BUILT BY		50
23.	FED. AID PROJECT DESIGNATION		51-54
	ROUTE/SECTION NUMBER		55-58
	AGREEMENT NUMBER		59-61
	POSTMILE		62-64
24A.	FED. AID SYSTEM ON		65-66
25.	ADMINISTRATIVE JURISDICTION		67
26A.	FUNCTIONAL CLASSIFICATION ON		68-69
27.	YEAR BUILT/RECONSTRUCTED		70-75
28A.	NUMBER OF LANES ON/UNDER		76-79
28B.	ONE OR TWO WAY TRAFFIC		80

(Rev. 1/76)

ITEM	DESCRIPTION	DATA	COL.
	FILE/CARD NUMBER	0 3 0 4	1- 4
	COMPUTER NUMBER		5- 9
29A.	AVG. DAILY TRAFFIC ON		10-15
30A.	YEAR OF AVG. DAILY TRAFFIC ON		16-17
31.	DESIGN LOAD		18-19
32.	APPROACH ROADWAY WIDTH		20-22
33.	BRIDGE MEDIAN WIDTH/TYPE		23-25
34.	SKEW DIRECTION/ANGLE		26-28
35.	STRUCTURE FLARED		29
36A.	CONSTRUCTION ROUTE		30-36
36B.	CONSTRUCTION SECTION		37-61
36C.	CONSTRUCTION STATION		62-71
38.	NAVIGATION CONTROL		72
39.	NAVIGATION VERTICAL CLEARANCE		73-75
40.	NAVIGATION HORIZONTAL CLEARANCE		76-79
	FILE/CARD NUMBER	0 3 0 5	1- 4
	COMPUTER NUMBER		5- 9
37.	MICROFILM NUMBER		10-18
42.	TYPE OF SERVICE ON/UNDER		19-20
43.	MAIN STRUCTURE MATERIAL/TYPE		21-23
44A.	NEAR APPROACH MATERIAL/TYPE		24-26
44B.	FAR APPROACH MATERIAL/TYPE		27-29
45.	NUMBER OF SPANS - MAIN STRUCTURE		30-33
46.	NUMBER OF SPANS - APPROACHES		34-37
48.	LENGTH OF LONGEST SPAN		38-41
49.	STRUCTURE LENGTH		42-46
50A.	SIDEWALK WIDTH - RIGHT SIDE		47-49
	- LEFT SIDE		50-52
50B.	SIDEWALKS UNDER STRUCTURE		53
50C.	GUARDRAILS ON STRUCTURE RIGHT/LEFT		54-55
51.	BRIDGE ROADWAY WIDTH - TOTAL		56-59
52A.	HORIZONTAL CLEAR. RT-ONLY RDWY		60-63
52B.	HORIZONTAL CLEAR. LT RDWY		64-67
53A.	MIN. VERT. CLEAR. OVER RT-ONLY RDWY		68-70
53B.	MIN. VERT. CLEAR. OVER LT RDWY		71-73
53C.	10 FT. VERT. CLEAR. OVER RT-ONLY RDWY		74-76
53D.	10 FT. VERT. CLEAR. OVER LT RDWY		77-79
	FILE/CARD NUMBER	0 3 0 7	1- 4
	COMPUTER NUMBER		5- 9
54A.	VERTICAL UNDERCLEAR. RT-ONLY RDWY		10-12
54B.	VERTICAL UNDERCLEAR. LT RDWY		13-15
54C.	10 FT. VERT. UNDERCLEAR. RT-ONLY RDWY		16-18
54D.	10 FT. VERT. UNDERCLEAR. LT RDWY		19-21
54E.	HORIZ. UNDERCLEAR. RT-ONLY RDWY		22-25
54F.	HORIZ. UNDERCLEAR. LT RDWY		26-29
55.	LATERAL UNDERCLEAR. RIGHT EDGE		30-32
56.	LATERAL UNDERCLEAR. LEFT EDGE		33-35
57A.	WEARING SURFACE ON		36
57B.	WEARING SURFACE THICKNESS ON		37-38
19B.	BYPASS LENGTH UNDER		39-40
24B.	FED. AID SYSTEM UNDER		41-42
26B.	FUNCTIONAL CLASSIFICATION UNDER		43-44
29B.	AVERAGE DAILY TRAFFIC UNDER		45-50
30B.	YEAR OF AVG. DAILY TRAFFIC UNDER		51-52

Figure 2.

STRUCTURE APPRAISAL SHEET

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

File Number __________ 0 3
Card Number __________ 0 8
Computer No. __________
Structure __________

CONDITION

No.	Item	Material	Condition Analysis	Rating (9-0)
58	Deck			
59	Superstructure			
60	Substructure			
61	Channel & Channel Protection			
62	Culvert & Retaining Walls			
63	Estimated Remaining Life			
64	Operating Rating			
65	Approach Roadway Alignment			
66	Inventory Rating			

APPRAISAL

No.	Item	Deficiencies	Rating (9-0)
67	Structural Condition		
68	Deck Geometry		
69	Underclearances-Vertical & Lateral		
70	Safe Load Capacity		
71	Waterway Adequacy		
72	Approach Roadway Alignment		

PROPOSED IMPROVEMENTS

73 Year Needed____Completed____Describe (Item 75)____
74 Type of Service____
75 Type of Work____
76 Improvement Length________ft.
77 Design Loading____
78 Roadway Width________ft.
79 Number of Lanes____
80 ADT____
81 Year____
82 Proposed Rdwy. Improvement - Year____
83 -Type____

COST OF IMPROVEMENTS

84 $____,000.
85 Date of Inspection____
Remarks____

Form DB 551 (Rev. 5/72)

Figure 3.

ILLINOIS STRUCTURE INVENTORY	Item No. 43 (Revised)
Main Structure Material and Type	Effective 10 Sep 71

Description

A one-digit code and two-digit code used to identify the material and type of construction used. The main structure is all spans of most bridges (but the major unit only of sizable structures), or a unit of the structure with a different design and/or material from the approach spans. The major unit is usually the portion that spans the obstruction being crossed over and may consist of multiple spans with only one design and material type. Refer to Figure 1.1 and Figures 2.01 - 2.12. I

Purpose

To provide the data required by FHWA for national summaries and listings and to relate needs to the material and type of construction used.

Procedures

Code the type of material in the first digit and the type of construction in the next two digits.

Coding

Material (1st Digit)

Code	Material
1	Concrete
2	Concrete continuous
3	Steel
4	Steel continuous
5	Prestress concrete
6	Prestress concrete continuous
7	Timber
8	Masonry
9	Aluminum, Wrought Iron or Cast Iron
0	Other

• Type of Structure (2nd & 3rd Digits)

Bridges

Code	Type
01	Slab
02	Stringer/Multi-beam
03	Girder and Floorbeam System
04	Tee Beam
05	Box beam or Girders-Multiple
06	Box Beam or Girders-Single or Spread
07	Frame-Rigid & Other
08	Orthotropic
09	Truss - Deck
10	Truss - Thru & Pony
11	Arch - Deck, Filled Spandrel
12	Arch - Thru
13	Suspension
14	Stayed Girder
15	Movable - Lift
16	Movable - Bascule
17	Movable - Swing

Other Structures

Code	Type
18	Tunnel
19	Culvert
20	Pipe Line
21	Toll Plaza
22	Tollway Restaurant (overhead)
23	Pedestrian Overpass
24	Thru Girder without Floor Beam System
25	Arch-Deck, Open Spandrel
00	Other

Figure 4.

COUNTY________________
MUNICIPALITY________________
TOWNSHIP________________
SECTION________________

STATE OF ILLINOIS

Bureau of Local Roads and Streets

BRIDGE RECORD

BRIDGE NUMBER__________

On Federal Aid Highway System?
Yes☐ No☐

Station__________Built as__________Route__________Marked Route__________Year Built__________
Name of Stream, Railroad or Other Route__________
Total Length (Bk.-Bk. Abut.)__________Roadway Width__________Sidewalk Width__________Handrail Type__________
Type of Surface__________Minimum Vertical Clearance__________Skew Angle__________
Weight of Structural Steel (tons)__________Type of Bridge__________
Abutment Type__________Pier Type__________

Spans

No.	Type	Length	No.	Type	Length

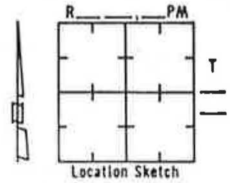
Drainage Area__________
High Water Elevation__________
Year of High Water__________
Waterway Opening__________Sq. Ft.
Flood Frequency__________Year

Design Loading__________

Maximum Load Limit

Date	Amount	Date	Amount	Date	Amount

R____,____PM



Location Sketch

Date of Inspection

Form BLR M 1030 Rev.(8-72)

Figure 5.

STATE OF ILLINOIS
Division of Highways
Bureau of Local Roads and Streets

BRIDGE INSPECTION REPORT - County __________

Highway No. __________ Section No. __________ Bridge No. __________
Location __________
Design or Posted Loading __________
Stream or Name __________

SUPERSTRUCTURE No. & Type of Spans __________

Truss: Paint __________ Lower Chord __________
Upper members __________ Stringers __________
Floor Beams __________ Gussets __________
Floor __________ Drains __________
Steel Beam: Paint __________ Handrail __________
Floor __________ Drains __________
Timber: Stringers __________ Floor __________
Hubguard __________ Handrail __________
Concrete: Stringers __________ Floor __________
Handrail __________ Drains __________

BEARINGS & EXPANSION DEVICES

In Expansion __________ In Contraction __________ Paint __________
Bearing Seats __________
Expansion Remaining __________

SUBSTRUCTURE

Abutment Type __________ Condition __________
Pier Type __________ Condition __________
Piling __________ Stream __________
Slopewalls __________

APPROACHES

Riding quality __________
Guardrail __________
Expansion Joints __________

REMARKS __________
____
____
____
____

Date __________ Inspected by __________ Approx. Temp. _____

Form BLR ML-1030
10-70

Figure 6.

STRUCTURE INVENTORY AND APPRAISAL SHEET

IDENTIFICATION Code
1 State 1 1
2 Highway District
3 County
4 City/Town
5 Principal Route
6 Feature Crossed
7 Principal Route ☐ Over ☐ Under
8 Structure No. ___ 1 of ___
9 Location
10
11 Route Principal Route Other
12 Milepost
13 Road Section No.
14 Sub-section No.
15 Latitude ° '
16 Longitude ° '
17 DOD Road Section
18 DOD Bridge Letter
19
20 Toll ☐ Yes ☐ No
21 Custodian
22 Owner
23 F.A.P. No.

CLASSIFICATION
24 Fed.Aid System
25 Administrative
26 Functional

By Date
Transfer of Data
Condition Analysis
Appraisal
Cost
General Review
Maintenance Inspection

STRUCTURE DATA
27 Year Built
28 Lanes on Str. ___ Under ___
29 ADT on Str. ___ 30 Year ___
31 Design Load Code
32 Appr. Rdwy. Width W/ Sh'ld
33 Median ___ ft. ☐ Open ☐ Closed
34 Skew °
35 Ground Level By-Pass ☐ Yes ☐ No
36 Hydraulic Structure ☐ Yes ☐ No
37 Report Available ☐ Yes ☐ No
38 Navigation Control ☐ Yes ☐ No
39 -Vertical ___ ft.
40 -Horizontal ___ ft.
41 Relief Structures

42 Type Service Code
43 Structure Type -Main
44 -Approach
45 No. of Spans -Main
46 -Approach
47 -Total
48 Max. Span Length ___ ft.
49 Structure Length ___ ft.
50 Sidewalk Rt. ___ ft. Lt. ___ ft.
51 Roadway (curb-curb) ___ ft.
52 Deck Width (out-out) ___ ft.
53 Vert. Clear. over Deck ___ ft.
54 Underclearance-Vert. ___ ft.
55 -Lateral-Right ___ ft.
56 -Left ___ ft.
57 Utilities

Item No. | Card Control Number | Card Col.

CONDITION Material Condition Analysis Rating (9-0)
58 Deck
59 Superstructure
60 Substructure
61 Channel & Channel Protection
62 Culvert & Retaining Walls
63 Estimated Remaining Life ___ 65 Approach Alignment
64 Permit Capacity ___ 66 Rated Loading

APPRAISAL Deficiencies Rating (9-0)
67 Structural Condition
68 Deck Geometry
69 Underclearances - Vert. & Lateral
70 Safe Load Capacity
71 Waterway Adequacy
72 Approach Alignment

PROPOSED IMPROVEMENTS
73 Year Needed ___ Completed ___ Describe
74 Type of Service
75 Type of Work
76 Improvement Length ___ ft.
77 Design Loading
78 Roadway Width ___ ft.
79 Number of Lanes ___ 82 Prop. Rdwy. Improvement - Year
80 ADT ___ 81 Year ___ 83 - Type

COST OF IMPROVEMENTS 84 $___,___,000.

Remarks

Figure 7.

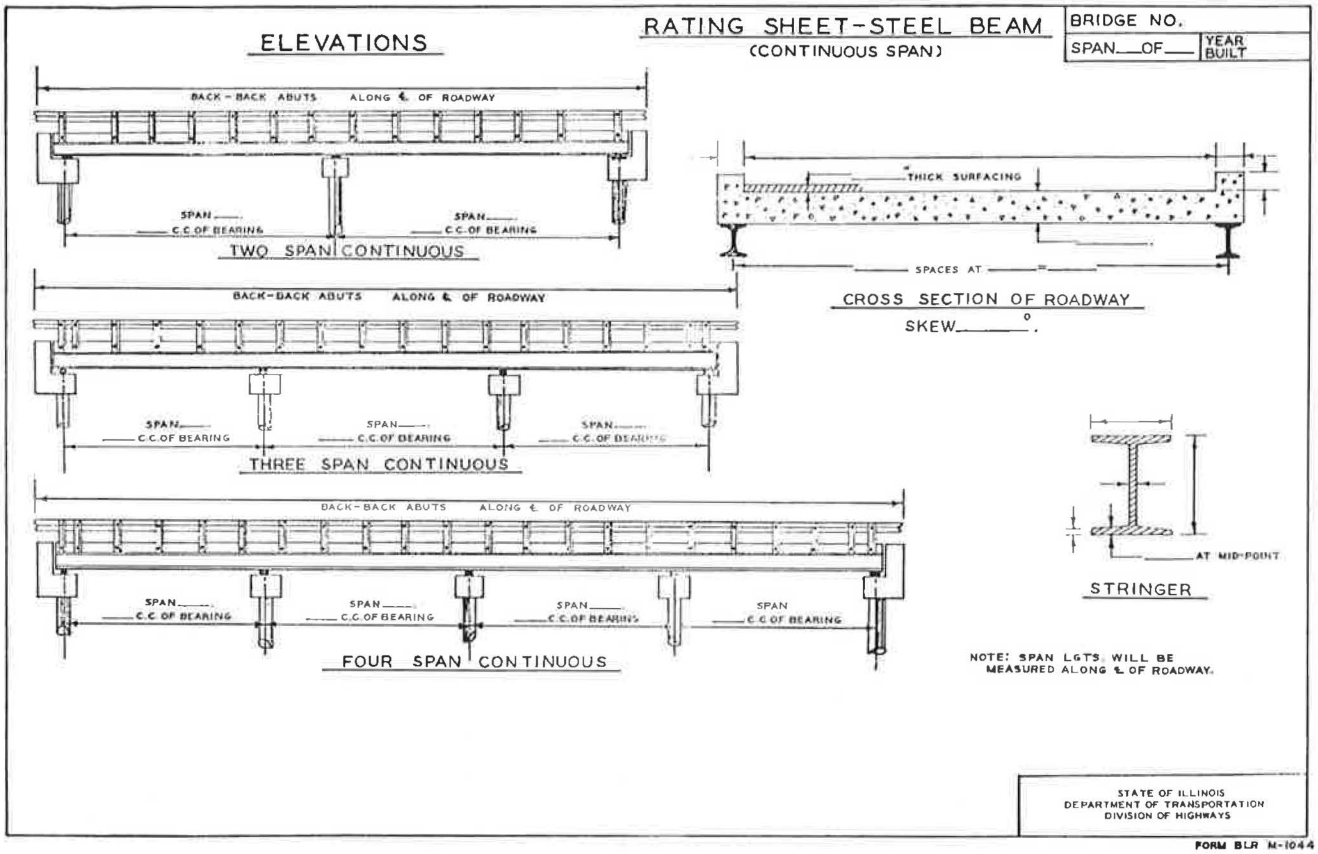
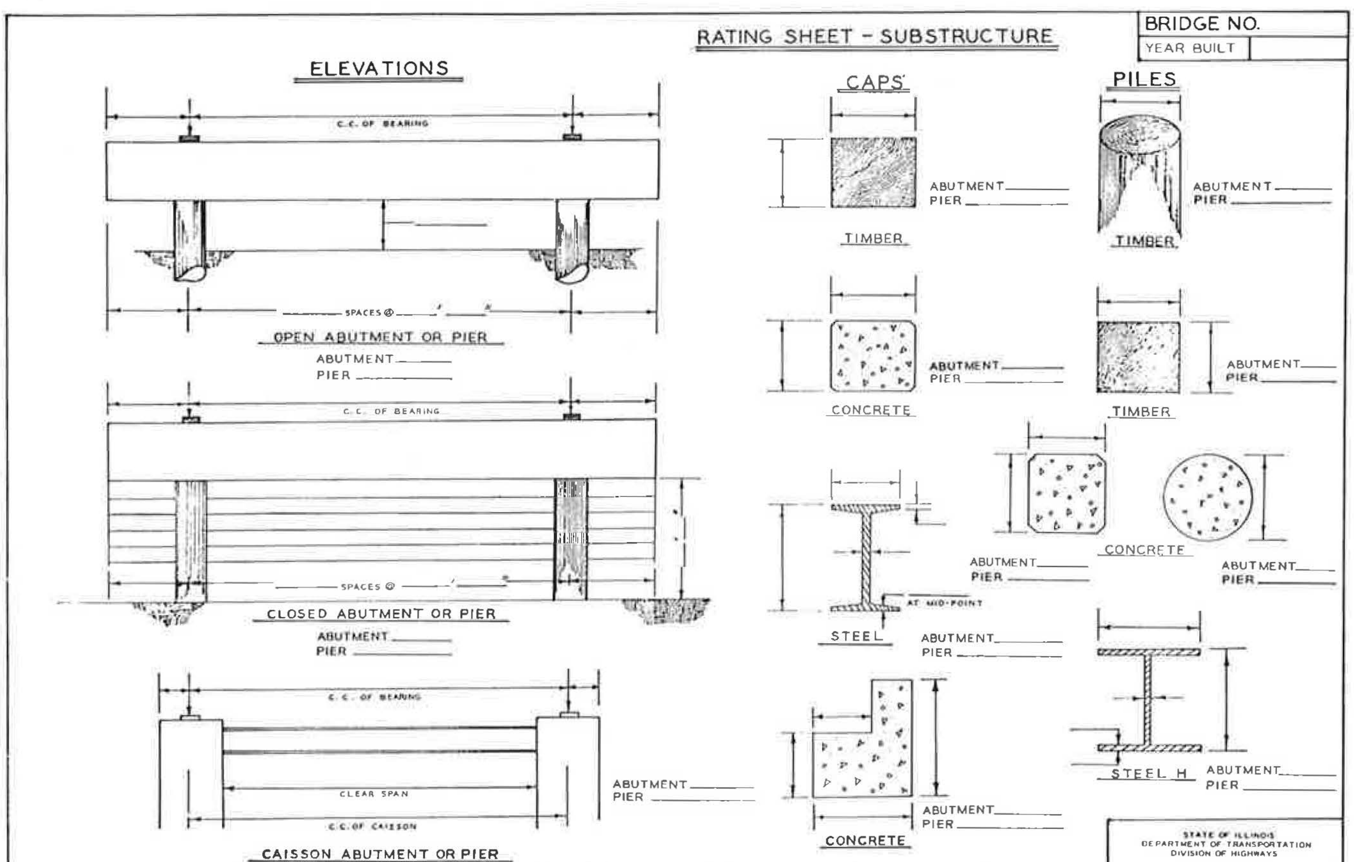


Figure 8.



LOW WATER CROSSINGS

Gerald Coghlan and Neil Davis, Forest Service, USDA

This paper provides a rationale for planning and constructing low water crossings on low volume roads. Fords, fords with culverts and crossings on low structures are described. Examples are given of good and poor designs using different types of materials and involving a variety of environmental considerations. Examples come from National Forests in Minnesota, Missouri, West Virginia and New Hampshire.

Definitions

Low water crossings are road-stream crossings designed to allow flooding approximately once a year. This contrasts with the more conventional stream crossings designed for 25-50 year floods. Low water crossings include the following, as shown in Figure 1:

1. Fords (or dips) - - Formed by lowering the road grade to the streambed level from bank to bank. Commonly used across dry drainages or where the day-to-day stream flow is low.
2. Vented fords (or dips with culverts) - - Formed by partially lowering the road grade for floods and providing culverts to handle the day-to-day flow. Commonly used where day-to-day flow exceeds a fordable depth.
3. Low Water Bridges - - Formed by partially lowering the road grade for flooding and providing a bridge type structure to handle day-to-day flow that can not be handled by culverts.

It is reasonable to drive even a modern car through 10-15 cm (4"-6") of water. Single use access roads, such as those for logging or recreation, rarely need 100% access. Often the activity itself may not be feasible because of the same heavy rain which closed the road. Scattered farm residences usually can operate adequately with occasional road closures. Often there may be alternate but longer access routes available when high water temporarily closes one crossing.

Most reference books on highway engineering (1, 2, 3, 4, 5, 6) stress the higher standard stream crossings designed to pass 25-50 year floods. A few texts (7, 8) do make brief reference to low water crossings as economical alternatives for low volume roads, but they provide few details. T.R. Agg referred to fords in his 1929 edition of "Construction of Roads and Pavements" (9), but the reference was dropped in his 1940 edition.

The only details on low water crossings that we found were given by A.D. Leydecker (10), and Sharma and Sharma (11). Leydecker describes the use of gabions in constructing a ford. Sharma and Sharma describe both a rubble and concrete ford, and a dip with culvert.

Location and Design Considerations

Raised road grades and constricted drainage waterways can cause flooding in broad, flat stream valleys. These flood plains often include valuable crops as well as residences. The low water crossing permits a large, natural waterway which minimizes the flooding of adjacent lands. High approaches to flood-free crossings may be expensive and aesthetically undesirable. Occasionally, structures that have adequate clearance to pass floods have low approach roads. Since access can be restricted when approach roads become flooded, it makes little sense to overbuild a structure to insure access. A low approach road which ramps up and over a high stream crossing also presents an undesirable hump or roller coaster appearance and ride.

In mountainous terrain, streams on alluvial fans often appear to be running on top of the ridge. These streams have widely varying, rapidly changing flows and very unstable channels. Stream crossings built to handle peak flows can be large and may also appear humped. Floods backed up by the constricted waterway often jump around the ends of such structures causing washouts or even running down the road. Maintenance to keep a mountain stream in its channel can be very expensive or even impossible. Low water crossings in these cases offer a low investment in a high risk situation and minimize the chances of causing a channel change.

Figure 1: Types of Low Water Crossings.

(a) Ford (or dip) on the Mark Twain N.F., Missouri

(b) Vented ford (or dip with culvert) on the White Mountain N.F., New Hampshire.

(c) Low water bridge on the Monongahela N.F., West Virginia

Swampy areas have weak foundation conditions, making high approach embankments and large structures particularly expensive and impractical.

In arid or semi-arid regions, drainages carry little or no water except during sudden, severe storms. Structures adequate to carry these large, infrequent floods may be prohibitively expensive. In mountainous areas, peak flows are often short lived because of the mountain-caused squalls, small drainage areas and steep stream gradients. In northern climates, floods are often associated with spring runoff, while flows may be low and steady during the remainder of the year. In nearly all areas, small drainages can be found that run only a small volume of water even during heavy rains.

Economics provide the common denominator for comparing alternatives, whether low crossings are considered because of limited access needs, environmental concerns, terrain conditions, foundation conditions, or climates. The savings in construction cost and materials, property damage, and maintenance for low water crossings can be significant. Decreased environmental and aesthetic impacts are significant although difficult to assess economically.

A suggested criteria for access (11) is closure one to three days at a time, totaling not more than 15 days a year. The designer, of course, has the flexibility to vary these criteria for each specific situation. We typically design low water crossings for the maximum annual storm.

The exposure of a low water structure to the full impact of overflowing water is possibly the most important design consideration. Erosion downstream and around the ends of the structure cause the major maintenance problems and even failures. Debris carried by the stream, such as ice or logs, contribute to these erosion problems. Cutoff walls and riprap usually are not carried far enough along the roadway to protect against high water. While the waterways of low water crossings may only be designed for annual storms, the structure itself may necessarily be designed to resist washout from the 25-50 year storm.

Fords

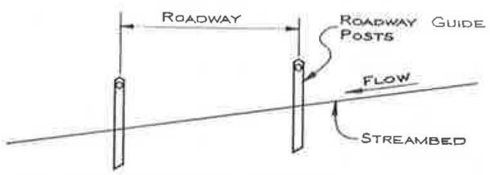
In their simplest form, fords consist of an unsurface stream crossing on the natural bed of the stream or drainage. More commonly, the stream bed is leveled for the width of the roadway as shown in Figures 2 and 3. Leveling may be accomplished by placing a row of boulders along the downstream roadway edge and filling behind the boulders with gravel. A more reliable design may utilize gabions along the downstream edge. Low water crossings are usually upgraded as their use increases.

Once the stream gradient has been changed, as with boulders or gabions, downstream erosion usually accelerates. Unless the crossing is on bedrock or large boulders, or the stream gradient is zero for some distance downstream, a plunge pool develops just below the crossing. As this plunge pool grows, it may undermine the roadway support, creating maintenance problems. Adequate embedment of the boulders or gabions and additional boulders for stream energy dissipation are important.

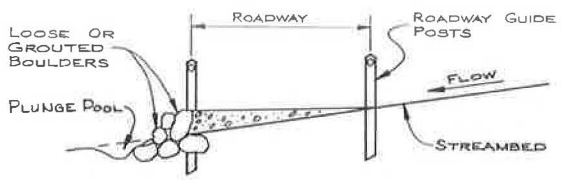
Fords may be surfaced with concrete, asphalt or gabions. The surfacing protects the crossing from erosion, and provides the driver a stable, tractive surface.

Figure 2: Examples of unsurfaced fords (no scale).

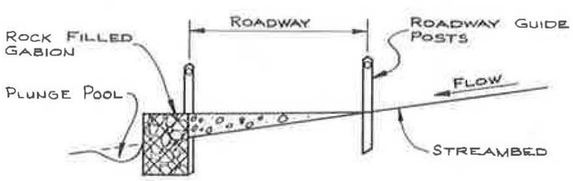
(a) Ford on natural streambed.



(b) Ford with downstream boulders.



(c) Ford with downstream gabion (10).



Reinforced concrete provides the strength adequate to carry traffic over weak stream beds. Concrete fords such as shown in Figure 3(c), built 20-30 years ago, have given maintenance-free service. On the other hand, concrete has a high initial construction cost and erosion through cracks in the concrete can cause settlement and further deterioration.

Asphalt surface treatments are usually adequate to protect the driving surface. However, where used for erosion protection along the downstream edge of the roadway, a 10-15 cm (4-6 in.) layer of asphalt and aggregate must be used.

Gabion surfaced fords provide the flexibility and erosion resistance characteristic of gabion structures. Traffic may eventually break down wires on the surface, but the side baskets continue to hold the crossing together.

Vented Fords

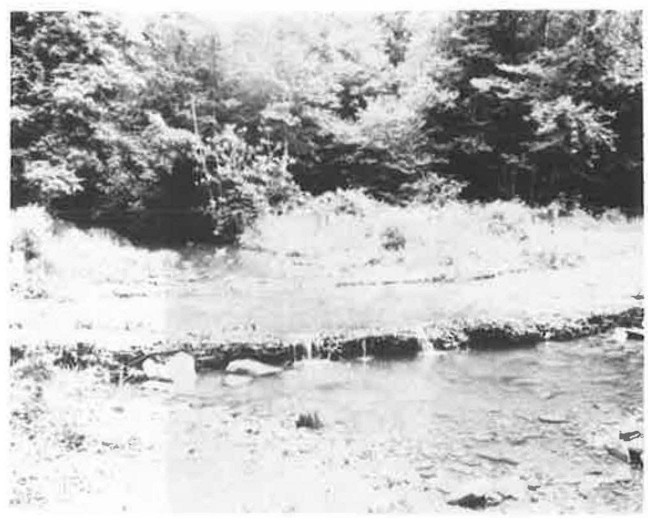
Vented fords, or dips with culverts, create a very significant velocity barrier with severe erosion potential.

When maintenance crews upgrade fords to small vented fords without benefit of a design, problems often develop. Materials usually include hot or cold mixed asphalt and concrete grouted rubble stone. Experience with these smaller vented fords has resulted in development of the typical sections shown in Figure 4. The sloped culvert entrance and sloped embankment catch less debris and clean themselves during high water. The sloped culvert entrance, particularly the formed metal entrance section, also improves the culvert capacity. A splash apron along the downstream

Figure 3: Examples of fords.

(a) Gravel ford on the Mark Twain N.F., Missouri.

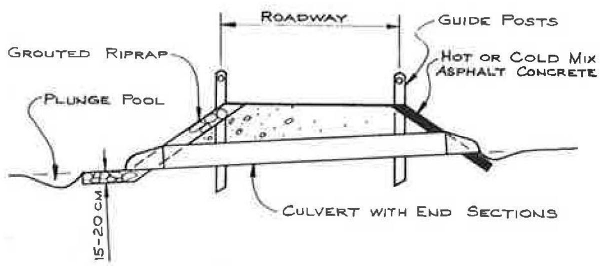
(b) Gabion ford on the Monongahela N.F., West Virginia.



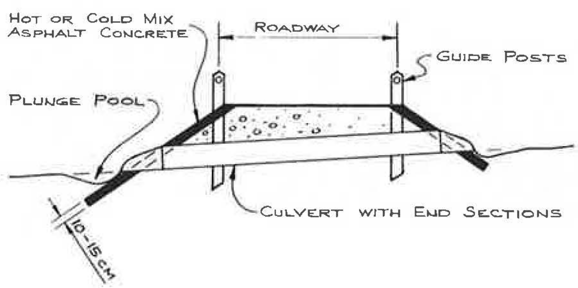
(c) Concrete paved ford on the Mark Twain N.F., Missouri.

Figure 4: Small vented ford sections (no scale).

(a) Grouted riprap and asphalt section.



(b) All asphalt cover.



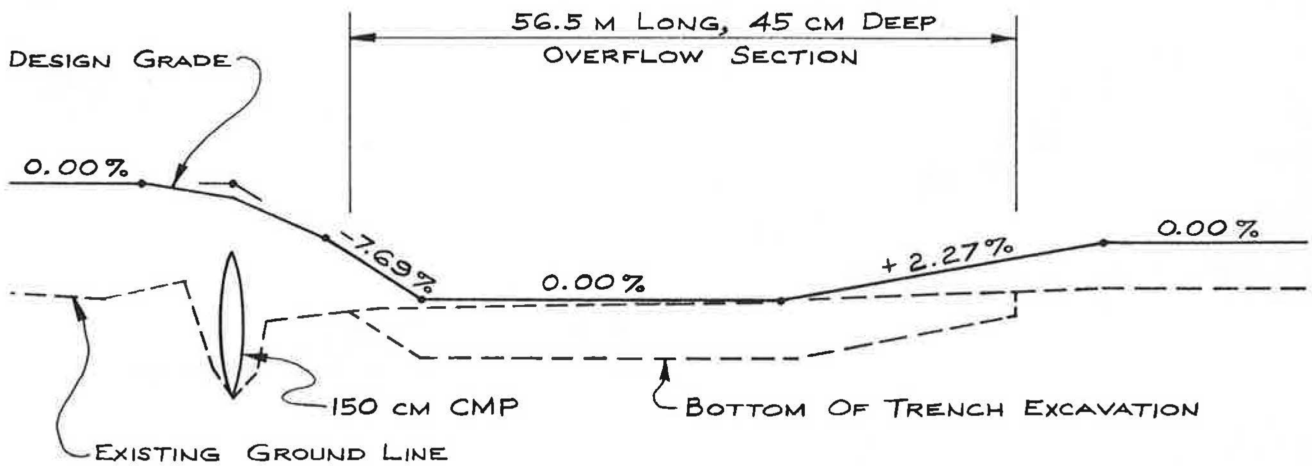
edge of the crossing will move the plunge pool further downstream, preventing undermining of the roadway and culverts. However, where only asphalt contains the road section, Figure 4(b), a cutoff wall should be used rather than a splash apron. The weaker asphalt aprons break off and deteriorate, quickly becoming a maintenance problem.

Figure 5 shows a variation of a vented ford in which a dipped overflow section was offset from the culvert. The single 150 cm (5 ft.) CMP and 56.5 m (185 ft.) dipped section was designed as an economic alternative to the five CMP's that would have been necessary to provide for a 25 year design period. Woven plastic filter fabric and rock were used for erosion protection of the overflow section and the roadway surface was given a single asphalt treatment.

Larger vented fords may be cast-in-place concrete structures encasing culverts, as in Figure 6. These structures have been designed both with and without wheel guards. Experience has shown that sloping the traffic face of the wheel guards, particularly on the down stream edge, reduces the collection of ice and debris during the overflow periods (Figure 6(c)). The surface slabs on these type structures must be well anchored to prevent uplift and displacement during high flows.

Figure 5: Culvert with offset overflow section, Chippewa N.F., Minnesota (no scale).

(a) Road centerline profile.



(b) Cross section through overflow.

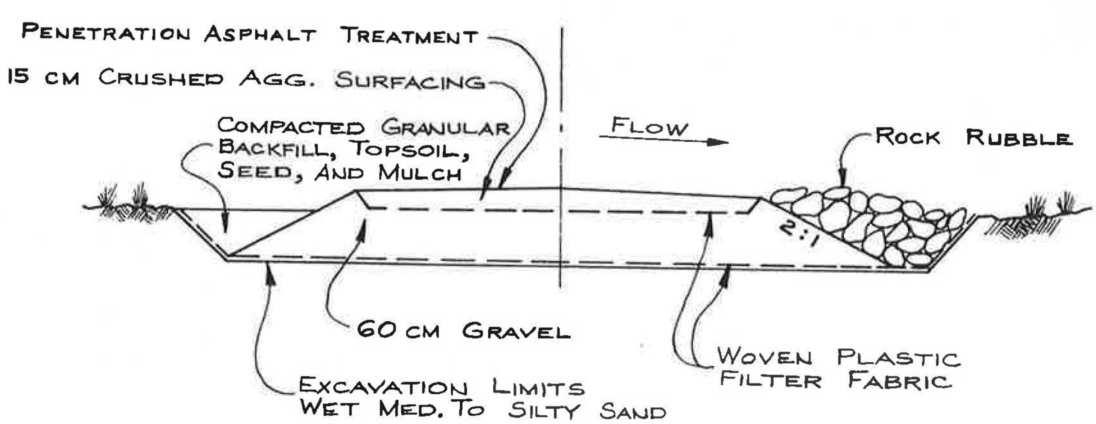
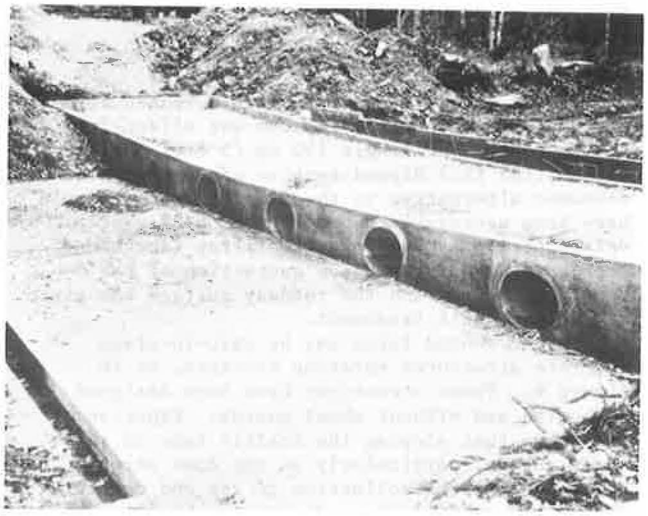


Figure 6: Examples of concrete vented fords on the Monongahela N.F., West Virginia.

(a) Vented ford under construction.



(b) Completed vented ford.

(c) Overflowing vented ford.

Figure 7: Examples of Low Water bridges.

(a) Low water bridge on the Mark Twain N.F., Missouri.

(b) Low water bridge overflowing on the Monongahela N.F., West Virginia

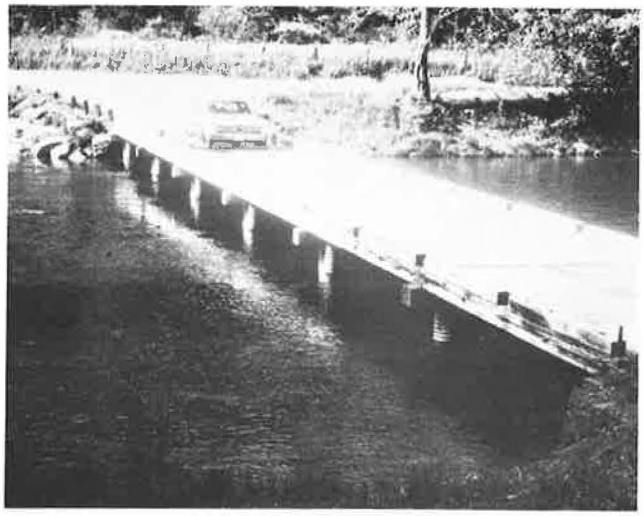
Low Water Bridges

Figures 1(c), 7 and 8 show examples of low water bridges constructed of concrete and concrete with wood decks. These structures can include 10-45 m (30 - 150 ft.) crossings, and spans of 5-10 m (14-30 ft.). Cutoff walls and/or riprap should be carried above the 50 year storm level where practical. Where this may be impractical due to the low approaches to the crossings, cutoff walls should be carried around the ends of the structures to protect the structure itself.

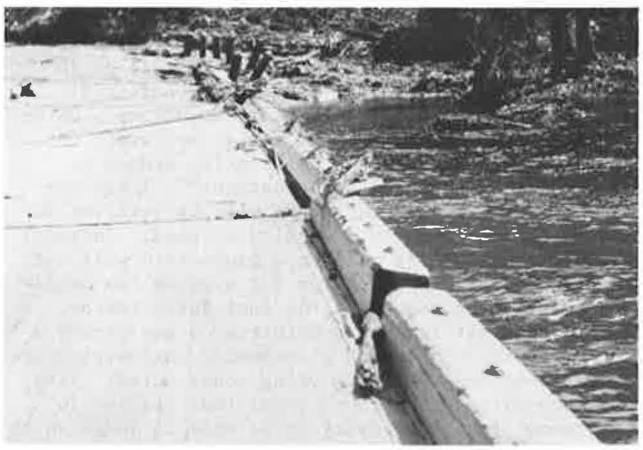
The low water bridge shown in Figure 8 was designed with pier footings to be buried 100 cm (3 ft.) into the river bottom. Built in 1966, the footings were embedded only 30 cm (1 ft.) into the stream bed. The structure performed well until the winter of 1976-77, when an ice flow caught on

Figure 8: Low water bridge on the Monongahela N.F., West Virginia.

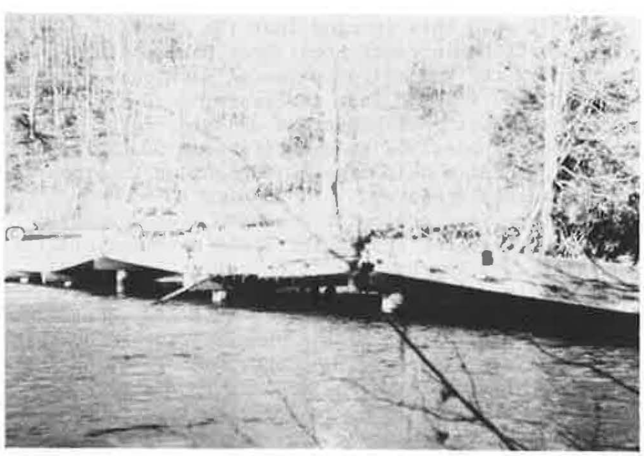
(a) Structure shortly after construction in 1966.



(b) Downstream wheelguard caught and held ice flow during the winter of 1976-77.



(c) Displacement caused by ice flow, water pressure and erosion.



the downstream wheelguard. The excessive forces and current lifted and tilted most of the pier footings. Figure 8(c) shows the structure with as much as two feet vertical displacement.

Had the footings been buried the designed depth, damage would most likely not have occurred. Also, sloped wheelguards may not have caught the ice flow. The gabion protected approaches suffered significant erosion. However, without the gabions, the approaches would certainly have been completely lost. A new all-concrete replacement low water crossing has been designed.

Summary

Low water crossings have proven adequate and economical under a variety of environmental and terrain conditions. While difficult at first for some designers and planners to accept, the advantages of low water crossings can be seen, particularly for low volume roads.

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EVALUATION OF THE STRUCTURAL ADEQUACY OF BITUMINOUS PAVEMENTS FOR COUNTIES AND MUNICIPALITIES IN MINNESOTA

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Erland O. Lukanen, Minnesota Department of Transportation

This paper presents the results of a cooperative study between the Minnesota Department of Transportation and a number of the counties and municipalities in Minnesota. It is sponsored by the Minnesota Local Road Research Board. The purpose of the project has been to make flexible pavement evaluations techniques available and usable by the local agencies. The procedures are now available in Minnesota. The evaluation procedures include the Benkelman beam to evaluate the pavement strength and load carrying capacity, surface condition surveys, and roughness measurements with the Brokaw Roadmeter to determine present serviceability index. In addition, detailed traffic analyses are being made on specific roads in the three counties and municipalities. The vehicle weights and distributions are being determined to compare with similar distributions on primary roads. A procedure developed by the Minnesota Department of Transportation (Mn/DOT) is used for converting traffic weights and distributions to predict equivalent 80 kN (18,000-lb) single axle load applications. The project includes evaluation of about 193 km (120 mi.) of bituminous pavements in each of three counties and one 1.6 km (1 mi.) segment in each of six municipalities. The paper will demonstrate the relevance of the pavement evaluation techniques on low volume roads and how the results can be used to set up a pavement inventory system. This system can then be used to lay out a maintenance schedule for the participating counties and municipalities. All of the procedures presented are usable and available to the counties and municipalities in Minnesota. The results of the study are making it possible for the local engineers to make decisions on when maintenance is needed on a given road and what the most appropriate procedure would be. The data obtained give the engineer factual information about the road to aid his judgment in making maintenance decisions.

The cost of 2.54 cm (1 in.) of asphaltic concrete overlay is now approximately $10,000 per 1.6 km (1 mi.). Pavement engineers, therefore, want to make sure that the pavement to be overlaid is really in need of that overlay. Also, the engineer would like to be assured that with the investment in an overlay or some other type of maintenance the structure there will withstand the expected traffic. Questions such as "Will the additional investment last long enough to be justified?" or "Will a 5.08 cm (2 in.) overlay last 10 or 20 years, 10 or 20 months, or 10 or 20 days?", should be answered. Another question that could be asked, "Is a 5.08 cm (2 in.) or 10.16 cm (4 in.) overlay necessary or could one get by with just a leveling course or some other type of surface treatment?" There are many criteria and factors that will be involved in making the decision on a particular road. In fact, the decision that is made on a given road will not only depend on its condition but also on the condition of other roads under the same jurisdiction.

In the last few years maintenance has become a much more important part of pavement engineering because fewer new roads are being constructed. With the many miles of surfaced roads that are now in existence, it is necessary to be able to judge which ones should be maintained and what maintenance procedures are most appropriate. For all except the very smallest of jurisdictions, some type of a maintenance management system should be used to establish priorities.

It is with this in mind that the steering committee of the Minnesota Local Road Research Board has chosen the subject of pavement maintenance management. During the last few years, work has been done to present rating systems and procedures which will make it possible to quantify some of the parameters needed to help make a judgment as to proper maintenance procedures.

So far, four presentations have been developed and been presented around the State of Minnesota.(1) These are: 1) Surface condition rating system; 2) Rideability; 3) Traffic; and 4) Strength.

The surface condition rating system presents a procedure for evaluating the characteristics of the bituminous surface only. By observing these and putting them into a rating scheme, it has been possible to determine if a given pavement is in need of a surface treatment or seal coat or some other type of resurfacing.

The presentation on rideability describes methods by which the roughness of the pavement can be converted into a rating from 0 to 5. This is defined in terms of the present serviceability concepts developed at the AASHTO Road Test.

In the traffic presentation, a method is described which makes it possible to calculate the load effect on the pavement in terms of equivalent 80 kN (18,000-lb) single axle loads.

The strength of the road is defined using two procedures. The first considers the type or strength of the embankment and the thickness of the pavement section layers. The second uses the Benkelman beam deflection test which gives a direct measure of the strength of the road at the time of the test. With these procedures it is possible to estimate the life of the road under the predicted traffic and also determine what the allowable load should be on that road during the critical spring period.

Each of the procedures presented uses concepts and equipment that are readily available to the counties and municipalities in the State of Minnesota. The procedures presented are either those used directly by Mn/DOT or are slightly modified from procedures which are used on the trunk highway system. It is recognized that there are other factors that will govern when and what is done to a particular pavement section. However, the parameters that are presented will help in the decision makers by making available more specific information on which to base their judgment.

This paper is a brief resume of each of the parameters and procedures which have been presented. A method of summarizing this information into usable form is then suggested.

Summary of Parameters

Surface Condition Rating System

The surface condition rating system includes procedures and suggestions for making a set of surface condition ratings. This information can be summarized on a surface condition rating form.(1,2) The conditions considered under this scheme are: 1) General structural condition; 2) Surface wear; 3) Weathering; 4) Skid resistance; 5) Uniformity; and 6) Crack condition, a) opening, b) abrasion, and c) multiplicity. Each of these eight conditions can be given a rating from 0 to 5.

The general structural condition gives the rating of how good that pavement is performing structurally. The ride may be satisfactory, but it may still have some cracking and patching developing.

The surface wear is a measure of how much the pavement is being worn down by the effect of tires or how badly the pavement is bleeding.

The weathering gives a rating of how deteriorated the surface is due to the affects of temperature, water and wind.

The skid resistance rating is suggested as a means of estimating skid resistance when a number from a skid trailer is not available.

The uniformity rating gives an indication of how blotchy, streaked or nonuniform the surface looks generally.

The crack condition rating is a measure of the crack width, how much they are abraided and whether there are associated multiple cracks along with the transverse of longitudinal cracks. Descriptions for ratings of these parameters are given.(1,2) In the surface condition presentation, some examples are given showing pictures of pavements which have been rated at the various levels of these conditions.

It is important that those who would wish to use this rating system do some practice rating before surveying a system of roads.

The ratings can be recorded on the surface condition rating sheet (Table 1) and then entered in Table 14, which is a summary of the pavement conditions.

Rideability

For the pavement management procedures developed in Minnesota, the rideability has been defined using the present serviceability concept. With this concept the rideability is defined as the ability of a section of road to serve the traffic that it was designed for. The rating is actually an average of the opinions of a group of individuals on how well that road rides based on a scale from 0 to 5. In the rideability presentation, two procedures are suggested for determining the rideability of a given section of road: 1) Use of a rating panel; and 2) Use of the Portland Cement Association (PCA) Roadmeter.

The rideability can be estimated using as few as three raters. However, if more ratings or opinions are obtained, this will give a better estimate of the rideability. The following nine rules should be followed if the rating panel system is to be used.

1. Use the following descriptions to define the ride as related to the numerical ratings: 4 to 5 - Very Good; 3 to 4 - Good; 2 to 3 - Fair; 1 to 2 - Poor; and 0 to 1 - Very Poor. Ratings between these descriptions (eg., 2.4) can be used to indicate levels between those shown. The rater should ask himself how he would like to ride on a pavement like this all day. This guideline may not strictly apply to shorter county roads or city streets. However, an indication of how well that road is serving the public should be made.
2. The rater should disregard grade, alignment, right-of-way width, shoulders, ditch conditions, etc., and other conditions which do not directly affect ride or are governed by the structure of the pavement.
3. Ride the pavement sections at the posted speed limit.
4. Ratings should be made for each 0.8 km (1/2 mi.) in rural areas and 0.4 km (1/4 mi.) in urban areas as it is difficult to remember the level of ride for longer distances.
5. There should be no discussion of ratings during a session if there is more than one rater in a car. There could be some discussion after a session, but it should be remembered that there is no absolute right rating. Two people will not necessarily judge the ride in exactly the same way.
6. The average rating for each 0.8 km (1/2 mi.) or 0.4 km (1/4 mi.) should be recorded as the present serviceability rating for that portion of road. These distances need not be exact and if there are other limits which are appropriate they should be used.
7. Raters should go on practice runs periodically to help calibrate themselves. It would be good to have a series of roads in the area which are examples of high and low ratings to ride over periodically.
8. Ratings should be done in passenger cars in relatively good condition. The raters should also be in relatively good condition (not tired, etc.).
9. Ratings on roads in good condition (3.0 to 3.5) or higher need only be taken every two or three years, whereas those with lower ratings (less than 3.0) should be rated about every year.

Use of the PCA Roadmeter

The PCA Roadmeter is composed of a set of counters which accumulate the number of 3.2 mm (1/8 in.)

deviations between a car axle and frame from a null position when driving over a section of road. The rating is done using a standard automobile. The PSR is determined using a relationship between counts and serviceability established for that vehicle. There are ten Roadmeters available throughout the State of Minnesota, one in each Mn/DOT construction district and one in the Mn/DOT central office. When the Roadmeter is used it should be calibrated periodically. Rules to follow in the operation are included in an appendix to the rideability presentation. Usually a Mn/DOT district will be able to run Roadmeter ratings for counties or municipalities if enough lead time is allowed for scheduling. It may also be possible for three or four counties and/or municipalities to cooperate to obtain their own Roadmeter. The devices are available commercially or can be built and installed in any standard car using plans available from Mn/DOT.

After the present serviceability rating is determined either using a panel or the PCA Roadmeter, this value should be entered in the appropriate place on the Summary of Pavement Conditions Sheet (Table 14).

Traffic

On the Summary of Pavement Conditions Sheet (Table 14), there are four entries for traffic. The first is the AADT which is the total two-way average annual daily traffic. This value can be obtained either from a traffic flow map for the municipality or county, or by a traffic study on a road considered to have similar traffic, or if it is an existing road, on that given road. To calculate the one-way ADT, the two-way value is usually multiplied by 0.5 for two lane roads or 0.45 for four lane roads. The speed limit should also be recorded.

The equivalent 80 kN (18,000 lb) single axle loads ($\Sigma N18$) can be determined using one of the two methods available in Minnesota. The procedure is summarized on the calculation sheet (Table 2). The parameters that are required for this calculation are the AADT, the distribution of vehicles, the average effect of each vehicle at that location, and some indication of a growth factor. The AADT can be determined using a traffic flow map. The distribution of vehicles can be determined either by making a vehicle type study on the road being proposed for maintenance, on a similar road. An assumed distribution could also be used.

If a vehicle type survey is to be made, it is conducted for 16 hours on two consecutive weekdays other than Monday or Friday. The survey should be made from 6:00 a.m. to 2:00 p.m. on one day, and 2:00 p.m. to 10:00 p.m. on the next. Vehicles are classified according to the types listed in Table 3 which are used for classification by the Planning and Programming Section of Mn/DOT. The results of the 16 hour count are listed in Column 2 of Table 2. These values are then modified with the seasonal adjustment factors listed in Table 4. The appropriate factors are entered in Column 3. The seasonally adjusted number of each type vehicle (Column 4) is obtained by multiplying Column 2 by Column 3. The seasonally adjusted percentage is then calculated by summing Column 4 and taking each truck type as a percent of the total.

If it is not possible to run a traffic survey, then assumed percentages listed in Table 5 can be used. Some judgment should be used in modifying these values if it is felt there is some appropriate variation to use. The design lane AADT had been determined and entered in Column 6. The design lane distribution is then calculated for each vehicle type by multiplying Column 5 by Column 6 for each vehicle type.

The average effect of each vehicle type on the performance of the road is called the N18 factor. This can also be considered the number of equivalent 80 kN (18,000 lb) single axle loads on the average imparted to that pavement each time one of that type vehicle passes a location. Table 6 is a listing of average N18 factors for 62.3 kN (7-ton) and 80 kN (9-ton) roads in Minnesota. For specific situations, these factors can be modified in consultation with a knowledgeable traffic engineer. The N18 factor for each vehicle type is entered in Column 8 of Table 2. To calculate Column 9, which is the design lane daily N18 for each vehicle type, each entry in Column 7 is multiplied by the respective value in Column 8. The summation of Column 9 is then the total daily N18 at present for that road.

The number of years to be used for design is then entered along with an estimated percent growth on the bottom of Table 2. Time growth factors for 10 and 20 year periods are listed in Table 7 for various rates of growth. As indicated in the table, a growth of 0.5 percent is suggested for 62.3 kN (7-ton) roads and 3.5 percent for 80 kN (9-ton) roads. Again, if the conditions warrant it, other design periods and/or annual growth rates can be used. The time growth factor is an annuity factor and thus can be found in standard annuity tables.

The $\Sigma N18$ for the design period is then calculated by multiplying the daily N18 by 365 and multiplying that product by the time growth factor. This value should be entered in the appropriate location in Table 14.

Strength

The strength of a pavement section is defined using two methods. These include either the structure of pavement section or using a direct measure of strength which for the State of Minnesota has been defined using the Benkelman beam deflection test.

The definition of strength is related to the number of equivalent 80 kN (18,000 lb) single axle loads that the road can take before the serviceability is reduced to some level defined as failure. For most state highways, this level is taken as the PSR, or serviceability level of 2.5. However, for lower traffic municipal and county roads, this level may be taken as a serviceability level rating of 1.5.

Strength Defined Using Pavement Section. One method of measuring the strength is using the pavement structure. The pavement structure is made up of the embankment and the various layers of base and surfacing. In order to determine the structure by this method, it is necessary to know the type or strength of embankment and the thickness of the layers broken down into subbase, base and surface. This information can be obtained from records or by making borings.

For the embankment, the stabilometer R-value must be determined. If the R-value has not been run on the given soil in the laboratory, it can be estimated using the AASHTO classification or the textural classification with Table 8, which is Table F of the Mn/DOT Road Design Manual.(3)

The layer thicknesses are converted to granular equivalent thicknesses for the section using the G.E. factors listed in Table 9, which is Table D of the Mn/DOT Road Design Manual.(3) The granular equivalent can then be calculated with the following formula:

$$G.E. = a_1D_1 + a_2D_2 + a_3D_3$$

The values of a_1, a_2, and a_3 can be obtained from Table 9. If a pavement section is deteriorating, then some judgment has to be used to estimate what factor is appropriate for the layer. The R-value and granular equivalent thickness should be entered in the appropriate location in Table 14.

Using the R-value and the granular equivalent thickness for a given pavement section, the present Mn/DOT design chart for flexible pavements can be used to estimate how much traffic the section should be able to withstand before the serviceability level has dropped to 2.5 or 1.5.

Estimation of Strength Using the Benkelman Beam Deflection Test. One of the direct measurements of strength using a load test is the Benkelman beam deflection test. For this test, a known axle load can be run over the pavement section and the deflection under that load measured. An advantage of this approach for estimating strength over the granular equivalent method is that moisture conditions and other local variations are taken into account. A disadvantage is that there are different levels of flexibility of pavements and, therefore, what would be a critical deflection level for one may not be for another.

The testing equipment and operational procedures for running the Benkelman beam deflection tests are given in Appendix A of the fourth part of Reference 1. The procedure outline gives the equipment and procedures required to obtain deflections every 152 to 305 m (500 to 1000 ft.).

With the deflections determined, it is then necessary to calculate the design deflection which represents a given section of road (usually taken as a mile). In order to do this, the following variables are considered: 1) Temperature; 2) Time of year; 3) Load; 4) Thickness of layers; 5) Strength of embankment; and 6) Variability measurements.

The temperature is corrected by using Table 10, which shows the temperature correction to 27°C (80°F) for deflections run at other temperatures. The deflections are corrected only for tests at a temperature less than 27°C (80°F).

Deflections are converted to a critical spring value using the factors in Table 11. The ratios are dependent upon the time of year and the thickness of the asphalt layer in the pavement section.

The axle load on the test vehicle used is typically a 62.3 kN (7-ton) or 80 kN (9-ton) axle. It is important to know what the load is. Then deflections can be calculated for other loads by taking an arithmetical ratio of the loads. Using the spring ratios, a spring deflection for that section of road is determined.

Table 12 shows allowable deflections for various thicknesses of bituminous surface and levels of traffic. The allowable spring axle chosen from the table, the axle load used for the deflection test and the design spring deflection are substituted into equation 1 to calculate the allowable spring axle load in tons.

$$L_A = L_B \times \frac{\text{allowable deflection}}{\text{design spring deflection}} \qquad (1)$$

Where:

L_A = Allowable axle load, kN

L_B = Test vehicle axle load, kN

This calculation can be made for each of the deflections run on the section of road.

It is also possible to estimate the life of a flexible pavement section based on the design spring deflection. If the deflection is run using an 80 kN (9-ton) axle load, 62.3 and 44.5 kN (7 and 5 ton) deflections for the same section of road can be obtained by multiplying 7/9 and 5/9, respectively. Equation 2 is the design equation presently used to predict pavement life based on the Benkelman beam deflection.

$$\text{Log } \Sigma N18 = 11.06 - 3.25\, D_S \qquad (2)$$

Using the 80, 62.3 and 44.5 kN (9, 7 and 5 ton) deflections, it is possible to calculate a $\Sigma N18$ value for each of the load restrictions. The assumption in each case would be that the maximum deflection represents a situation where the load would be restricted to 80, 62.3 and 44.5 kN (9, 7 and 5 ton) during the critical spring period.

Table 13 shows the solution to the performance equation for various design spring deflections. As would be expected, if the road is restricted to a lower load during the critical spring period, it will theoretically be able to carry a greater number of total equivalent 80 kN (18,000 lb) single axle loads.

By comparing the number of $\Sigma N18$ predicted from the deflection tests with the number of years to accumulate that level of traffic from the previous calculations, the number of years of life to a serviceability level of 2.5 for that section of road can be estimated.

A worksheet has been developed which can be used to summarize the calculations for design spring deflections, allowable tonnages and estimated road life. The worksheet is set up to use each deflection measured. By using this procedure, the variation in pavement strength in terms of tonnage and predicted life can be observed. It is also possible to calculate the average and standard deviations of the deflections in 1.6 km (1 mi.) and calculate tonnages and road life for an average, plus two standard deviation values. This could also be done for the calculated tonnages within each 1.6 km (1 mi.). It is suggested that the latter procedure be used because it would then be possible to see what areas within the 1.6 km (1 mi.) are low in strength. It may be possible to upgrade the whole section by strengthening relatively short segments of the roadway, as shown by the example in Appendix 1.

Summary of Conditions

So far in this presentation, procedures have been summarized for determining the surface condition, calculating the traffic factor in terms of equivalent 80 kN (18,000 lb) single axle loads, determining the rideability in terms of serviceability rating of a pavement section, and estimating pavement life or strength using the pavement structure and a direct measure of strength of a pavement. Table 14 is an example of how this information might be summarized. An attempt has been made to put as much information as possible on one sheet of paper for a given pavement section. A brief discussion of how to fill out this table follows.

Under the heading of General description of the pavement section, the approximate date at which the evaluation is being done, year the road was constructed and the year it was overlaid are entered.

Under the Structure, the type of surface base and subbase are listed along with the thickness of each. These can be obtained from either records in the office or by measuring with borings in the field. The granular equivalent factors are obtained using Table 9 as a guide. The granular equivalent for each of the layers is then calculated by multiplying the thickness by the respective factor. The total granular equivalent is calculated by adding up the values for each of the layers. The embankment R-value can be either obtained by running an R-value test on the soil in the lab or by estimating the R-value using either the AASHTO soil classification or a textural classification from Table 8.

The Traffic Factors listed are, first, the AADT which can be obtained, as indicated in the traffic presentation, either from a flow map or by making a 16 hour count. The speed of the section of road is the speed limit. The equivalent loads in terms of 80 kN (18,000 lb) single axle loads should be determined since construction or the last structural overlay. This can be done using the techniques and the calculation chart from the traffic presentation. The traffic to 20 years of age or any other age can be obtained using those procedures. The road conditions are defined using the present serviceability rating which is obtained either with the PCA Roadmeter or a panel using the procedures outlined.

The Surface Conditions are those obtained with the surface condition rating scheme which has been summarized.

The rut depth can be determined using either the A-frame or by running a stringline across the road and measuring the depressions in the wheel path.

The Strength and Life Predictions of Table 14 summarizes the two methods suggested for estimating the years of life with the existing pavement section. The first part uses the structure and the embankment strength to determine the first $\Sigma N18$ that this structure could withstand according to the present Mn/DOT Design Chart. This can be read directly from the chart when the granular equivalent and the R-value of the embankment are either measured or estimated. To determine the number of years to accumulate this $\Sigma N18$, a table or plot of the predicted accumulation of $\Sigma N18$ can be used which compares the predicted number of loads to 2.5 serviceability level with the accumulation predicted with time. This can be obtained using the calculations from the traffic presentation. In the next part of Table 14, Benkelman beam deflection information is used to predict the life of the pavement, again without any structural overlay or improvement. The $\Sigma N18$ predicted in this manner is the $\Sigma N18$ for the total life of the pavement. Therefore, if the pavement is presently 10 years old and its total life is estimated to be 22 years, it can be assumed there is 12 years left before the PSR will drop to 2.5. The 80, 62.3 and 44.5 kN (9, 7 and 5 ton) deflections are calculated as given in the fourth presentation. The $\Sigma N18$ values will be greater for the 44.5 kN (5 ton) deflection than for the 80 kN (9 ton) deflection because the lower deflection will result in a longer predicted life. The years to accumulate this traffic then can be obtained in the same way as for the prediction of life for the structure by looking at the relationship of the accumulation of traffic with years under the traffic level using the traffic calculations.

References

1. Eugene L. Skok, Jr., "Pavement Management System Seminars." Part I, "Surface Condition Rating System," 1975. Part II, "Calculation of Equivalent 18,000-Pound Single Axle Loads," 1975. Part III, "Determination of the Rideability of a Pavement Section," 1976. Part IV, "Determination and Use of Pavement Strength," 1976. Mn/DOT Investigation No. 645, "Research Implementation."
2. Eugene L. Skok, Jr., and Miles S. Kersten. Criteria for Sealing or Other Surface Maintenance on Bituminous Roads. Proceedings, First International Conference on Low Volume Roads, Transportation Research Board, Special Report 16, 1976.
3. P. C. Hughes. Development of a Rating System to Determine the Need for Resurfacing Pavements. Final Report, Minnesota Department of Highways, Investigation No. 189, 1971.

Appendix 1

Examples of several uses of pavement evaluation information.

Washington County - Spring Road Restrictions

A pavement evaluation implementation "pilot project" is under way, conducted by the Physical Research Unit of Mn/DOT. This "pilot project" concerns itself with some of the pavement attributes described in this paper and shows some of the benefits of measuring these attributes.

The pavement strength, as measured by the Benkelman beam, is of particular interest in Washington County. Benkelman beam measurements were taken at 0.16 km (0.1 mi.) intervals on about 200 km (125 mi.) of county collector routes.

In the 200 km (125 mi.) of road, there were 61 segments with different structure or traffic levels; of the 61 sections, 43 sections were posted with a spring restriction and 31 were tested with the Benkelman beam. Of the 31 sections, 14 62-kN (7-ton) sections were tested resulting in them all being increased to all season 80-kN (9-ton) roads, three sections were 53-kN (6-ton) and one was changed to a 62 kN (7-ton) road. Fourteen 44 kN (5-ton) sections were tested resulting in changing 7 to an 80 kN (9-ton) road, 3 to a 62 kN (7-ton) road and 4 remaining at 44 kN (5-ton). Traffic levels on all of these sections are low, so there is no danger of a reduced fatigue life because of the increased axle loads.

Clay County - Design Evaluation

Clay County, a participant in the project used the results to evaluate the effectiveness of their design for a 62 kN (7-ton) road. A portion of County State Aid Highway (C.S.A.H.) 10 was constructed in 1972 and evaluated as part of the "pilot project" in 1976. An evaluation of the pavement by Benkelman beam showed that the road was 80 kN (9-ton) over portions having a sand loam subgrade and 62 kN (7-ton) over a clay loam subgrade. The clay loam subgrade exists only on about 20 percent of the section, so the section could be upgraded to an 80 kN (9-ton) road by adding additional structure over the clay loam portion of the road. An adjoining segment of C.S.A.H. 10 was programmed for construction after the evaluation. The evaluation showed the predominant subgrade to be clay loam, indicating the design used would result in a 62 kN (7-ton) road. It also gives the county engineer the information to do a cost/benefit analysis for an 80 kN (9-ton) design based on the traffic volumes and loadings on C.S.A.H. 10.

Chisago County - Pavement Evaluation of a Heavy Move Route

An electrical power utility company (NSP) applied for a permit to move 4 transformers from a railroad siding to a substation site. Each of these transformers and the vehicle that would move them would have a gross weight of about 2000 kN (450,000 lb). As a condition of the permit, the move route was evaluated for rideability, strength with the Benkelman beam, and surface condition before and after the transformers were moved. The permit had a payment schedule to refund the county for any measurable loss of service from the road due to the moves.

Because of the vehicle used to move the transformers, there was no measurable loss of service. The vehicle was a trailer with 96 tires suspended hydraulically so that the downward force on each tire would remain constant and equal to the rest.

Tables

Table 1. Surface condition rating form.

Date ____________________

Job Description ______________________________ Surface Sealed Before ☐ Yes ☐ No

	GEN. STR. CONDITION	SURFACE WEAR	WEATHERING	SKID RESISTANCE	UNIFORMITY	CRACK CONDITION: OPENING	CRACK CONDITION: ABRASION	CRACK CONDITION: MULT.
		☐ Excess Asphalt		Skid Number				
5	Good	None	None	Coarse	Good	Hairline	None	None
				Good		1/16		
				Gritty				
4	Long Crk.	Slight	Slight	Coarse	Strkd.		Slight	Slight
				Fair		1/8		
				Gritty				
3	Map Crk.	Moderate	Moderate		Cr. Fill.		Moderate	Moderate
				Agg. Sl. Pol.		1/4		
2	Allig Crk.	Severe	Severe	Agg. Pol.	Blotchy		Severe	Severe
						1/2		
1	Eros.	Abrasion	Erosion	Bleeding	Non Unif.	> 1/2	Abrasion	Erosion

Table 2. Calculation sheet for equivalent 18,000-lb. axle loads.

Date ______________

Road Location ______________________________ **No. Lanes** __________ **Design Lane** ______________

Design Lane AADT; ____________________ **Map** ☐ **Count** ☐ **Other** ______________________

Vehicle Distribution; Assumed ☐ **Manual Count** ☐ **Machine Count** ☐ **Other** ______________________

1 Vehicle Type	2 16 hour Count	3 Adjustment Factor	4 Seasonally Adj. No.	5 Season. Adj. Percent	6 X Design Lane ADT	7 Design Lane Distribution	8 X N18 Factor	9 Design Lane Daily N18
1					X		X	
2					X		X	
3					X		X	
4					X		X	
5					X		X	
6					X		X	
7					X		X	
8					X		X	
9					X		X	
10					X		X	
Totals								

Design Number Years ______________

Percent Growth ______________

Design ΣN18 = 365 x (Daily N18) x (Time Growth Factor)

= 365 (   ) x (   ) = ☐

Table 3. Vehicle type definitions for equivalent load calculation.

Vehicle Type Number	Description
1	Passenger Cars
2	Panel and Pickups (under one ton)
3	Single Unit; 2-axle, 4-tire
4	Single Unit; 2-axle, 6-tire
5	Single Unit; 3-axle
6	Tractor Semitrailer Combination; 3-axle
7	Tractor Semitrailer Combination; 4-axle
8	Tractor Semitrailer Combination; 5-axle
9	Tractor Semitrailer Combination; 6-axle
10	Trucks and Trailers Combinations plus Buses

Table 4. Seasonal adjustment factors for vehicle types.

	Vehicle Type						
Data Taken	1 - 3	4	5	6	7	8 - 9	10
Jan.-April	1.45	0.81	1.68	0.88	0.87	1.01	0.95
May-August	0.96	0.78	0.76	0.77	0.73	0.96	0.90
Sept.-Dec.	1.27	0.78	0.92	0.73	0.91	0.95	1.07

Table 5. Assumed percent distributions.

Vehicle Type	62 kN (7-ton)	80 kN (9-ton)
1	76.5	78.1
2	15.2	10.0
3	2.0	1.4
4	3.7	3.9
5	1.0	1.3
6	0.1	0.3
7	0.1	0.5
8	0.5	3.0
9	----	----
10	0.9	1.5

Table 6. Average N18 factor by vehicle type.

	Load Limit	
Vehicle Type	62 kN (7-ton)	80 kN (9-ton)
1	0.0004	0.0004
2	0.007	0.007
3	0.01	0.01
4	0.17	0.19
5	0.55	0.48
6	0.37	0.60
7	0.43	0.84
8	1.00	1.50
9	----	----
10	0.33	0.33

Table 7. Time growth factors for 10 and 20 years.

Annual Growth %	10 Years	20 Years
0	10.00	20.00
0.5[a]	10.23	20.98
1.0	10.46	22.02
1.5	10.70	23.12
2.0	10.95	24.30
2.5	11.20	25.54
3.0	11.46	26.87
3.5[b]	11.73	28.28
4.0	12.01	29.78
4.5	12.29	31.37
5.0	12.58	33.07
5.5	12.88	34.87
6.0	13.18	36.79

[a]Suggested annual growth for 62 kN (7-ton) roads.
[b]Suggested annual growth for 80 kN (9-ton) roads.

Table 8. Stabilometer R-values by soil type.[a]

AASHO Soil Type	Textural	Assumed R-Value	Comments
A-1	Sands Gravels	75	Excellent confidence in using assumed value.
A-1-b	Sands Sandy Loams (nonplastic)	70	If percent passing No. 200 sieve is 15 to 25 %, R-value may be as low as 25. In such cases, it is highly desirable to obtain laboratory R-values.
A-2-4 & A-2-6	Sandy Loams (non-plastic, slightly plastic, or plastic.)	30 (70 for LS and LFS)	Loamy Sands and Loamy Fine Sands commonly have R-value of 70. Laboratory R-values range from 10-80 for the entire A-2 classification. It is highly desirable to obtain laboratory R-values for the Sandy Loams. See Table 11 for sampling frequency.
A-3	Fine Sands	70	Excellent confidence in using assumed value.
A-4	Sandy Loams (plastic), Silt Loams, Silty Clay Loams, Loams, Clay Loams, Sandy Clay Loams	20	Laboratory R-value range from 10 to 75. It is highly desirable to obtain laboratory R-values. See Table 11 for sampling frequency.
A-6	Clay Loams, Clays, Silty Clay Loams	12	Laboratory R-values commonly occur between 8 and 20.
A-7-5	Clays, Silty	12	Data available are limited.
A-7-6	Clays	10	Laboratory R-values commonly occur between 6 and 18.

[a]Based on data collected by MHD through 1974.

Note: In using the above assumed R-values for flexible pavement design it is essential that the subgrade be constructed of uniform soil at a moisture content and density in accordance with Mn/DOT Spec. 2105. To minimize frost heaving and thaw weakening it is also essential that finished grade elevation be placed an adequate distance above the water table. This distance should be at least equal to the depth of frost penetration. In the case of silty soils in the distance could be significantly greater.

Table 9. Granular equivalent (G.E.) factors.

All bituminous and aggregate courses are converted to an equivalent thickness of Class 6 Aggregate Base (denoted as granular equivalent = G.E.) using factors listed below.

Material	Specification	G.E. Factors
Plant-mix surface	2341, 2361	2.25
Plant-mix surface	2331	2.00
Plant-mix binder	2331	2.00
Plant-mix base	2331	2.00
Road-mix surface	2321	1.50
Road-mix base	2321	1.50
Bituminous treatment base	(Rich) 2204	1.50
Bituminous treatment base	(Lean) 2204	1.25
Aggregate base	(Cl. 5, Cl. 6) 3138	1.00
Aggregate base	(Cl. 3, Cl. 4) 3138	0.75
Selected granular material		0.50[a]

[a]May be used in design when so approved by central office Soils Section.

Note: Where the subgrade consists of granular material the district materials and/or soils engineer may recommend the treating of the upper portion of the selected granular material with 2.5 cm (1 in.) or 5.1 cm (2 in.) of stabilizing aggregate (Spec. 3149.2C) or treating the upper 7.6 cm (3 in.) with 0.36 liter/m^3/cm (0.2 gal./sq. yd./in.) of asphalt emulsion, SS-1.

Table 10. Temperature correction to 26.7°C (80°F) for Benkelman beam deflections.

Range of Deflection in mm ≤ BB <	Temperature in Degrees Celsius				
	T < 1.6	1.6 ≤ T < 7.2	7.2 ≤ T < 12.8	12.8 ≤ T < 18.3	18.3 ≤ T < 23.9
.000 < .254	.127	.102	.076	.051	.025
.254 < .508	.178	.152	.102	.076	.025
.508 < 1.016	.254	.203	.152	.102	.051
1.016 < 1.270	.305	.254	.178	.127	.051
1.270 < 1.524	.381	.305	.229	.152	.076

Note: 1 mm = .04 in.

Table 11. Benkelman beam deflection ratio table.

Deflection rations to approximate critical spring deflections from deflections taken during other nonfrozen times of the year for:

Plastic Embankments

Asphalt Surface Thickness	Date of Test				
Conventional Construction	Aug.-Sept.	July	June	May 16-May 31	May 1-May 15
≤ 8.9 cm	1.73	1.64	1.52	1.32	1.14
> 8.9 cm ≤ 14 cm	1.68	1.54	1.40	1.24	1.14
> 14 cm ≤ 20.3 cm	1.49	1.28	1.25	1.25	1.17
> 20.3 cm	1.37	1.16	1.14	1.18	1.13
Full-Depth Construction					
> 20.3	1.45	1.12	1.13	1.16	1.12

Semi-Plastic Embankments (L, Sil, and sl. pl. SL)

	Date of Test				
Asphalt Surface Thickness	Aug.-Sept.	July	June	May 16-May 31	May1-May 15
≤ 12.7 cm	1.46	1.52	1.45	1.35	1.16
> 12.7 cm	1.68	1.56	1.48	1.40	1.29

Non-Plastic Embankments (S, S & G, FS, and LFS)

	Date of Test				
Asphalt Surface Thickness	Aug.-Sept.	July	June	May 16-May 31	May 1-May 15
≤ 5.1 cm	1.88	1.83	1.76	1.41	1.30
> 5.1 cm ≤ 14 cm	1.48	1.57	1.50	1.36	1.21
> 14 cm ≤ 20.3 cm	1.10	1.05	.99	1.02	1.00

Note: Critical deflections correspond to maximum deflections which occur in the spring, during which the pavement is most likely to be damaged by heavy loads. This ratio table is based on a continuous ten year record (1964 to 1973) of measured rebound deflections taken throughout the year on various Minnesota pavements.

Note: 1 cm = 0.4 in.

Table 12. Allowable spring deflections.

Traffic	Two-way	HCADT[a]	<50	50-100	100-150	>150
	Two-way	AADT[b]	<500	500-1000	1000-3000	>3000
Bituminous Surface Thickness			Allowable Deflection, cm			
Less than 7.62 cm			0.191	0.178	0.154	0.114
7.62 cm to 15.24 cm			0.165	0.152	0.127	0.102
Greater than 15.24 cm			0.140	0.127	0.102	0.089

[a]HCADT - Heavy commercial average daily traffic volume (excludes passenger cars and 4-tired trucks).
[b]Use AADT only when HCADT is not known.

Note: 1 cm = 0.4 in.

Table 13. Solution to performance equation predicting equivalent loads to PSR = 2.50.

Deflection, mm	ΣN18	Deflection, mm	ΣN18
0.508	6,800,000	1.905	92,500
0.635	3,300,000	2.032	75,000
0.762	1,800,000	2.159	61,600
0.889	1,100,000	2.286	51,100
1.016	710,000	2.413	43,000
1.143	490,000	2.540	36,300
1.270	345,000	2.667	31,000
1.397	253,000	2.794	26,600
1.524	191,000	2.921	23,100
1.651	147,000	3.048	20,100
1.778	116,000	3.175	17,600

Equation: $\log \Sigma N18 = 5.88 - 3.25 \log D_s$

Where: $\Sigma N18$ = Equivalent 80 kN (18,000 lb) single axle loads.
D_s = Design Spring Deflection, mm.

Note: 1 mm = 0.04 in.

Table 14. Summary of pavement conditions.

GENERAL

Location ________________ Date ________

Year Constructed ________________ Last Overlaid ________

STRUCTURE Type	Thickness in	G.E. Factor	G.E.
Surface, (D_1)	______	______	______
Base, (D_2)	______	______	______
Subbase, (D_3)	______	______	______
TOTALS -		-	G.E. = ______

Embankment R-Value ______ Laboratory / ______ Estimated

TRAFFIC

AADT ________

Speed ________

Equivalent Loads (ΣN18)

Since last O.L. or construction ________

Future

5 yr. ______ 10 yr. ______ 20 yr. ______

CONDITIONS

PSR ______ Roadmeter / ______ Panel

Uniformity ________

Surface Condition

Structural ________

Surface Wear ______ Abrasion / ______ Bleeding

Weathering ________

Skid Resistance ______ Number / ______ Rating

Crack Conditions

Opening ________

Abrasion ________

Mult. ________

Rut Depth, in. ______

STRENGTH AND LIFE PREDICTIONS

Structure

N18 from Design Chart ______

Years to accumulate
Assuming O.L. ________

Recommended Action ________

____

Est. Cost ________________

Benkelman Beam
Deflection Predictions

Spring	Defl. (.001 in.)	ΣN18	Yrs.
9 T			
7 T			
5 T			

Present Restricted Tonnage based on Mn/DOT 603 Procedure
________ tons

GEOTECHNICAL ASPECTS OF LOW VOLUME ROAD DESIGN AND CONSTRUCTION IN NORTHEASTERN THAILAND

TEERACHARTI RUENKRAIRERGSA, Department of Highways, Thailand.

Northeastern Thailand occupies about one-third of the total area of the kingdom, and this region is physiographically called the Khorat Plateau. The most available local materials in this area are lateritic soil, gravel and silty sand. Road construction in this vicinity encounters with the problem of material deficiency, especially crushed rock for base and surface courses. As the traffic volume of most routes is rather low, in order to help accelerating development in this region the approach of low cost road should be introduced to construct the road network in the Khorat Plateau. In order to satisfy this approach, the uses of the local material as base course are recommended. The method of cement stabilization with local materials are strongly emphasized to replace crushed rock. Examples of cement stabilized roads in Thailand with their performance are shown.

Introduction

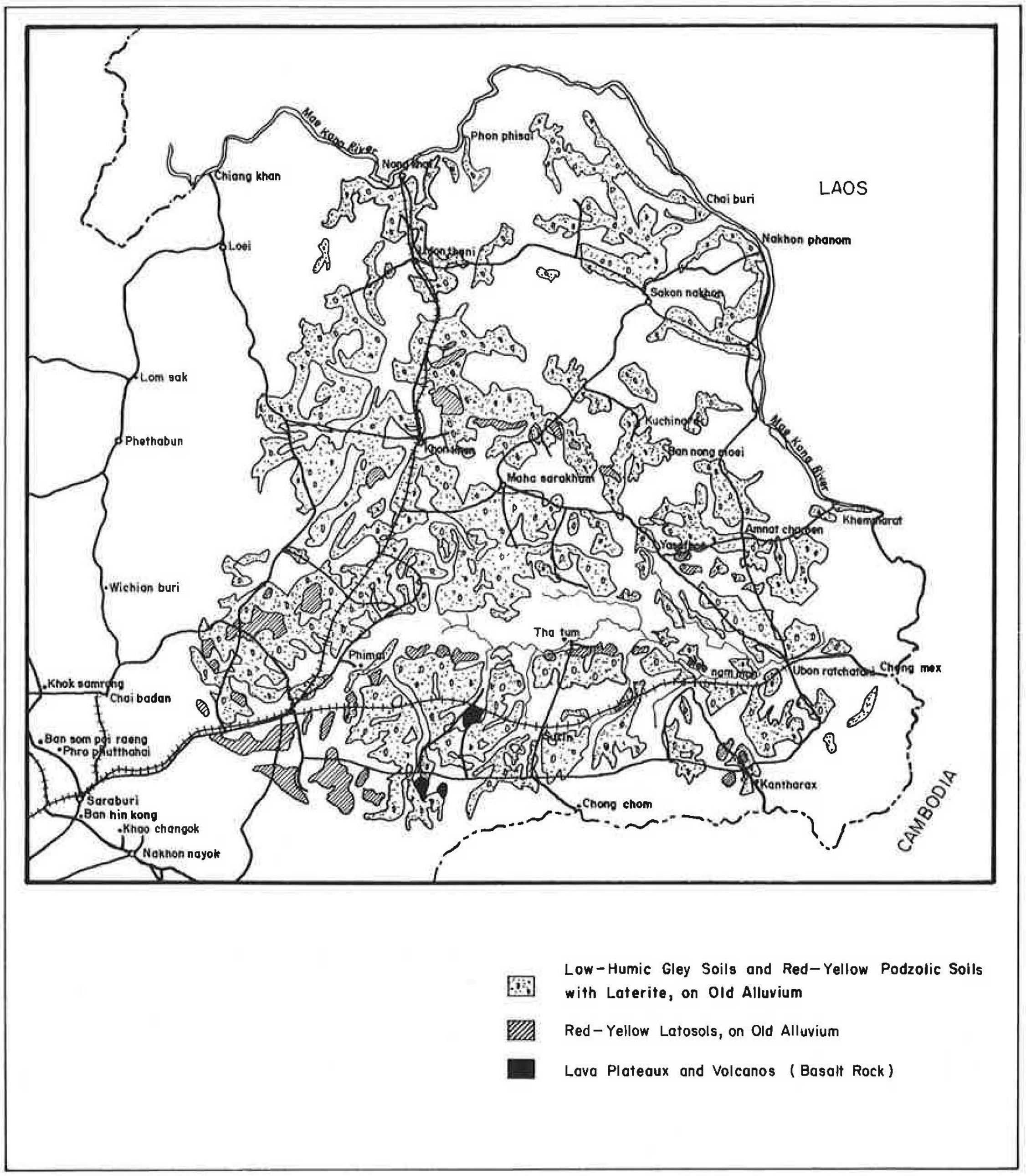
Northeastern Thailand occupies about one-third of the total area of the kingdom, and this region is physiographically called the Khorat Plateau or the Northeast Plateau for the relatively flat elevated plain. There are about 80,000 km of unpaved road in Thailand. Out of this, there are about 5,000 km under the responsibility of the Department of Highways, and more than 50 percent is in this plateau. Every year the mileages of the unpaved road under the responsibility of the Department of Highways increase. Traffic volume of these routes is generally low to medium, and most of them are less than 500 vpd. It is the policy of the Thai Government to build up the road network linking the nearby villages to induce communication between the local people and to raise the standard of living of the dwellers in the remoted areas. Many routes are built because of the political and military influences. So it is hoping that in the near future quite a few mileages of road are going to be constructed in this region. It is interesting to analyze how these roads are built, and what pavement type will be suitable for the developing countries that most of them have the financial problem, and they have to spend money in other phases of development too, not only to construct the road. Northeast Plateau has encountered the problem of material deficiency for years, especially crushed rock for base and surface courses. Since the time of an oil crisis the long haul distance of crushed rock from the remoted quarries has made the cost of material go up considerably. For the relatively low traffic volume of most routes and for the problem of material shortage associated with the financial status of the country, the approach of low cost road and staged construction could be significantly applied to the disign and construction of the road network in this vicinity. According to this approach, the local available material and manpower must be utilized as much as possible. The most abundant local materials are lateritic soils, silty and sandy soils and terrace gravel which easily found everywhere in the Khorat Plateau. This paper describes about some geotechnical aspects such as some properties and the suitability of employing such materials in low volume road design and construction. The most suitable method of stabilization is also suggested. Conclusions and recommendations are drawn on the basis of the local experience, research conducted in this region, and the results of regional international seminars in Southeast Asia.

Geology, Parent Rock and Climate

Rocks in the Northeast Plateau are mainly arenaceous rocks (clastic sediments of sand grain size) which are either subhorizontal or very gently folded. Igneous rocks are relatively rare. The modified geologic sketchmap is shown in Figure 1 (1). Sandstone is an important parent rock associated with sparsely formed basalt plugs, alluvial deposits and limestone. The surficial materials of the Northeast Plateau are silty and sandy soils of about 7 meters in thickness. The surface drainage of the area is dominant. Thus, even though flooding commonly occurs in some places in the rainy season, the soil is dried out rapidly because water will seep through the surficial soil of relatively high permeability.

Thailand has a savana-typed climate influenced by seasonal monsoons. There are three major seasons as the dry, the hot and the rainy seasons. The annual rainfall in most of the Northeast varies from 1,100 to 1,500 mm with the average of about 1,300 mm. Rainfall increases to 2,000 mm annually near the borders between Thailand and Laos, and Thailand and Cambodia. The average temperature is about 27° C.

Figure 1 Geologic Map of Northeastern Thailand (1)



Local Highway Materials and Material Improvement

Various local highway materials available in the Northeast Plateau are laterites, lateritic soil, silty and sandy soils, terrace gravel, basalt and limestone. Lateritic soil and terrace gravel from some borrow pits oould be directly employed as base course of the paved feeder road. However, some sources have to be stabilized with cement to attain the proper strength for serving the mentioned purpose. Distribution, properties and utilization of the abovementioned materials are going to be described in more or less detail in the following sections.

Laterites and Lateritic Soils

General Characteristics. Laterites and lateritic soils are extensively used in road construction in Thailand for many decades. Sources of these types of soil are found widely in the Northeast Plateau. This is due to the suitability of the climatic conditions, the good drainage of the area, and other environmental factors inducing the formation of lateritic soils. Literatures concerning the occurrence of lateritic soils could be found out elsewhere (2,3). The modified geologic map showing the probable areas of lateritic soil formation in the Northeast Plateau is illustrated in Figure 1.

Lateritic soil is generally used as subbase material for the primary and secondary highways. However, lateritic soils from some particular sources have low plasticity, durable particles, and high CBR

Figure 2. Relationships Between Compressive Strength and Carifornia Bearing Ratio for Soil-Cement Specimens (11)

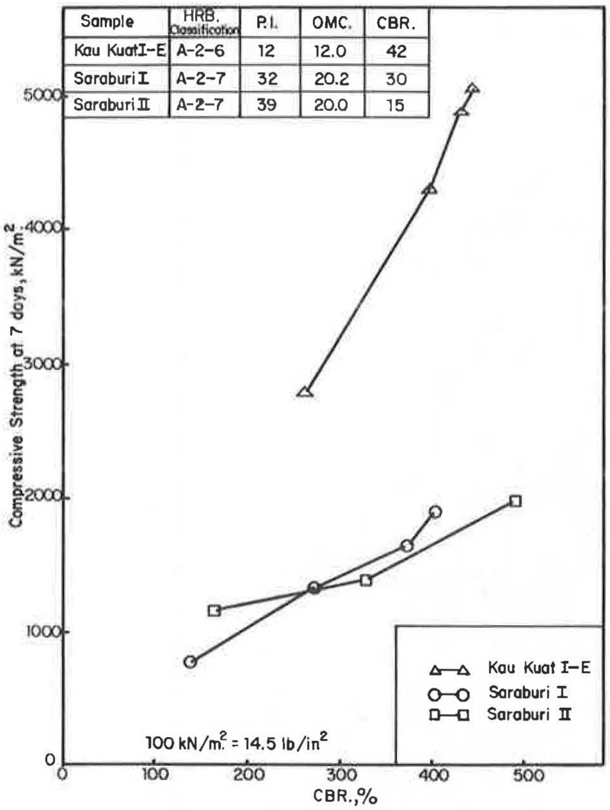
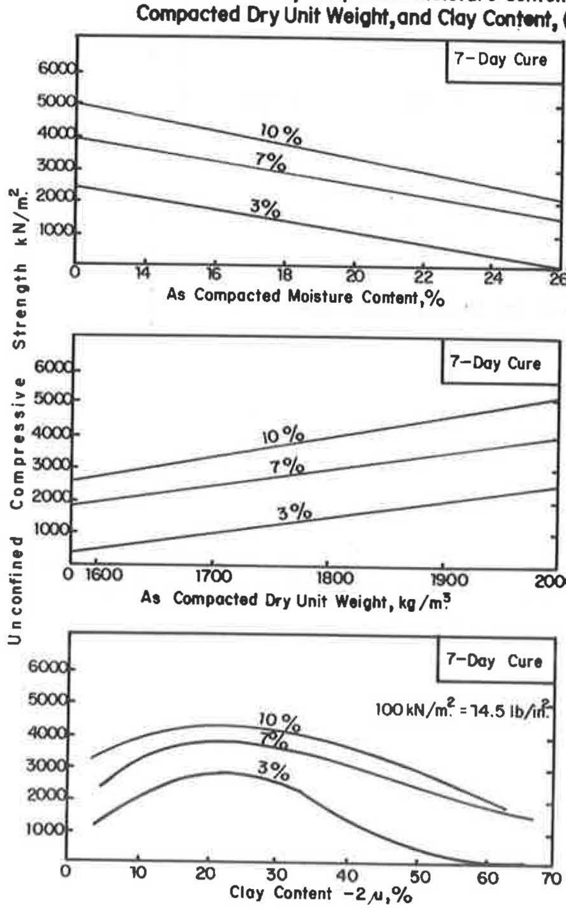


Figure 3. Variations of Unconfined Compressive Strength with Cement Content, Compacted Moisture Content Compacted Dry Unit Weight, and Clay Content, (13)



value, so they could be employed as base course of the paved low volume road. In many feeder or access roads, they are also used as soil-aggregate typed surface properly. Laterites forming as a massive structure are not suitable for employing in the pavement structure, even though their bearing value is very high. They can be properly used as the riprap instead of rock which is deficient in this area. According to researches conducted at the Thai Highway Department, most lateritic soils in Thailand have wide variations in CBR from 7 to 60, depending on grain size distribution, plasticity, durability, and irregularity of the grains. The experience of the Thai Highway Department Laboratory showed that unconfined compressive strength of lateritic soil specimen compacted at optimum moisture content in the Proctor mold varies from 275 to 1000 kN/m^2 (40 to 150 lb/in^2) which is not considered to be low (4).

Data on the triaxial compression test for the strength parameters C and Ø are very limited. MAHMOOD and MOH (5), reported the effective strength parameters of lateritic soil with PI of 11 and compacted at optimum moisture content are as C=48 kN/m^2 (7 lb/in^2) and Ø = 34°. REMILLON (6) recommended a minimum cohesion of 48 kN/m^2 (7 lb/in^2) as a limiting value and over 70 kN/m^2 (10 lb/in^2) for acceptable lateritic soil.

Classification of Lateritic Soil. Quite a few classification systems have been specifically developed for lateritic soils (2, 3). These systems are generally based on agricultural, chemical and mineralogical factors for which the parameter determination is rather complicated and time consuming. Systems of engineering soil calssification such as AASHO, Unified Soil, and U.S.Bureau of Soils, all based on the data of grain size distribution and Atterberg limits. However, these index properties changed with treatment during the test and methods of preparation as demonstrated by many investigators (7, 8, 9, 10). Described properties affecting the soil classification systems which have proved so successful for soil from temperate regions are not readily applicable to lateritic soils.

VALLERGA, VAN TIL and RANANAND (10) tried to modify the existing soil classification system by introducing some parameters that reflect the nature of lateritic soils other than grain size distribution and Atterberg limits. After conducting research on lateritic soils in Thailand, VALLERGA, VAN TIL, and RANANAND (10) proposed the Extended Unified Soil Classification System to incorporate the durability characteristics of gravel and sand to the original system.

Stabilization of Lateritic Soil. Stabilization of Thailand lateritic soils was studied in the past by many investigators (11, 12, 13, 14). JONES and YIMSIRIKUL (11) made an experiment on cement stabilization of lateritic soils with PI in the range of 12 to 39. It was reported that CBR value increased considerably to more than 100 after adding 3 percent cement. Both unconfined compressive strength and CBR values will be higher for greater compactive effort. The relationship between unconfined compressive stength and CBR was shown in Figure 2.

Satisfactory results were reported from the laboratory investigation by MOH, CHIN, and NG (12) on samples with PI range from 11 to 19, while textural composition is extremely variable. It is found that after stabilizing with 4 to 7 percent cement all the samples have unconfined compressive strength more than 1725 kN/m^2 (250 lb/in^2) which is

Table 1 Some Soil-Cement Roads in Thailand

Route No.	Km.	Year Constructed	Age Years	Pavement Structure, mm.			Performance
				Subbase	Base[2]	Surface	
23	120-187	1969	9	150 SA[3]	150	DBST[4]	Good riding quality, low deflection
	187-212	1968	10	470 SA	180	DBST	" " "
24	4-9	1966	12	100 L[5]	150	DBST	Slightly rolling, patching
201	0-64	1967	11	150 SA	150	DBST	—
202[6]	12-66	1969	9	150 L	150	40 AC[8]	—
	66-85	1970	8	150 SA	150	PM[9]	—
202[7]	0-53	1970	8	150 SA	150	DBST	Reflected crack, good riding quality, and low
	53-125	1970	8	150 SA	150	50 AC	deflection
205	244-321	1970	8	150 SA	150	DBST	—
	321-340	1968	10	150 L	150	DBST	—
	340-403	1967	11	150 SA	150	DBST	—
212	57-138	1969	9	100 SA	150	DBST	Good riding quality, mostly uncracked
213	0-84	1968	10	100 SA	150	DBST	
214	0-55	1969	9	150 SA	150	40 AC	Good pavement condition, some cracks and
	55-74	1967	11	100 SA	150	40 AC	rutting
217	5-59	1969	9	150 SA	150	40 AC	Good condition, low deflection, much cracked
221	0-64	1970	8	150 L	150	DBST	Good condition, rutting and crack

1. Data from Maintenance Division
2. Base is soil – cement
3. SA is soil-aggregate
4. DBST is double surface treatment
5. L is lateritic soil
6. Route Chaiyaphum – Bua Yai
7. Route Amnatchareon – Kemmarat
8. AC is asphaltic concrete
9. PM is penetration macadam

the criterion suggested by the British Road Research Laboratory from his experience in Africa (15). The importance of proper compaction is strongly emphasized by the fact that a small decrease in percent compaction resulted in considerable reduction of compressive strength.

WOO (13) made a detailed investigation on cement stabilization of lateritic soils extensively sampling from Thailand. The samples employed in this investigation varied from sandy loam (A-2-4) to clay (A-7) with the plasticity indices of 9 to 35. After interpreting the Woo's data, the variations in unconfined compressive strength with molded water content, compacted dry unit weight, and percentage amount of clay content are shown in Figure 3. The unconfined compressive strength increases with cement content for all values of compacted dry unit weight, molded water content and clay content.

Most lateritic soils usable for soil-cement base course construction have dry density generally not less than 2000 kg/m^3 (125 lb/ft^3). According to Figure 3, at the value of dry density of 2000 kg/m^3 (125 lb/ft^3) and cement content of 3 percent, the unconfined compressive strength will be about 2500 kN/m^2 which is higher than the criterion suggested by the British Road Research Laboratory, that is 1725 kN/m^2 (250 lb/in^2). After making an extensive investigation as reported by WOO (13) and interpreting his data in Figure 3, it is clearly seen that cement requirements for lateritic soil in Thailand could be ranged from 3 to 7 percent, with an average of 5 percent. This tends to substantiate the past investigations (11, 12). However, the actual construction projects in Thailand indicated that lateritic soil in Thailand requires only 3 to 5 percent cement to attain the unconfined compressive strength not less than 1725 kN/m^2 (250 lb/in^2).

Another interesting finding interpreted from WOO's data is that the critical clay content for maximum unconfined compressive strength varying from 20 to 25 percent for the same type of clay mineral in all samples. Any lateritic soils having clay content greater or lower than this critical range the unconfined compressive strength will drop as evidenced by Figure 3.

The Siam Cement Company introduced a soil-cement road to Thailand for the first time in 1966 by constructing a test road of 5 km long using cement stabilized lateritic soil as base course. He recommended the CBR value of the soil-cement base of 120 be a criterion for low volume road with low typed surface in the Northeast. The criterion for unconfined compressive strength is the same as that suggested by the British ROAD RESEARCH LABORATORY (15), that is 1725 kN/m^2 (250 lb/in^2).

The Thai Highway Department adopted a criterion of minimum unconfined compressive strength of cement stabilized soil of not less than 1725 kN/m^2 (250 lb/in^2) like the British Road Research Laboratory. From the experience of more than 1400 km of lateritic soil-cement road constructed in the Northeast Plateau, it was found that 3 to 5 percent cement is enough to satisfy the unconfined compressive strength requirement for the soil with PI of less than 18. For highly plastic lateritic soil, 2 percent of lime was added and mixed with the soil to cut down its plasticity before applying cement. At this range of cement content, the unconfined com-

Figure 4. Map Showing Soil-Cement Routes in Northeastern Thailand

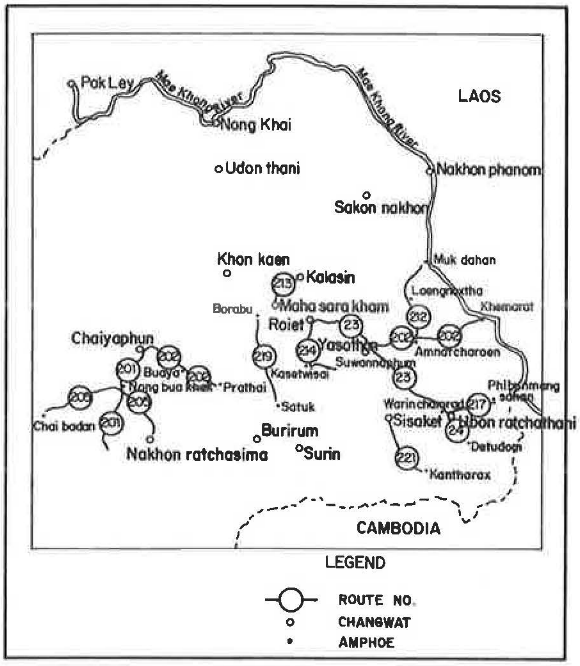
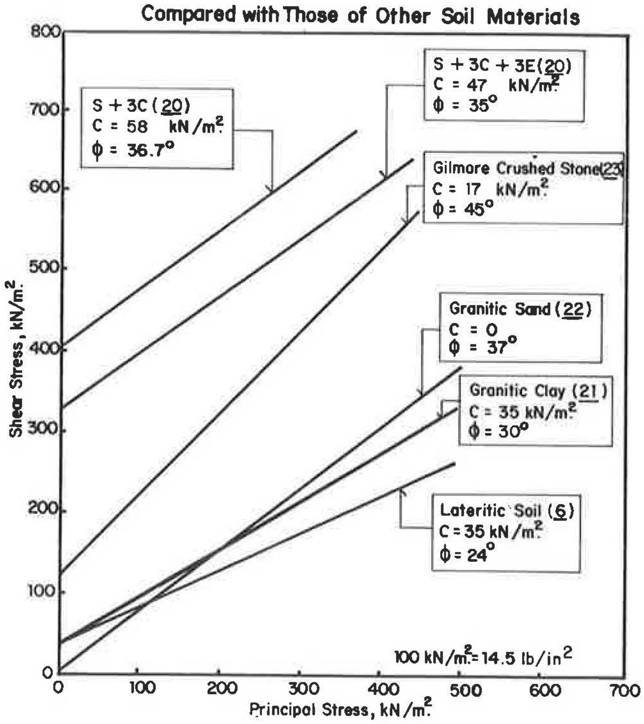


Figure 5. Strength Parameters of Stabilized Silty Sand Compared with Those of Other Soil Materials



pressive strength varied from 1725 to 2760 kN/m^2 (250 to 400 lb/in^2).

WOO and MOH (14) studied lime stabilization of eight lateritic soil samples from the Northeast Plateau. The PI are in the range of 9 to 35. It was found that most soil samples stabilized with 7 to 10 percent of hydrated lime developed more than 1725 kN/m^2 (250 lb/in^2) compressive strength after 28 days curing and one day immersion. Lime stabilization is not recommended to use in this region for its slower rate of strength development than cement. Its cost is also slightly higher than cement. Lime is generally employed as a secondary additive for cement stabilization to reduce the plasticity of highly plastic lateritic soil as stated before.

Pavement Design and Performance of Lateritic Soil - Cement Roads. In Northeastern Thailand there are about 1400 km of soil-cement roads with varying ages of 8 to 12 years. Table 1 shows the pavement structure of individual route including other data concerned such as age of service, type of surface, and etc. The route map is also shown in Figure 4. The thickness of the cement stabilized base is limited to 150 mm for every route.

The surface type of soil-cement road may be penetration macadam, double surface treatment, or asphaltic concrete. Performance of soil-cement road varied with the type of surface. The performance is evaluated on the basis of the Benkelman Beam deflection, riding quality rut depth, and roughness of road surface. After such evaluation it is found that performance of most soil-cement roads in this region is satisfactory. The plastic failure is not significant. In the future when the traffic volume increases, only additional few centimeters of high typed surface will be enough to strengthen the pavement. This is corresponding to the modified PIARC definition of low cost - low volume road that will be described in the Discussion section. So it is clear that soil-cement could replace crushed rock, and the road could serve low to medium traffic (ADT less than 500 vpd) in Northeastern Thailand. Even though the performance of soil-cement roads tends to be acceptable, but they usually encounter with a serious problem, that is the shown up reflected crack on the surface which increased much work in the maintenance program. The causes of crack were due to shrinkage of the soil-cement slab itself associated with many other factors (16). RANANAND (16) stated, on the basis of his investigation on some test roads, that reflected crack could be minimized if the surface is made penetration macadam because this type of surface could sustain larger deflection without crack than asphaltic concrete and double surface treatment.

According to the ECAFE's Report of the Seminar on Low Cost Roads and Soil Stabilization (17), it was urged to make use of the local materials in low volume road construction as much as possible in order to reduce the cost. Applying the concept of employing the local materials, staged construction approach and appropriate technique of soil stabilization together, cement stabilized lateritic soil is a suitable material to serve as base course of low volume road in Northeastern Thailand. From experience of soil-cement design and construction in this region associated with researches conducted, it is expected that the problems of reflected crack, the field control and compaction could be eliminated.

Silty and Sandy Soils

Silty and sandy soils are earth materials covering an extensive area of Thailand. Both of them are surficial soil in the Northeast Plateau. As the most available materials, they are expected to be employed as parts of the pavement structure for low volume road.

Table 2. Recommended Pavement Thickness for Low Volume Road in Thailand (25)

Unsoaked CBR. of Subgrade Compacted at OMC.	Buses and Trucks Less Than 25 Vpd.*			Buses and Trucks 26–60 Vpd.*			CBR. of Subgrade Compacted at OMC.	Buses and Trucks 61–120 Vpd.			Buses and Trucks 121–200 Vpd.		
	Surface Type – Single Surface Treatment.			Surface Type – Double Surface Treatment				Surface Type – Double Surface Treatment			Surface Type – Double Surface Treatment		
	Base CBR ≮ 60 (mm)	Subbase CBR ≮ 25 (mm.)	Selected Materials CBR ≮ 8 (mm.)	Base CBR ≮ 80 (mm.)	Subbase CBR ≮ 25 (mm.)	Selected Materials CBR ≮ 8 (mm.)		Base CBR ≮ 80 (mm.)	Subbase CBR ≮ 25 (mm.)	Selected Materials CBR ≮ 8 (mm.)	Base CBR ≮ 80 (mm.)	Subbase CBR ≮ 25 (mm.)	Selected Materials CBR ≮ 8 (mm.)
2	100	120	140	120	120	150	2	140	120	180	150	130	200
4	100	120	120	120	120	120	4	140	120	140	150	150	150
6	100	150	–	120	150	–	6	140	150	–	150	130	–
8	100	120	–	120	120	–	8	140	120	–	150	–	–

*In both directions

Silty sand, a fine grained soil with well gradation, has been used as subbase of many roads in Northeastern Thailand. As the problem of material deficiency is so critical, a pilot project was conducted using silty sand stabilized with cement and emulsified asphalt for hoping that it may be strong enough to be employed as base course of low volume road. From the result of the laboratory test it was found that the molded specimen of silty sand mixed with 5 percent emulsified asphalt, or 3 percent cement and 3 percent emulsified asphalt, had high stability enough to serve this purpose (18). The performance of the test road was found to be satisfactory (19).

RUENKRAIRERGSA and DEOPANICH (20) made a further investigation on strength characteristics of silty sand stabilized with cement, emulsified asphalt or both additives with the same proportions as in the past studies (18). The strength of stabilized material was evaluated in terms of strength parameters C and ϕ by conducting the undrained triaxial compression test on the specimens after curing for 7 days. The result is shown Figure 5. Also included in the same figure are strength parameters of some other soils obtained from the past studies namely decomposed granite, crushed gravel, and lateritic soil (5, 21, 22, 23)

From Figure 5 it is obviously seen that silty sand stabilized with cement or cement and emulsified asphalt has very high strength as evaluated from the strength parameters C and ϕ. The strength of stabilized silty sand is higher than compacted lateritic soil (5), granitic clay (21), and granitic sand (22). In the case of crushed stone, cohesion of stabilized silty sand is higher, while the angle of internal friction is lower than the Gilmore crushed stone (23). This indicated that for low pressure range (low traffic volume) stabilized silty sand could be suitably used as base course, but for higher pressure (or high traffic volume) it is better to employ crushed stone base for longer life.

Terrace Gravel

Terrace gravel is one of the most important highway materials that easily found in the Northeast Plateau. Sometimes it is formed as lateritic gravel which has thin layer of iron oxide coating around the gravel particles. Terrace gravel, like lateritic soil, is a well graded material with some quantities of fines. The plasticity index is generally lower than lateritic soil. After blending with tractor in the borrow pit, it can be directly used for subbase or base courses of low to medium traffic road. Good terrace gravel from some sources in this region may have CBR as high as 80.

Design of Pavement Structure for Low Volume Road in Thailand

The design of road pavement in Thailand is based on the CBR of subgrade and number of heavy truck of that particular route, following the method of THE ASPHALT INSTITUTE (24). As there are so many low volume roads, both paved and unpaved, in Thailand. Sometimes the division engineer has to design the pavement structure of some roads or some sections of road in his area by himself. In order to standardize the typical pavement structure for any particular soil bearing value and traffic volume in all divisions, the Department of Highways appointed a Working Group for Drafting the Most Probable Pavement Design for Low Volume Road. This Working Group has to compromise between the theoretical design following the method of THE ASPHALT INSTITUTE (24) and the local experience for the individual regional traffic and climatic condition in Thailand to set up the most probable pavement structure. Table 2 shows some pavement designs of low volume road for different subgrade soil bearing values in Thailand as compiled by POMTYEN and THUM-UMNAUYSUK (25). According to Table 2 CBR of the base required is 60 to 80. Good lateritic soil and terrace gravel in some localities have CBR values fall in this range and they can satisfy the design criterion for low volume road.

Discussion

In 1958 ECAFE organized the Seminar on Low Cost Roads and Soil Stabilization in New Delhi, India(17). The Seminar emphasized the use of local materials in road construction in order to cut down the cost of the road. In case the available material is not suitable, some methods of soil stabilization should be tried. However, during the past time, the meaning of the word "low cost" had changed dramatically. During the Sixth World Highway Conference held in Montreal, Canada, in 1970, BENNETT (26) stated that staged construction could provide a minimum standard facility which could be improved as the need increases. BENNETT also pointed out that current economic analysis methods will seldom provide justification for a low cost road. In many international seminars there are some controversies about the real meaning of the words "low cost". Many suggested to use "low volume" instead. However, the Permanent International Association of Road Congress (PIARC) compromised the argument and use the terms "low cost - low trafficked road" to satisfy both ideas. In 1976 the Indonesian Road Research Institute organized the Regional Seminar on Low Cost Roads in Bandung, Indonesia. A modified PIARC definition of low - cost roads by VANCE (27) seemed to be generally accepted by the Seminar as:

> "A low cost road is one which, having regard to conditions of climate and traffic, has been located and built to geometrical standards commensurate with future requirements, but has been constructed down to a price rather than up to a standard. It is, however, one which should be so designed, constructed and maintained that it allows for stage construction when improvement in economic conditions permits."

The objective of organizing this Seminar is to promote the low cost - low volumed road construction and the technical cooperation in road engineering to develop the Southeast Asian countries. Some conclusions pertaining to the use of local materials are reached as follows (28):

> "The need for more information about the distribution, properties and road performance of naturally occurring materials was stressed. Some possible practical steps to overcome this was spelled out as follows:
> a) identification of those materials of widespread occurrence and identification of greatest problem to be faced in recording national inventories of material;
> b) undertaking studies of these materials, such as full-scale trials, to evaluate their usage in low-cost road construction; and
> c) creation of more realistic specifications for low cost road application as the final step in the abovementioned process.
> In this way more effective use be made of local resource , perhaps the most important constideration in lowering the cost of construction."

From the conclusions above it will be clearly seen that the effective utilization of the local materials plays a great role in cost reduction of road construction in many developing countries such as those in Southeast Asia. The regional highway materials must be investigated associated with determining their properties. Lateritic soil, silty and sandy soils, and terrace gravel are major types of soil most available in the Northeast Plateau. The appropriate specifications for each must be set up specially for use with low volume roads and for local application. Today the specifications of highway materials adopted by the Department of Highways are slightly modified from those of AASHO. The modified specifications are found to limit the effective employment of local materials in the tropical areas. The modes of formation of lateritic soil and other tropical soils are not comparable to the materials in the temperate regions. So their nature will be different and then should their specifications. Realizing about this fact, the Department of Highways conducted many research projects studying about the properties and stabilization of some local materials in Thailand. These are lateritic soils, silty and sandy soils in the Northeast Plateau, granitic soil in the North and the South. The research programs of the Department of Highways tend to substantiate the conslusion of the Bandung Regional Seminar in 1976 that research needs in various phases of planning, design, and construction of low cost roads are important for the countries in the tropical region in which most of them are developing.

From the above discussion it will be seen that the effective employment of local materials, the future planning for upgrading the road to satisfy the future traffic volume, the appropriate techniques of soil stabilization and staged construction must be considered harmoniously in design and construction of low cost-low volume roads. This type of road is not designed for the present situation alone, but also for the future expansion. If not, the word "low cost" will have no meaning. So, why the UNESCO's experts emphasized in the book entitle "Low Cost Roads" that

> "----- the disign and construction of roads in developing countries have a special character." (29).

Conclusions

1. Geotechnical aspects concerning the low cost - low volume road design and construction are the effective employment of local materials, the appropriate methods of soil stabilization, and the technique of staged construction. All of these should be kept in mind when dealing with low volume road.

2. Specifications adapted from experience in the temperate countries are found to limit the effective use of local materials in the Northeast Plateau and other parts of Thailand.

3. Researches on materials and their stabilization are needed in Thailand in order that the more suitable specifications are developed on the basis of data from the local resources.

4. Major soil materials employable for base course of low volume road in Northeastern Thailand are lateritic soil, terrace gravel, and stabilized silty and sandy soils.

5. On the basis of the CBR value, lateritic soil and terrace gravel which are abundant in Northeastern Thailand could be directly employed as base course

of low volume road.

6. For fine grained or high plasticity lateritic soil, about 3 to 5 percent of cement is required to get the unconfined compressive strength of 1725 kN/m^2 (250 lb/in^2) which is adopted as a criterion for soil-cement base in Thailand. At this range of cement content the CBR will increase to more than 100.

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USE OF SOIL SURVEYS FOR PLANNING AND DESIGNING LOW VOLUME ROADS

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A method was developed to use soil surveys made under the guidelines of the National Cooperative Soil Survey to aid highway engineers in designing the most economical routes and pavement structures for low volume roads. The support value of a subgrade soil is needed in the pavement design process, but this value is not normally readily available without extensive field sampling and testing. Using detailed soil maps, a correlation was developed between the soil series shown on the maps and the California Bearing Ratio (CBR) values for those same soils. Data on soil samples and borings obtained by the Soil Conservation Service (SCS) were compared to similar data obtained by Indiana State Highway Commission engineers in order to develop estimated CBR values for 275 different soils in Indiana. The SCS soil maps and the correlation of the soil series names to estimated CBR values allow an engineer to quickly determine the value of the subgrade soil support for any desired routing of a low volume road. In addition, the SCS maps and CBR values can be used together by the engineer to determine preliminary thickness design calculations for the various routes chosen.

Four primary factors must be considered when a highway engineer begins a pavement design analysis. These factors include: (a) Traffic--the number as well as the type and weight of the vehicles; (b) Subgrade soil strength--the ability of the soil to adequately support the overlying pavement layers; (c) Material characteristics--the type and quality (relative strength) of the layers used in the pavement structure; and (d) Environmental variables--climate conditions and drainage requirements.

The goal of every highway engineer should be to design a roadway that will adequately carry, at the lowest possible cost, the traffic volumes using the pavement. This minimum cost criterion must include both initial construction costs and long-term maintenance costs.

This paper describes the use of soil surveys to determine the relative values of subgrade soil strength for a particular stretch of highway pavement. The soil survey can be used during several steps in the design process to reduce the cost of designing and constructing a roadway. The survey information can be utilized during the preliminary route selection phase to determine the choice of highway location which crosses the best subgrade soil conditions and bypasses the poorest soil areas. It can also be employed during the preliminary thickness design phase to determine the estimated structural number of the pavement cross section. Finally, it can be utilized by the contractor during the construction phase to indicate the existence of suitable borrow pits and potential problem soil areas.

Evaluating Subgrade Soil Strength

CBR Method

One of several methods available for estimating the relative strength of a subgrade soil for road construction purposes is the California Bearing Ratio (CBR) test. This method is primarily a penetration test that measures the shearing resistance of a soil (1). The procedure is fully described in ASTM D 1883 and AASHTO T 193 (2,3).

CBR tests can be conducted on in-place, undisturbed field soil samples. Such tests, however, evaluate the relative strength of the soil only at the field moisture and density conditions existing at the time of the test. Most CBR investigations, therefore, are conducted on remolded laboratory soil specimens. The laboratory procedure determines the relative strength of the soil after it has been soaked in water for 96 hours.

The lab CBR value depends on three primary factors: the soil density, the moisture content of the soil when the test specimen is prepared, and the moisture content of the soil after soaking. Since density and moisture content greatly affect the strength of the soil, the initial moisture and density values of the laboratory prepared specimen should be similar to those obtained by construction equipment in the field.

Soil Maps

Another way to estimate the CBR value of a soil is by using the detailed soil surveys published by the United States Department of Agriculture (USDA), Soil Conservation Service (SCS). For this method, a correlation is needed between the soil series names on the soil maps and the CBR value of that soil determined from a laboratory test. Such a correlation study was completed recently for 275 different soils in Indiana.

Soil Surveys

The Soil Conservation Service is an agency of the USDA charged by Congress with responsibility for soil and water conservation and proper land use (4). All soil survey work, including soil interpretations, is done by the SCS in cooperation with state agricultural experiment stations and other governmental agencies under the guidelines of the National Cooperative Soil Survey. In Indiana, the SCS works with the Purdue University Agricultural Experiment Station in making soil surveys. From the very beginning of soil surveys in 1899, they have been beneficial to land users who desire knowledge about the soils' physical and chemical properties shown in Tables 1 and 2 (5) as well as their location and extent shown in Figure 1 (6, map 11).

Soil

Soil is a natural, three-dimensional body at the earth's surface that is capable of supporting plants and has properties resulting from the integrated effect of climate and living matter acting on earthy parent material, as conditioned by relief over periods of time (7). Soils have distinct horizons or layers, as shown in Figure 2 (1, p. 19). The horizons or layers, which are approximately parallel to the surface, have distinct characteristics produced by the soil-forming processes.

An organic layer of fresh and decaying plant residue is at the surface of most mineral soils. Below this organic layer is the A horizon, formed or forming at or near the surface. It is an accumulation of humified organic matter mixed with mineral matter. The A2 horizon is mainly a residual concentration of sand and silt, which is high in resistant minerals content as a result of the loss of silicate clay, iron, aluminum, or a combination of these.

The B horizon is a layer of change between the overlying A and underlying C horizon. The B horizon is characterized by (a) the accumulation of clay, sesqui-oxides, humus, or a combination of these; and/or (b) a prismatic or blocky structure; and/or (c) redder or browner colors than those in the A horizon. The combined A and B horizons are generally called the solum, or true soil. If a soil lacks a B horizon, the A horizon is the solum.

The C horizon, excluding indurated bedrock, is little affected by soil-forming processes and does not have the same properties as the A or B horizon. The material of the C horizon may be either similar or dissimilar to that from which the solum is presumed to have formed. The R layer is consolidated rock. It commonly underlies the C horizon, but can be directly beneath either the B or A horizon.

The depth or thickness of an individual soil horizon varies within defined limits for each particular soil. Some soils, however, form in two materials. The properties of the top part of the B horizon soil can be different from the properties of the bottom part of the same horizon; each part of the soil can then have a different CBR value.

Making a Soil Survey

Soil surveys are conducted in the field by soil scientists who walk the area mapping soil landscapes (8). They take many soil samples in order to determine the soil profiles. The profiles are compared with those in other soil survey areas. The soils are then classified according to their individual properties, conforming to a uniform, nationwide procedure (7).

Soils that have similar soil profiles make up a soil series. Except for different textures in the surface layer, all soils of one soil series have major horizons that are the same in thickness, arrangement, and other characteristics. Each soil series is named for a town or geographic feature near the place where a soil of that series was first observed and mapped. All the soils in the United States having the same series name, such as Crosby or Plainfield, are essentially alike in those characteristics that affect their behavior in the undisturbed landscape.

Soils of a particular series, however, can differ in the texture of the A horizon as well as the slope or some other characteristic that affects the use of the land (9). The name of a soil phase indicates a feature that affects land use and management. For example, Crosby silt loam, 2 to 6 percent slopes, is one of several phases within the Crosby soil series.

When conducting a soil survey, the soil scientists gather soil samples for laboratory testing. Some of the data collected during the laboratory part of the investigation are shown in Tables 1 and 2 (5). Among the data available for use by highway design engineers are the Unified and AASHTO soil classifications, sieve analysis, Atterberg limits, permeability, soil reaction, shrink-swell potential, depth of the water table, depth to bedrock, and frost heave potential.

After determining the extent or area of a particular soil, the soil scientist delineates the boundaries of each soil on aerial photographs. Essentially all soil maps in the United States are drawn at a scale between 1:15840 to 1:24000 (10.16 cm or 4 inches to 6.70 cm or 2.64 inches per mile). The larger scale allows contrasting soil areas as small as 0.81 to 1.21 hectares (2 or 3 acres) to be drawn on the aerial photographs. Packets of different soils smaller in size than this, however, are not shown on the soil maps. The properties of the soils in the small, unmapped areas may be more or less favorable than the soil delineated on the map.

Detailed soil surveys have been completed for about 60 percent of the United States and approximately 65 percent of Indiana. In areas not yet surveyed, local SCS personnel are available to assist highway design engineers determine the type of soil in a particular location.

Correlation of CBR Values

ISHC Data

In conjunction with the construction of the interstate highway system across the state, the Indiana State Highway Commission (ISHC), Division of Materials and Tests, has collected many soil samples from the various soils found along the routes. Sometimes these soil specimens were taken along several possible route centerline locations in order to determine which route encountered the best soil conditions. The only way to judge field conditions was to take field soil samples, Once a particular route had been selected for a project, additional soil samples were taken to determine the type of subgroup soil along the proposed roadway. These samples were usually taken at predetermined intervals along the centerline, in some cases without regard to actual field conditions. Pockets of poor soil were thus sometimes missed during the field sampling, only to be "discovered" during construction.

A small number of collected soil samples were used to determine the CBR value of some of the soils found along the route. These values were used by ISHC design engineers to determine the required pavement thickness for individual paving projects. If several soils with different CBR numbers were determined within one project limit, generally the lowest value was used for design purposes, and all the pavement for the total length of the section was set at the same thickness. This procedure led to very conservative and costly design practices when better soil conditions (higher CBR numbers) were encountered over a significant distance within a particular project.

Data Correlation

For both preliminary route selection and preliminary pavement thickness design analysis, SCS soil survey data and soil maps can be used to estimate the CBR values of the soils along a particular roadway route. A way was needed, however, to correlate the data in the SCS soil surveys with actual laboratory CBR values for the same soils.

Several joint meetings were held between ISHC personnel, SCS soil scientists, and other interested engineers to determine if such a soil correlation could be obtained. Many hours were spent in review of ISHC information to determine exactly what data were available for each individual CBR test number, particularly (a) the exact location in the field where the sample was taken, and (b) other soil sample characteristics, such as Atterberg limits, field moisture content, field density, soil sieve analysis, and soil classification.

The SCS soil maps were then used to identify the field location when the actual soil samples had been taken. This location correlation required several months of extensive cross checking between ISHC project plans, field soil sampling notes, and the SCS soil maps. In addition, SCS personnel went back to every field site (over 162 in number), examined and classified the soil where the ISHC sample had been taken, and compared the data obtained to the ISHC CBR test information.

Some obvious errors were discovered--the CBR value for a given test site might be 3 while the soil maps would indicate an A-4 or ML soil, with an estimated CBR value of 6 to 10. Usually, however, the laboratory CBR values agreed well with the expected CBR value for a particular individual soil series name.

Considerable scatter was found in some of the data. Table 3 shows 18 actual laboratory CBR values obtained on Crosby soils from highway projects in 9 different Indiana counties across the central part of the State. The values range from a low of 2.2 to a high of 4.7. The average laboratory CBR value for this soil series is 3.60, with a standard deviation for the 18 values of 0.62. For this particular soil, an estimated CBR value of 3.0 was selected as the design value. Approximately 84 percent of all laboratory CBR test values are equal to or greater than the chosen design CBR value.

Of the 275 soils in Indiana, sufficient data (at least 8 samples of each soil) were available on about 58 primary soils to determine estimated CBR values in a manner similar to that described above for the Crosby soil series. Due to the variability of the CBR values obtained for each soil, however, and because of a very limited number of samples available for some particular soil series, a statistically based analysis could not be completed. As more data is gathered from future correlation work between ISHC soil tests and SCS soil maps, a revised and updated listing of estimated CBR values will be published for Indiana. Once the CBR numbers for the major soils were calculated, the values for the remaining soils were assigned according to similar soil properties. Nine CBR classes were used to group the 275 Indiana soils for pavement design purposes. The CBR values selected were 2 through 8, 10, and 15. A tenth CBR class CBR=0, was used to indicate those soils which are peats or mucks and are completely unsuitable as foundation soils for highways.

Table 4 shows a correlation of the estimated CBR values determined for Indiana soils with both the Unified and AASHTO soil classification systems (10). Each group of AASHTO soils is shown in the first column, with the most probable comparable Unified system soil in the second column. The typical CBR number range for each soil classification is listed in the third column, followed by the most probable soaked CBR value within the range.

Table 5 lists the estimated CBR values for Indiana soils. Some of the soils listed have been formed in two different parent materials. These soils, marked with an asterisk, can have two different values of soaked soil strength; thus the two given CBR values--the first for the upper part and the second for the lower part of the B horizon.

Words of Caution

The information shown in Tables 4 and 5 must be used with caution. The CBR values listed are valid only for Indiana soils. In addition, the estimated CBR numbers have been determined based on a limited amount of laboratory testing. No soil has only one CBR number. Depending on the density and moisture content of the soil, its CBR value can vary significantly. The numbers listed, therefore, are the most probable values expected for a particular soil.

The CBR values given for use with the soil survey maps are those for the B horizon. For road building purposes, the A horizon soil should be stripped and removed before a pavement structure is constructed on the B horizon material. In relatively flat terrain, the roadway subgrade is normally built entirely on the B horizon soil. In rolling countryside, however, roadway cut sections more than five feet deep may be encountered. Thus the C horizon soil may be used as the subgrade soil foundation. The information contained on the soil maps for this horizon is less reliable than that for other soil horizons near the surface.

The information contained on the soil survey maps should be used only for preliminary highway design purposes. It can be used to determine probable roadway route centerline soils. It can also be utilized for preliminary thickness design calculations without a detailed analysis of the subgrade soil. It must be emphasized, however, that field soil samples should be obtained, tested, and analyzed before a final pavement structural section is selected.

Summary

The subgrade soil CBR data developed for Indiana soils using SCS soil maps and laboratory CBR tests can be established for other areas through cooperation between the Soil Conservation Service (SCS) and the state highway department. It will take some effort and patience to collect and correlate the necessary information. Applied together, the CBR data and soil surveys can be used effectively during the route selection phase, preliminary pavement thickness design phase, and construction phase of a highway project.

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Table 1. Morphological[a] and estimated soil properties of Crosby soils.

ESTIMATED SOIL PROPERTIES

DEPTH (IN.)	USDA TEXTURE	UNIFIED	AASHTO	FRACT >3 IN (PCT)	PERCENT OF MATERIAL LESS THAN 3" PASSING SIEVE NO. 4	10	40	200	LIQUID LIMIT	PLASTICITY INDEX
0-11	SIL, L	CL, CL-ML	A-4, A-6	0	100	95-100	80-100	50-90	22-34	6-15
11-30	CL, SICL	CL, CH	A-6, A-7	0-3	92-99	89-97	78-93	64-76	37-55	17-31
30-60	L, CL, SL	CL, ML, CL-ML	A-4, A-6	0-3	88-94	83-89	74-87	50-64	17-30	2-14

DEPTH (IN.)	CLAY (PCT <2MM)	MOIST BULK DENSITY (G/CM3)	PERMEABILITY (IN/HR)	AVAILABLE WATER CAPACITY (IN/IN)	SOIL REACTION (PH)	SALINITY (MMHOS/CM)	SHRINK-SWELL POTENTIAL	EROSION FACTORS K	T	WIND EROD. GROUP	ORGANIC MATTER (PCT)	CORROSIVITY STEEL	CONCRETE
0-11	11-22	1.50-1.70	0.6-2.0	0.20-0.24	5.1-6.5	-	LOW	.43	3	5	1-3	HIGH	MODERATE
11-30	35-45	1.50-1.70	0.06-0.2	0.15-0.20	5.1-7.3	-	MODERATE	.43					
30-60	15-26	1.70-2.00	0.06-0.6	0.05-0.19	7.9-8.4	-	LOW	.43					

FLOODING FREQUENCY	DURATION	MONTHS	HIGH WATER TABLE DEPTH (FT)	KIND	MONTHS	CEMENTED PAN DEPTH (IN)	HARDNESS	BEDROCK DEPTH (IN)	HARDNESS	SUBSIDENCE INIT. (IN)	TOTAL (IN)	HYD GRP	POTENT'L FROST ACTION
NONE			1.0-3.0	APPARENT	JAN-APR	-		>60		-		C	HIGH

[a]The Crosby series consists of deep, somewhat poorly drained soils formed in loess and the underlying glacial till on moraines and till plains. Typically these soils have dark grayish brown silt loam surface layers 22.9 centimeters (9 inches) thick and mottled light brownish gray silt loam subsurface layers 5.1 centimeters (2 inches) thick. The subsoil is mottled yellowish brown clay loam in upper 48.3 centimeters (19 inches) and yellowish brown and grayish brown loam in lower 15.2 centimeters (6 inches). The underlying material is brown loam. Slopes range from 0 to 6 percent.

Table 2. Morphological[a] and estimated soil properties of Plainfield soils.

ESTIMATED SOIL PROPERTIES

DEPTH (IN.)	USDA TEXTURE	UNIFIED	AASHTO	FRACT >3 IN (PCT)	PERCENT OF MATERIAL LESS THAN 3" PASSING SIEVE NO. 4	10	40	200	LIQUID LIMIT	PLASTICITY INDEX
0-8	LS, LFS	SM	A-2, A-4	0	100	100	55-95	15-40	-	NP
0-8	S, FS	SP-SM, SM	A-3, A-2	0	100	100	50-95	5-35	-	NP
8-60	S	SP	A-3, A-1, A-2	0	75-100	75-100	45-70	1-4	-	NP

DEPTH (IN.)	CLAY (PCT <2MM)	MOIST BULK DENSITY (G/CM3)	PERMEABILITY (IN/HR)	AVAILABLE WATER CAPACITY (IN/IN)	SOIL REACTION (PH)	SALINITY (MMHOS/CM)	SHRINK-SWELL POTENTIAL	EROSION FACTORS K	T	WIND EROD. GROUP	ORGANIC MATTER (PCT)	CORROSIVITY STEEL	CONCRETE
0-8	5-15	1.35-1.65	2.0-6.0	0.10-0.12	4.5-7.3	-	LOW	.17	5	2	<1	LOW	HIGH
0-8	4-9	1.35-1.65	6.0-20	0.07-0.09	4.5-7.3	-	LOW	.17	5	1	<1		
8-60	1-4	1.50-1.65	6.0-20	0.05-0.07	4.5-6.0	-	LOW	.17					

FLOODING FREQUENCY	DURATION	MONTHS	HIGH WATER TABLE DEPTH (FT)	KIND	MONTHS	CEMENTED PAN DEPTH (IN)	HARDNESS	BEDROCK DEPTH (IN)	HARDNESS	SUBSIDENCE INIT. (IN)	TOTAL (IN)	HYD GRP	POTENT'L FROST ACTION
NONE			>6.0			-		>60		-		A	LOW

[a]The Plainfield consists of excessively drained soils formed in sandy drift on outwash plains, stream terraces and glaciated uplands. The surface layer is brown loamy sand 20.3 centimeters (8 inches) thick. The subsoil is dark yellowish-brown sand 30.5 centimeters (12 inches) thick. The substratum is light yellowish-brown, yellowish-brown and strong-brown sand.

Table 3. CBR values for Crosby soils.

County Sampled	Lab CBR Value
Bartholomew	4.4
Boone	3.6, 4.0
Hancock	3.1, 2.2
Henry	3.0, 3.9
Madison	4.7, 3.8
Marion	4.0, 3.5
Montgomery	3.0, 3.0
Tippecanoe	3.9, 3.2, 4.6
Wayne	3.3, 3.6

Table 4. Comparable soil groups.

AASHTO Group	Unified Group	Usual CBR Range	Most Probable CBR
A-1-a	GW,GP	20+	25
A-1-b	SW,SP,GM,SM	15-20	15
A-3	SP	8-12	10
A-2	GM,SM,GC,SC	8-12	10
A-4	ML	6-10	7-8
A-6	CL	4-7	5-6
A-7-5	MH	3-6	4
A-7-6	CH,CL	2-5	3

Table 5. Estimated California Bearing Ratio (CBR) alues for Indiana soils.

Soil Name	Estimated CBR	Soil Name	Estimated CBR	Soil Name	Estimated CBR
Ade	10	Crosier	4	Huntington	4
Adrian	0	Cuba	5	Huntsville	5
Alford	5				
Algiers	5	Dana	5	Iona	5
Alida	7	Darroch	6	Ipava	3
Allensville	6	Del Ray	3	Iva	4
Allison	5	Dickinson	8		
Alvin	8	Door	6	Jasper	6
Armiesburg	5	Dowagiac	6	Johnsburg(a)	4
Aubbeenaubbee	4	Dubois(a)	4	Jennings	5
Ava	4	Dunning	3	Jules	5
Avonburg(a)	5				
Ayr*	7-4	Eden(b)	3	Kalamazoo	6
Ayrshire	6	Edwards	0	Kerston	0
		Edenton(b)	3	Kings	2
Bartle(a)	5	Eel	6	Kokomo	3
Baxter	2	Elkinsville	5		
Beasley	3	Elliott	4	Landes	7
Bedford*(a)	5-2	Elston	8	Lawrence*(a)	5-2
Belmore	8	Evansville	3	Lenawee	3
Berks(b)	4			Lindside	4
Birds	5	Fabius	10	Linkville	4
Bloomfield	10	Fairmount(c)	3	Longlois	6
Blount	3	Fincastle	5	Lorenzo	10
Bonnie	4	Flanagan	3	Lowell	3
Bono	2	Foresman	6	Lucas	3
Boonesboro(b)	6	Fox	6	Lydick	6
Boyer	10	Frederick	2	Lyles	5
Brady	7	Fulton	2		
Brems	15			Mahalasville	3
Bronson	8	Genesee	6	Manlove	5
Brookston	3	Gessie	6	Markham	4
Burgin	3	Gilford	5	Markland	3
Burnside	8	Gilpin(b)	4	Martinsville	6
		Ginat(a)	4	Martisco	0
Camden	5	Glenhall	6	Massie	8
Carlisle	0	Granby	8	Matherton	6
Casco	10	Grayford	5	Maumee	8
Catlin	5	Guthrie*(a)	4-2	McGary	3
Celina	4			Medway	6
Chalmers	3	Hagerstown	2	Mellott	5
Chelsea	15	Hanna	7	Mermill*	4-2
Cincinnati(a)	5	Haskins*	5-3	Metamora	4
Clarence	3	Haubstadt(a)	4	Metea*	7-4
Clermont	4	Haymond	6	Miami	4
Colyer(c)	4	Hennepin	6	Milford	3
Conover	4	Henshaw	4	Millsdale(b)	3
Conrad	0	Hickory	4	Milton(c)	4
Corwin	4	High Gap(b)	4	Monitor	5
Cory	4	Hillsdale	7	Montgomery	2
Corydon(c)	2	Homer	6	Montmorenci	4
Coupee	8	Hoopeston	7	Morley	4
Crane	5	Hosmer(a)	5	Morocco	10
Crider*	5-2	Houghton	0	Muren	5
Crosby	3	Hoytville	2	Muskingum(b)	4

Soil Name	Estimated CBR
Mussey	10
Nappanee	3
Negley	7
Newark	4
Newton	8
Nicholson(a)	4
Nineveh	6
Nolin	5
Oakville	15
Ockley	6
Octagon	4
Odell	4
Ormas	10
Oshtemo	8
Otwell(a)	4
Owosso*	5-4
Palms	0
Parke	6
Parr	4
Pate	3
Patton	3
Pekin(a)	5
Peoga	4
Petrolia	3
Pewamo	3
Pike	5
Pinhook	5
Plainfield	15
Plano	5
Pope	7
Princeton	6
Proctor	5
Quinn	5
Ragsdale	3
Rahm	3
Randolph(b)	4
Rarden(b)	3
Raub	5
Rawson*	5-3
Reesville	4

Soil Name	Estimated CBR
Rensselaer	4
Riddles	4
Rimer*	5-3
Robinson	3
Rockcastle(b)	3
Rodman	15
Romney	3
Ross	6
Rossmoyne(a)	5
Runnymede	4
Rush	5
Russell	5
Ryker	5
Saranac	2
Saugatuck	8
Sciotoville(a)	6
Sebewa	4
Seward*	5-3
Shadeland*(b)	6-4
Shipshe	10
Shoals	6
Sidell	5
Sleeth	5
Sloan	5
Sparta	15
St. Clair	3
Starks	5
Steff	5
Stendal	5
Stonelick	7
Stoy(a)	5
Strole	2
Sunbury	3
Switzerland	3
Swygert	3
Sylvan	5
Taggart	5
Tama	5
Tawas	0
Tedrow	10
Tilsit(a)	4
Tippecanoe	6
Toledo	2

Soil Name	Estimated CBR
Toronto	5
Tracy	7
Trappist(b)	4
Treaty	3
Troxel	5
Tyner	15
Uniontown	4
Vigo	4
Vincennes	4
Volinia	6
Wallkill	0
Wakeland	6
Wanatah	5
Warners	0
Warsaw	6
Wasepi	10
Washtenaw	3
Watseka	10
Wauseon*	4-2
Wawasee	5
Wea	6
Weikert(c)	4
Weinbach(a)	5
Weiss	10
Wellston	4
Westland	4
Wheeling	6
Whitaker	6
Whitson	3
Wilbur	6
Willette	0
Wingate	5
Woodmere	4
Woolper	3
Wooten	10
Wynn(b)	5
Xenia	5
Zanesville(a)	4
Zipp	2

[a] Soils that have a fragipan or compact, impervious layer at a depth of 18 to 32 inches below the top of the soils.
[b] Soils that have bedrock at a depth of 20 to 40 inches below the surface of the soil.
[c] Soils that have bedrock at a depth of less than 20 inches below the surface of the soil.

Figure 1. A soil survey delineating Crosby (CrA) and Brookston (Br) soils.

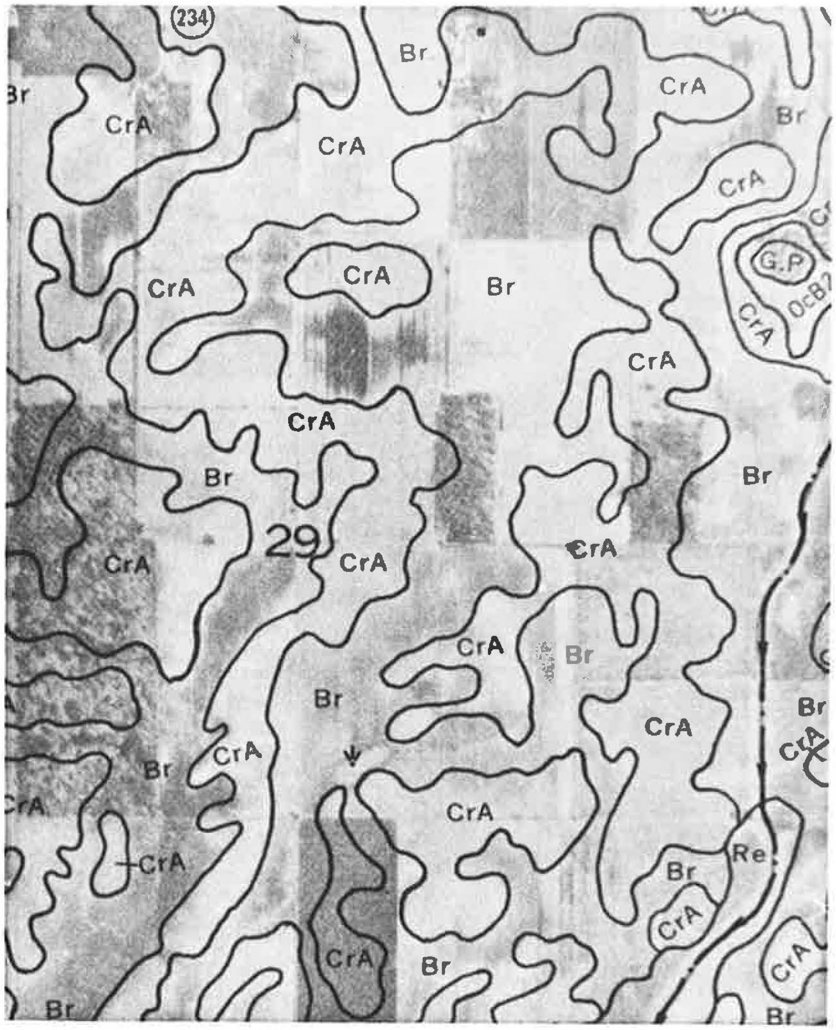
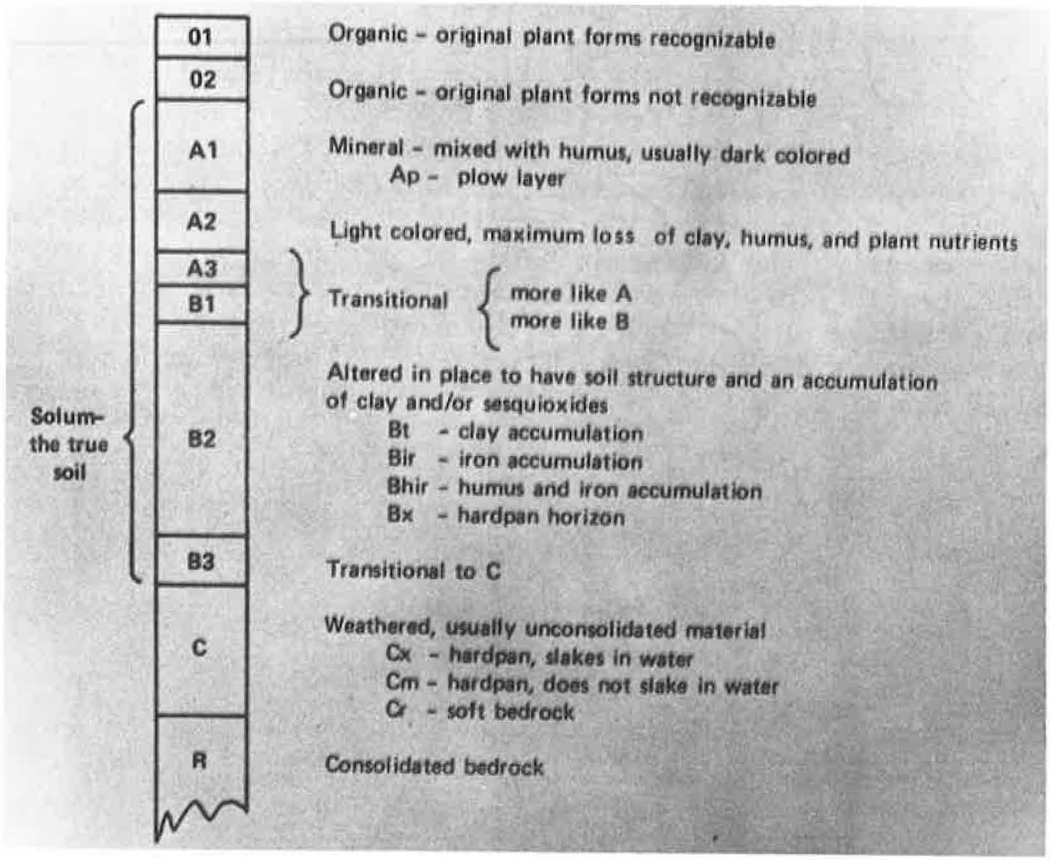


Figure 2. Major soil horizons. No one soil profile would contain all horizons listed, but most soils have some kinds of A, B, or C horizons.



DO EARTHWORK IN THE COLD WEATHER?

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Abstract

The latest (1973) survey of cold weather earthwork practices in the United States and Canada showed that the level of such activity was increasing. A number of operations may be carried out at unit costs equal to or less than those expected in the summer. Still other operations will have higher unit costs in the cold weather, but can be justified on the basis of benefits or savings to the road user and/or the agency or company doing the construction. Prominent among these savings are (1) the earlier availability of a new and safer facility; (2) the better utilization and retention of employees with year-around construction scheduling; and (3) the reduction in inflation which accompanies earlier project completion dates. This paper synthesizes the reported state-of-the-art, and makes recommendations with respect to the cold weather earthwork practices which seem to be promising for low volume roads. These recommendations are potentially useful to an agency that wishes to re-evaluate and possibly expand its cold weather activity.

The object of the research reported in this paper was to establish and summarize the overall state-of-the-art in cold weather earthwork, specifically for highways, airfields, and building construction in areas of seasonally frozen ground. A more complete presentation of the findings is contained in an unpublished report (3) for the U. S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL). Related research is contained in a published report (28) and a soon-to-be-published paper (29). While the topic of construction of low volume roads are given no particular attention in the literature search and questionnaires directed to practitioners, the potential applications are certainly present. The economic advantages afforded by an earlier completion of the project through cold weather work seem stronger today than ever before.

Literature Review

Temperature Effects on Soil Properties

Unfrozen Soil. The compaction characteristics of soils have been observed to be temperature dependent over the range of temperatures experienced in the field. The source of this behavior is believed to be the viscosity of the water, which determines the ease or difficulty with which soil particles or aggregation of particles can reorientate during the compactive process (22). Hence, a decrease in soil temperature results in a corresponding decrease in maximum dry density and an increase in the optimum moisture content for a given compactive effort. Highter et al (16), researching (in the laboratory) the temperature effects on the compaction and strength behavior of a clay, concluded that the detrimental effects produced by low temperatures might be overcome by modest increases in the compactive effort.

Frozen Soil. The engineering properties of frozen soils are very dependent on temperature. In general, some unfrozen liquid is present, representing the "adsorbed" water. In a frozen soil, the unfrozen water can exist in equilibrium with the ice over a wide temperature range below 0°C. Lovell (21) observed that substantial proportions of the total soil moisture of fine grained soils remained unfrozen at temperatures as cold as -25°C. In Figure 1, the percentage of unfrozen water versus temperature for various soil types is plotted. Observe that essentially all the moisture in the sand is frozen at 0°C (18).

The strength behavior of a frozen soil is dependent, in part, upon the amount of frozen water. With decreasing temperatures, an increasing proportion of the soil moisture becomes frozen and the ice "locks" the soil grains in position. Lovell (21) found that the compressive strengths of partially frozen saturated soils demonstrated a strong temperature dependency. One set of results, considering soils molded at approximate Standard AASHTO peaks, exhibited compressive strength ratios of 4.1 to 1 for silty clay and 2.8 to 1 for clay at temperatures of -18°C and -5°C. Laboratory data or well documented field evidence describing the compaction characteristics of frozen soils is meager. The compaction curves of Figure 2 are schematic for sandy soils at various freezing temperatures and ice contents.

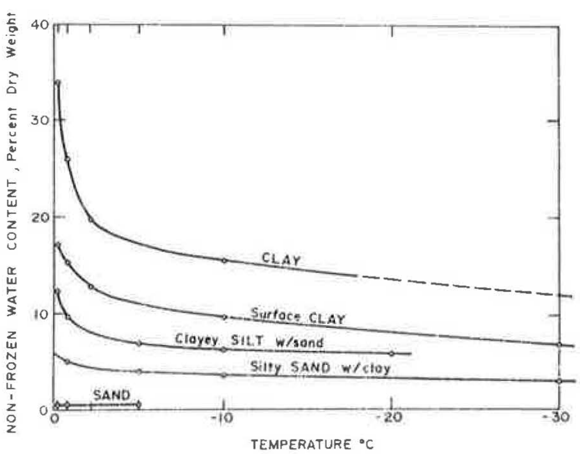

Figure 1. Non-Frozen Water Content of Soils vs. Temperature Below Freezing (From 18).

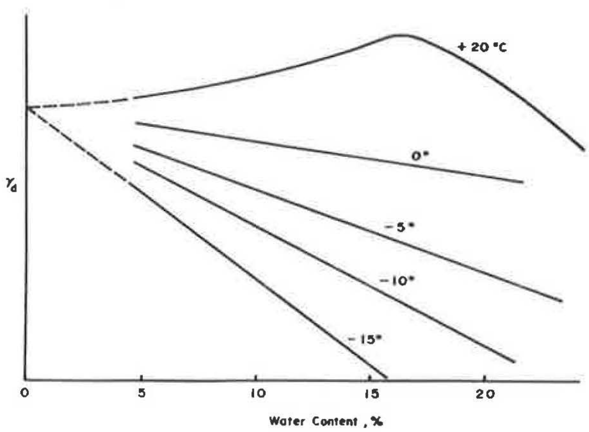

Figure 2. Compaction Studies with Frozen Soils (From 18).

Current Cold Weather Earthwork Practices

Scheduling. Two indispensable conditions for successful winter operations are careful preplanning and timely preparation (4). Cold weather earthwork cannot be accomplished economically if scheduling, planning, and preparation are haphazard.

With proper scheduling, operations fairly insensitive to cold weather, such as rock excavation, can proceed as during warmer periods of the year. The added equipment and supplies deemed necessary to carry out winter operations must be present on the job site in advance. A further example would be the advance preparation of an area to be excavated, prior to the ground freezing.

A vital asset in scheduling is knowing the weather forecast in advance. Up-to-date weather information, which statistically analyses weather trends for different areas in the United States, is available from 50 to 100 companies specializing in forecasting (30).

Siting. The primary considerations in site or route selection for cold weather operations are to avoid problem soils, and minimize haul distances. Wet silty soils, because of their frost susceptibility and ready release of moisture upon thawing, are especially troublesome and should be avoided (9). Heavier plastic clays with 3 to 5% moisture above optimum can be compacted if high impact tampers are utilized. Granular soils are the least sensitive to cold weather effects and give the most satisfactory results.

By minimizing the haul distance, the usual warm weather economics are effected. It is also possible to reduce special problems such as the freezing of borrow material to the truck body (20). Long haul routes also require greater maintenance. In the cold weather, much borrow material and equipment manhours may be required just to keep these routes trafficable (35).

In selecting sites for winter construction, consideration must be given to the project profile with relation to the natural grade. The normal practice is to limit winter work to deep cuts or deep fills. Due to the high cost of ripping frozen overburden and the difficulty encountered in cutting side ditches, shallow embankment profiles are better scheduled for summer operations. Transition sections, cut site to fill site, also incorporate the complication of construction operations within the depth of frost penetration.

Small Area Excavation. Small area excavation includes the procedures most commonly used to excavate for utility trenches, footings, culverts and the like. When working with non-frozen soil during cold weather, the usual procedures can be followed. However, it may be advantageous to provide some means to prevent frost penetration during the non-working periods. A continuous operation (24 hours) is a possibility under some circumstances.

Removal of soil frozen to a moderate depth will require extra effort, and often times challenges the ingenuity of the contractor. Each method has its merits and disadvantages, and the choice of procedure must be based on the particular job. Breaking through frozen ground with power shovels, dozers, front end loaders, rippers, air hammers, and wedges appears to be the popular procedure. In frozen ground 12 to 18 inches (0.3 to 0.5 m) thick, backacting shovels have proven to be the most successful. Bucket attachments which provide deeper penetration into the ground are available (19, 2). Excavation of frozen ground accelerates wear and tear of the equipment, requiring more frequent servicing.

Drilling and blasting to loosen frozen ground is another principal method. Several developments, e.g., delayed shooting, new and inexpensive explosives, and mechanized drilling, have considerably reduced costs and broadened the opportunities for frozen ground excavation (18).

Thawing (another option) is accomplished by many of the following procedures.

(1) Insulation with straw or wood scraps, possibly covered by black polyethelene sheeting to draw the heat (19).

(2) With steam and cover; by jets, steam points, or coils laid on the ground surface (1).

(3) Open fires of coke, tires, straw, wood; could possibly use reflectors or enclosures (10)

(4) Blanket of hot sand.

(5) Flame throwers (26).

(6) Ponding or spraying heated water on the open surface.
(7) Gas Burners (24, 33)

Method number (6) is disadvantageous in that ponding water is slow and will probably increase the moisture content of the soil being thawed.

The excavation and placement of rock embraces few new difficulties as a result of cold weather. The usual procedures employed during warmer periods are generally adequate for winter months. A difficulty may arise if the rock ledge is water bearing. Stockpiled shot rock containing free water may freeze into a solid mass. Frozen overburden will also impede progress in rock excavation (23).

Large Area Excavation. Highway cuts and borrow pits are examples of large area excavations. In cases where the soil is unfrozen and the temperatures do not fall consistently below the freezing mark, it may be sufficient to follow the usual excavating procedures. Modifications may be desirable when the temperatures are at or below freezing. The principal objective is to maintain the soil in an unfrozen state during excavating, hauling and placing operations.

Continuous operations, 24 hours per day, 7 days per week, can be utilized to this end, once a given cut or borrow area is opened. Restrict the excavation to as small an area as possible. This will reduce the surface exposed to freezing temperatures and lessen borrow waste. Loading the hauling equipment with power shovels and front end loaders has two distinct advantages. First, the exposed surface is limited. Secondly, working a high bank will tend to keep the temperature of the borrow material higher than if excavating from the surface down with scrapers.

During cold weather operations, the soil has a tendency to adfreeze to the steel surfaces of the loading and hauling equipment. Provisions must be made for periodic removal of the adhered soil to prevent equipment damage and loss of efficiency. The normal manner in which frozen soil is removed consists of chipping frozen chunks with air hammers, picks and shovels (20). Heating the boxes with the equipment exhaust is another possibility.

Excavation of frozen ground of moderate depth over a large areal extent is usually accomplished mechanically. Thawing is generally not practical. One of the most effective methods is by tractor mounted ripper (13, 14, 32). The forward tip of the ripper reaches beneath the frozen slab and tears out large chunks. Figure 3 depicts the method of successive passes, employed to rip greater depths of frozen ground.

Under some circumstances, the frozen ground may be too hard to penetrate with the ripper. In such a cases, breaking through the frozen crust with explosives may provide a starting point for an effective ripper operation. Several important tips for successful ripping operations are as follows (12):

(1) Hold ripping to a minimum; this operation does not move earth.
(2) Keep the cut small; prevent ripped areas from refreezing.
(3) Watch traction; side hill operations are difficult.

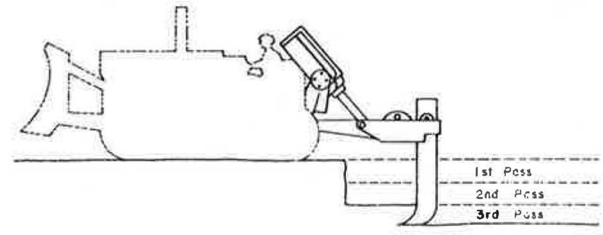

Figure 3. Loosening Ground with Dozer Mounted Ripper; Multiple Passes for Deep Penetration (From 18).

Large rock excavations are generally not affected by cold temperatures and work may proceed as during warmer periods. Peat bogs are unique situations, in that freezing temperatures tend to assist operations. Freezing to a moderate depth in such deposits increases the trafficability and the stability of the side slopes. Once the surface is trafficable, scrapers can easily remove the soil. Large surface areas are worked to expose the peat deposit to the freezing temperatures. Ripping may increase the depth of frost penetration.

Hauling. Hauling of non-frozen borrow to the job site is accomplished with the usual earth moving equipment. During periods of freezing temperatures or for long haul distances, it is advisable to cover the material in the truck with canvas to retain the heat. If feasible, heating the truck boxes or scraper bowls reduces the opportunity for the soil to adfreeze to the compartment.

Keeping the haul road, borrow pit, and spread area in good condition will permit an efficient, high speed operation. Operators must remain alert, particularly during periods of reduced visibility.

Spreading Fill Material. The fill spread will necessarily be kept to a minimum to reduce the surface area exposed to freezing temperatures. The ramp method of placing fill can be adopted to facilitate this end, Figure 4. This technique is

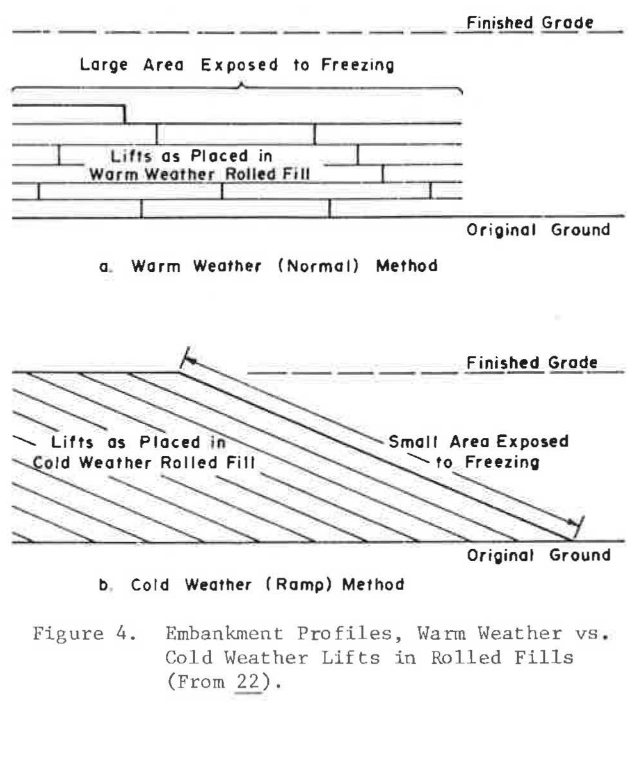

Figure 4. Embankment Profiles, Warm Weather vs. Cold Weather Lifts in Rolled Fills (From 22).

essentially the same as that used to cover and compact in solid-waste landfills. A smooth subgrade is essential for high speed operations and less equipment and operator fatigue. The work areas should be bladed smooth prior to shutting down each day. Lift thicknesses of 6 to 12 inches (0.2 to 0.3 m) for soils and 3 feet (1 m) for rock are generally acceptable. Utilizing the hauling equipment as compaction pieces, as much as practical, will enhance the efficiency of the operation. This practice has merit in either cold or warm weather.

Compaction. The degree of difficulty in obtaining the desired density is a function of the temperature, texture and moisture content of the soil. Dry sands and gravelly soils are relatively easy to compact, and fairly insensitive to freezing temperatures. Low evaporation rates and freezing temperatures make it impractical to lower the water content of wet cohesive soils through spreading and drying. It appears that the maximum water content for effective cold weather placement and compaction is approximately 2 to 3% above optimum. Heavy plastic clays with moisture contents above optimum can be satisfactorily compacted using high speed impact tampers. Silts are avoided, if possible, because of their high susceptibility to frost action (35). Wet cohesive soils have been used with alternating layers of granular soils. The granular soil layers provide horizontal drainage paths for escaping excess water during consolidation and spring thaw.

Use of Frozen Soil. Specifications are very restrictive with respect to the use of frozen soils. It is customarily written that no frozen soil be incorporated in the embankment, or that no foundation be placed on frozen soils. These rigorous specifications are warranted since ignoring them may lead to extensive damage from settlement or shear instability. There are, however, provisions for disposal much more satisfactory than simply wasting the frozen soil.

Some embankment designs permit the placement of frozen material outside the "design" section, i.e., beyond the shoulder break. However, even here the thawed material may produce an erosion problem. Frozen soil may also be used in stabilizing-berm structures. Stockpiling the frozen soil to thaw and reuse at a later date is another alternative. The thawed material can either be used as fill, or to dress up the borrow areas. Figure 5 illustrates the above mentioned dispositions (22).

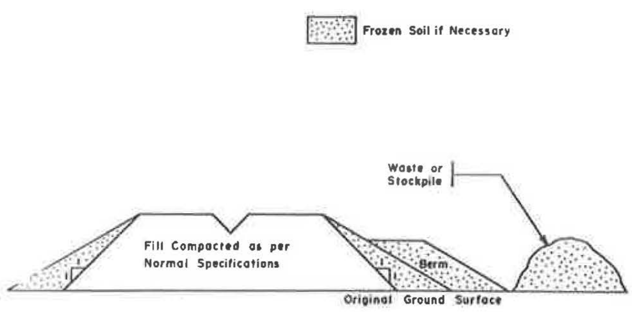

Figure 5. Locations Where Frozen Soil Can be Used without Detrimental Effects (From 22).

Field Engineering and Inspection. The surveying and engineering tasks performed in the field are generally adversely effected by cold weather conditions. The combination of windchill, extreme cold, and reduced visibility impair proficiency and operating effectiveness. One of the most common difficulties is trying to locate reference points in deep snow. Grade control is also harder to maintain because of greater equipment congestion on the confined work areas. More inspection is required during cold weather construction than during warmer periods, if a quality embankment is to be constructed. Eliminating snow, ice, and frozen soil from the fill material may require continuous inspection in the borrow area, depending on the attitude of the contractor and his equipment operators. Continuous operations, 24 hours a day, 7 days a week, will require more staffing to perform the engineering and inspection services.

Equipment. Special precautionary winterizing steps must be performed in order to maintain a fleet of operable construction equipment during the cold weather season (6, 17, 27, 34).

Table 1 illustrates the operational efficiency of construction equipment with respect to temperature (T), light (L) and precipitation (P) (22). Malfunctions resulting from various sources impose greater maintenance and servicing requirements. Hence, greater down time will require a larger fleet of equipment to perform the given task. Rental equipment is generally available to meet this requirement due to the reduction of work during the cold season. Ice, snow and frozen ground will make traction difficult, resulting in a slow speed operation. Equipment will need to be shut down periodically in order to remove adfreezing material from the truck and scraper compartments and crawler tracks. Several precautionary measures taken at shut down will facilitate "starting up" for the next day's operation (6, 12, 17, 27, 34).

Table 1. Relative efficiency of operation of construction machinery. From (22)

Weather Factor	Excavation Machinery	Hauling Machinery
Air Temperature (°F):	E_T %	E_T %
85	87	89
70	99	100
50	100	100
40	99	100
32	99	97
25	98	100
15	92	96
-5	78	88
-25	43	66
Light Condition:	E_L %	E_L %
Bright Sunshine	96	96
Indirect Sunshine	100	100
Dusk	88	96
Subartic Winter Twilight	65	82
Precipitation	E_P %	E_P %
Heavy Rain	81	85
Light Rain	97	98
Dry, Temperature 50°F	100	100
Light Snow	97	95
Heavy Snow	73	76

Personnel. Investigators (15) have found that cold weather affects men psychologically and physiologically. The net result is a loss in efficiency due to the climatic factors. The U. S. Army (8) has found that the elements which govern the comfort, and therefore the efficiency, of workmen in cold weather are air temperature, surface wind velocity, and relative humidity (22). Climatic factors can be related to human working efficiency as shown in Table 2. Any reasonable combination of the climatic variables shown in the three parts of Table 2 can be combined by multiplying the respective working efficiences. For example, at a temperature of 15°C at dusk with a light snow falling the working efficiency for manual laborers is (0.88)(0.92)(0.90) = 0.73 (22).

Table 2. Relative working efficiency of manual laborers. From (22)

Weather Factor	Working Efficiency (%)
Air Temperature (°F):	
85	72
70	95
50	100
40	98
32	97
25	95
15	88
-5	73
-25	33
Light Condition:	
Bright sunshine	97
Indirect sunshine	100
Dusk	92
Subartic winter twilight	56
Precipitation:	
Heavy rain	36
Light rain	89
Dry, temperature 50°F	100
Light Snow	90
Heavy Snow	41

Exposure to very cold temperatures may result in tissue or non-tissue damage. Havers and Morgan (15) present an excellent summary of the physiological reactions to cold weather. Improving the comfort of the individual will improve efficiency. Cabs and heaters will do much for the morale of the operator. If the unit cannot be equipped with a cab, plan as much work as possible so that the wind is at the operators back. Encourage workers to dress properly and to have additional clothing on the job site for unexpected cold or windy conditions. Several layers of clothing provides more warmth, allows more freedom, and permits faster adjustment to changing conditions than one heavy garment (15).

Economic Considerations. Much of the controversy concerning construction seasonality revolves around its economic feasibility. Proponents of cold weather construction regard "tradition" as the major obstacle to overcome; while opponents often emphasize the high cost of winterizing a site. Without cold weather activity, there is much seasonal unemployment and a host of labor problems, e.g., high hourly wages, labor shortages, difficulties in recruiting new workers, costly overtime payment, high unemployment insurance rates, restrictive labor practices, lack of a stable and properly trained work force, and an absence of employee loyalty to the contractor (31). These difficulties spread out, adversely affecting manufacturing wages, production schedules of building suppliers, and costs of building materials. The overall condition feeds inflation and places an economic burden on the nation (31).

Economic studies by various proponents of cold weather work have demonstrated that net economic benefits are expected, provided that planning is adequate (4, 5, 7, 9, 11, 23, 31, 36).

The contractor, although faced with the increased costs of winterization, is credited with the following obtainable benefits:

(1) Greater utilization of equipment and capital.
(2) Stable year round work force of experienced operators.
(3) Availability of rental equipment.
(4) Opportunity to meet completion deadlines; hence no penalty fees.
(5) Supplies readily available.
(6) Savings of cost of construction increases which occur from January to June (4).
(7) Savings in unemployment insurance rates.
(8) Savings derived from not staffing a skeleton crew during shutdown.
(9) Cost savings derived from easier construction under certain circumstances.

The cost of winterizing and performing cold weather earthwork will in part depend on the nature and conditions of the soil to be moved, the specifications covering the grading requirements, the experience of the contractor, and the permissable latitudes in the quality of the end product.

The public, the owner and the manufacturer also stand to benefit economically from cold weather construction. The public receives a new facility sooner, reducing inconvenience, travel time, traffic fatalities, injuries and property damage. Statistics show that Interstate highways facilities are safer than the older facilities they replaced.

"On completed sections of the Interstate system the fatality rate is 2.8 deaths per million vehicle miles, compared with a rate of 9.7 on older highways in the same traffic corridors" (25). Lovell and Osborne (22) attempted to express the increased safety in terms of economic savings.

An earlier completion date also means a reduction in the total elapsed time from financial commitment by the owner to use of the facility. With current high rates of interest on construction loans, there is considerable motivation to reduce construction time. The manufacturing of building products follows the same general seasonal curve as that of the construction industry. A savings in building material costs could be realized if seasonality were reduced. A leveling out of construction activity would enable the manufacturer to plan for nearly constant production each month of the year (36).

Replies to Questionnaires

Up-to-date information on techniques of cold weather earthwork and the state of the art in North American is lacking. One survey was completed in 1966 (37). The authors solicited replies to questionnaires in 1973 from state highway departments, district offices of the U. S. Army Corps of Engineers, and selected Canadian province highway departments. All questions emphasized earth embankment construction during conditions of subfreezing temperatures. The responses are fully presented only in an internal report (3). Since the tabulations are large and of unsuitable format for presentation here, only a selection of the 1973 responses from the state highway departments is included. Although the information is somewhat old, it is the most recent known to the authors.

The continuation of work or placement of certain embankment materials during adverse weather conditions is commonly determined by the "Engineer". In all but a few state specifications, placement of frozen soils in embankments or placement of fill on frozen foundations is prohibited. Practically every department which allows the placement of fill during subfreezing temperatures reported using granular soils or rockfill. In accordance with state specifications, frozen subgrades are either removed or left in place. When wasted, the frozen soil is generally deposited outside a designated slope beyond the highway shoulder. Frozen subgrades are left in place when they meet a depth-of-frost criterion.

Alaska does not place frozen material in embankment construction, but will permit a frozen subgrade to remain if it is granular, and if the specified density had been obtained prior to freezing. Had the lift not been compacted or if ice lenses have formed, placement of the next life is suspended until thawing of the frozen subgrade has occurred.

The heavy snowfalls in the mountainous areas of California suspend all work except some rock excavation. When absolutely necessary, California accomplishes highway embankment construction during cold weather by maintaining 24 hour shifts to keep the soil in an unfrozen condition. Kansas also uses the continuous construction approach on occasion.

Colorado normally does not permit the placement of soil during freezing weather, but suggested a "dry uncompacted lift" be placed prior to shutdown to act as an insulating blanket. This lift can then be reworked at the start of the following shift.

Combining depth-of-frost-penetration and soil-type criteria, Connecticut determines the acceptability of a frozen subgrade by visual inspection. Suspension of soil placement occurs as a consequence of a frost depth greater than 3 inches (0.1 m), provided the soil type is not predominantly silt or clay.

Peat treatment operations are scheduled in Indiana to take advantage of the beneficial freezing of these unsuitable soils, which facilitates handling and increases machinery efficiency over that experienced during the warmer months. Once the peat has been excavated, it is replaced by granular soils using techniques which will prevent the fill from freezing during placement.

An informal study in Maine showed that the compactive effort required to achieve the specified density at $15^{\circ}F$ was approximately six times as great as that effort employed at $40^{\circ}F$.

To reduce the surface area exposed to subfreezing temperatures, Michigan specifications require the contractor to construct the embankment by the ramp method of fill placement. Michigan also reports that a greater inspection effort is required to insure quality in the embankment construction. They also find winter construction gives rise to increased wearing of construction equipment, and noted that a characteristic of sand at or below $15^{\circ}F$ was an "apparent cohesion" or "gumminess".

New York (37) reported an investigation into the compaction characteristics of a relatively clean granular soil in the field and laboratory where "...as the temperature dropped below $30^{\circ}F$, the compactive effort necessary to achieve specified densities increased tremendously, and when a soil temperature of approximately $20^{\circ}F$ was reached, it was almost impossible to achieve specified densities, regardless of the compactive effort or the type of equipment". As a result of their experience, New York has limited its operations between 1 November and 1 April to approved rockfill only.

Summary and Conclusions

The practices of state highway departments, Canadian provinces and the U. S. Army Corps of Engineers in constructing earth embankments and proceeding with earthwork through the winter months was investigated via a questionnaire in 1973. The replies were tabulated and critically reviewed with respect to the amount of earthwork, procedures utilized, and respective attitudes regarding the feasibility of cold weather earthwork. They showed that cold weather earthwork in the U. S. is performed on only a small scale. Advances in technology are proceeding at a slow rate and increases in the volume of cold weather earthwork have been quite nominal. This is in spite of strong evidence that when such activity is carefully planned it can produce a net saving. The primary reason for such a situation seems to be "tradition". Hopefully those agencies constructing low volume roads will be less restrained by historic precedent, and can be motivated to experiment in order to achieve some of the potential economic advantages.

Acknowledgments

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USE THE GEOTECHNICAL DATA BANK!

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Abstract

Computerized systems of data storage, retrieval and analysis for information about the soils and rocks within a state are being used relatively frequently. These systems have particular and special potential uses in the design of low-volume roads, where the funds to generate original geotechnical data are very limited. This paper briefly describes the data likely to exist in such a system, how to access them, and what kinds of predictions can likely be made from them. Of special interest are: (1) frequency distribution analyses of particular soil characteristics to determine typical magnitudes and variabilities within a given area, and (2) correlation of simple and easy-to-measure soil characteristics with parameters that require complex and costly tests. Either approach may supply appropriate presumptive values for the structural roadway design, which can be verified by original subsurface investigation if the budget permits. Specific examples are drawn from the Indiana bank of geotechnical data.

The need for geological, pedological and geotechnical engineering information for use in site selection, planning, design, construction, and maintenance of transportation facilities and of most engineering structures is widely realized. Much of the information initially required by the engineer is used in preliminary construction planning, site selection and for guidance in further soil investigations. Unfortunately, most of these data are necessarily limited in quantity due to economic and time constraints.

The engineer is therefore faced with the problem of determining the location, sequence, thickness, and areal extent of each soil stratum, including a description and classification of the soils and their structure, by extrapolating the data from a few selected sites to an area many times greater than that which has been sampled. Even though large amounts of detailed soils data are available from previous work performed during planning and construction of nearby projects, these data are usually not readily accessible for use, or their existence is unknown.

The need therefore exists to make this information more accessible both for the engineer interested in detailed information of a site and the engineer interested in general soil characteristics over a large area. A computerized geotechnical data bank is the most efficient, expedient, and economical way to reduce the accumulated data to a form which can readily be made available to interested individuals, such as highway engineers, geotechnical engineers, contractors, and land use planners.

This paper briefly describes the development of a computerized geotechnical data bank, including the data likely to exist in such a system, and the kinds of predictions that can likely be made from them. Of special interest are: (1) frequency distribution analyses of particular soil characteristics to determine typical magnitudes and variabilities within a given area, and (2) correlation of simple and easy-to-measure soil characteristics with parameters that require complex and costly tests. Both uses of the data are particularly appropriate to low volume roads, where the geotechnical data generation for a particular job is quite limited.

Data Bank

Large amounts of geotechnical information for transportation projects are accumulated each year by highway departments throughout the United States and abroad. Geotechnical investigations are conducted to provide surface and subsurface information relative to soil, rock and water. This information is used in selecting the proper locations for the project and in making design decisions (3, 11). Subsequent use of this information *after* the design and construction of a project from which soil samples are taken and geotechnical data generated has been limited (4, 10).

Recognizing this, geotechnical data banks have been developed in a number of geographic locations, e.g., South Dakota (1), Kentucky (9), Indiana (2) and Sweden (5). The authors' experience is with the Indiana data bank, where data have been collected from private consulting firms, private soil testing firms, and from tests conducted by the Indiana State Highway Commission (ISHC).

What data are likely to be available from such sources? The minimum information stored for a particular boring is:

1. Location,
2. Gradational characteristics based on standard sieve sizes and hydrometer analysis,
3. Atterberg limits,
4. Visual textural classifications,
5. Color based on moist condition.

It is simple to write a computer program utilizing the above information to classify the samples by the American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification (USC) systems (2).

Other information to be stored, if available, includes:

1. Organic content (loss on ignition),
2. In situ moisture content,
3. In situ dry and wet densities,
4. Specific gravity,
5. Compaction test results,
6. California bearing ratio (CBR)
7. Unconfined compressive strength and failure strain,
8. Strength data from triaxial and direct shear tests,
9. Consolidation test results.

In addition to laboratory test data, field information to be stored should include:

1. Project identification,
   a. project number
   b. contract number
   c. road number
   d. data collection agency
2. Sample location,
   a. county
   b. district
   c. township
   d. range
   e. section
   f. line number
   g. station number
   h. offset and the left or right direction from the centerline
3. Sample identification,
   a. boring number
   b. laboratory number
   c. sampling procedure
4. Date the sample was taken from the hole,
5. Physiographic region,
6. Parent material from which the soil has been derived,
7. Ground surface elevation,
8. Depth from which the sample has been removed,
9. Depth to bedrock,
10. Depth to groundwater,
11. Standard penetration resistance (SPT),
12. Pedological soils information,
    a. soil association name
    b. soil series name
    c. horizon
    d. slope (topographic) class
    e. erosion class
    f. natural soil drainage class
    g. permeability
    h. flooding potential
    i. frost heave susceptibility
    j. shrink-swell potential
    k. pH

All of this information is transferred to a Data Input Form (DIF) such as is shown in Figure 1.

A User's Manual, explaining in detail the operation of the computerized data system, must be prepared (2). Included in the User's Manual are descriptions of the data items, the codification scheme to make the system compatible with computerized storage and retrieval, card formats, and card and column locations for each data item. Also needed is a listing of the programs used to add additional data to the data bank, to check data input errors, to use the computer programs for data management and manipulations, as well as example problems on the use of the data bank.

Benefits of the Data Bank

The benefits which can be obtained from the development of a computerized geotechnical data storage and retrieval system can be divided into two major categories: (a) the direct use of raw data; and (b) the use of statistical methods to reduce the data to a usable form via distribution characterizations, correlations, and predictions. Either approach may supply appropriate presumptive values for the structural design of low volume roadways, which can be verified by original subsurface investigation if the budget permits.

Direct Use of Raw Data

A computerized geotechnical data bank provides the capability of retrieving an extensive listing of available soil and rock information both quickly and economically. For example, the location of possible sources of granular and select borrow materials could be facilitated, along with route selection studies and right-of-way appraisals. Problem soil areas may be identified. In addition, the compilation of large scale engineering soil maps and profiles based on engineering characteristics is possible (1). This information would be of particular value in locating low volume roads.

Statistical Methods of Data Reduction

Statistical methods are used to study the variability of soil properties, to compare one soil type to another, and to group soil types with similar soil characteristics. Various correlations among selected soil properties can also be useful to the engineer when extensive laboratory testing is not possible (8, 12).

The first step in assessing the variability and typical magnitudes of selected soil characteristics is to develop frequency distribution characterizations of selected soil properties. In an attempt to explain the variation in the data, the soil samples may be grouped according to physiographic regions and parent material area. Figures 2 through 4 graphically illustrate the range, 95% confidence interval, and the mean of selected soil parameters, based upon such groupings for Indiana. These values will help the engineer to obtain an idea of the expected values of the soil parameters.

Prediction models of parameters that are difficult to measure and therefore require complex and costly tests are potentially of value if correlations can be made with simple and easy-to-measure soil characteristics. This was attempted for the Indiana data with dependent variables of:

(1) coefficient of consolidation (C_c) and compression ratio (C_r, which equals $C_c/1+e_o$, where e_o is the initial void ratio), (2) unconfined compressive strength (q_u), (3) standard Proctor maximum dry ($\gamma_{d_{max}}$) and wet ($\gamma_{m_{max}}$) densities and optimum moisture content (w_{opt}), and (4) soaked California bearing ratios (CBR) at 100 (CBRS01) and 95 (CBRS02) percent of standard Proctor maximum dry densities.

The independent variables included: (1) initial void ratio (e_o), natural moisture content (w_n), natural dry density (γ_d), liquid limit (w_L), plastic limit (w_p), plasticity index (I_p), percent clay, overburden pressure (p_o), and preconsolidation pressure (p_c) for the consolidation test data; (2) w_L, w_p, w_n, γ_d, and liquidity index (L_I) for the unconfined compressive strength data; and (3) w_L, w_p, I_p, and shrinkage limit (w_s) for the compaction and CBR test data.

If a particular dependent variable resisted state-wide modelling, or if data were contained in significant quantities to justify modelling on smaller units, that is, physiographic regions, parent material areas, and in some cases on soil types, the data were grouped accordingly to determine if the prediction models could be significantly improved.

Regression analysis was used to establish the prediction models for each dependent variable. The method of least squares was used to find "good" estimates of the regression parameters and to isolate the effects of the independent variables on the chosen dependent variables. The REGRESSION routine of the Statistical Package for the Social Sciences (SPSS) developed by Nie et al (7) was used. The regression results give adjusted coefficients of multiple determination (R_a^2). Usually, only the coefficient of multiple determination, denoted by R^2, is used as a measure of the proportionate reduction of the total variation in the dependent variable associated with the use of a set of independent variables. Since R^2 can be made large by increasing the independent variables in the model, R_a^2 was used as a criterion for selecting a good model. R_a^2 recognizes the number of independent variables in the model and may actually become smaller when another independent variable is introduced into the model (6). R_a^2 takes on values between 0 and 1. The larger R_a^2, the better the fitted equation explains the variation in the data.

The regression results summarized in Tables 1 through 3 are examples of analyses of stored data, with R_a^2 values greater than 0.65.

Summary

The geotechnical data bank has substantial potential for aiding in the design and construction of low volume roads. Frequency distribution analysis permits prediction of the range of values which may be expected for a given soil parameter, although these vary with the particular physical property and the population from which the soil has been sampled. The grouping of soils by physiographic regions and the origin of their parent materials shows that the predicability of some soil properties can be thereby improved.

Where budgets strictly limit the amount of geotechnical sampling and testing, prediction equations can be generated. We have shown how this was done for compressibility, strength, and compaction parameters. In some cases valid regression equations could be produced for an entire state, but generally the relations were stronger when the population of samples was from a smaller geologic or pedologic unit.

As new data become available for incorporation into the data bank, they should be used for validating the existing prediction models. The reliability of the equations can subsequently be improved. Soils information which was essentially "lost" after a project was completed, can now be utilized for future highway projects and improvements. The data bank should be maintained for all potential users, particularly the designers of low volume roads. Initial access to the bank for the latter group would undoubtedly be on a manual basis, with possible computerization later.

Acknowledgments

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Table 1 Summary of Regression Equations for Prediction of Compression Index (C_c) and Compression Ratio (C_r).

Unit	Dependent Variable	R_a^2	Regression Equation	Number of Samples, N
All Samples	C_c	0.856	$C_c = 0.5684\ (e_o + 0.0033\ w_L - 0.0082\ w_p + 0.0329\ p_c - 0.4322)$	96
		0.800	$C_c = 0.5363\ (e_o - 0.4110)$	
		0.792	$C_c = 0.0002\ (w_n^2 - 106.2727)$	
		0.783	$C_c = 0.0129\ (w_n + 0.1015\ w_L - 16.1875)$	
	C_r	0.691	$C_r = 0.2037\ (e_o - 0.2465)$	
Wabash Lowland	C_c	0.838	$C_c = 0.5673\ (e_o - 0.4422)$	29
	log C_c	0.831	$\log C_c = 2.7904\ (e_o - 0.3346\ e_o^2 - 0.8449)$	
	C_r	0.750	$C_r = 0.221\ (e_o - 0.3074)$	
		0.748	$C_r = 0.0065\ (w_n - 11.6361)$	
		0.735	$C_r = 0.0034\,((e_o)\ (w_n) + 8.3647)$	
Crawford Upland	C_c	0.859	$C_c = 0.0101\ ((e_o)\ (w_L) - 0.5765\ w_L + 12.665)$	28
		0.833	$C_c = 0.0114\ (w_n + 0.2491\ w_L - 18.8134)$	
		0.788	$C_c = 0.4941\ (e_o - 0.3507)$	
		0.777	$C_c = 0.0133\ (w_n - 12.1886)$	
	C_r	0.740	$C_r = 0.0001\ (w_n^2 + 455.8889)$	
		0.736	$C_r = 0.0033\ ((e_o)\ (w_n) + 12.5168)$	
		0.721	$C_r = 0.1164\ (e_o^2 + 0.3594)$	
Outwash and Alluvial Deposits	C_c	0.894	$C_c = 0.6076\ (e_o + 0.003\ w_L - 0.0095\ w_p + 0.0430\ p_c - 0.4186)$	63
		0.842	$C_c = 0.5621\ (e_o - 0.4215)$	
		0.822	$C_c = 0.0153\ (w_n + 0.1022\ w_L - 0.3104\ w_p - 11.6123)$	
	log C_c	0.772	$\log C_c = 2.1389\ (e_o - 0.2967\ e_o^2 - 0.9374)$	

Table 2 Summary of Regression Equations for Prediction of Unconfined Compressive Strength (q_u).
(q_u in kPa; γ_d in kg/m^3)

Unit	Dependent Variable	R_a^2	Regression Equation	Number of Samples, N
Calumet Lacustrine Plain	q_u	0.756	$q_u = 0.00268\ (\gamma_d^2 - 37.333)$	40
	$\log q_u$	0.750	$\log q_u = 0.0257\ (\gamma_d^2 - 116.265)$	
Lacustrine Deposits	$\log q_u$	0.699	$\log q_u = -\ 0.0257\ (\gamma_d^2 + 116.150)$	48

Table 3 Summary of Regression Equations for Prediction of Standard Proctor Maximum Dry ($\gamma_{d_{max}}$) and Wet ($\gamma_{m_{max}}$) Densities and Optimum Moisture Content (w_{opt}).
(γ's in kg/m^3)

Unit	Dependent Variable	R_a^2	Regression Equation	Number of Samples, N
All Samples	w_{opt}	0.894	$w_{opt} = -\ 7.958\ (\gamma_{d_{max}} - 9.005)$	138
Valparaiso Morainal Area	$\log \gamma_{d_{max}}$	0.816	$\log \gamma_{d_{max}} = -\ 3.683\ (1/w_L + 0.127 \log w_L - 0.454)$	26
		0.785	$\log \gamma_{d_{max}} = -\ 0.224\ (\log w_L - 5.269)$	
	$\gamma_{m_{max}}$	0.790	$\gamma_{m_{max}} = -7.118\ (\log w_L + 9.962\ (1/w_L) - 2.976)$	
	$\log \gamma_{m_{max}}$	0.694	$\log \gamma_{m_{max}} = -\ 0.135\ (\log w_L - 8.294)$	
	w_{opt}	0.972	$w_{opt} = 11.649\ (\gamma_{m_{max}} - 1.298\ \gamma_{d_{max}} + 3.203)$	
		0.870	$w_{opt} = 6.769\ (\gamma_{d_{max}} - 9.355)$	
		0.810	$w_{opt} = 23.0357 + 0.002\ (w_L)\ (w_p) - 285.939\ (1/w_L)$	
Residuum of Limestone Bedrock	$\gamma_{d_{max}}$	0.772	$\gamma_{d_{max}} = -\ 7.0843\ (\log w_L + 14.095\ (1/w_L) - 2.906)$	22
	$\log w_{opt}$	0.781	$\log w_{opt} = 0.0042\ (w_L + 259.0381)$	

Figure 1. Data input form (DIF).

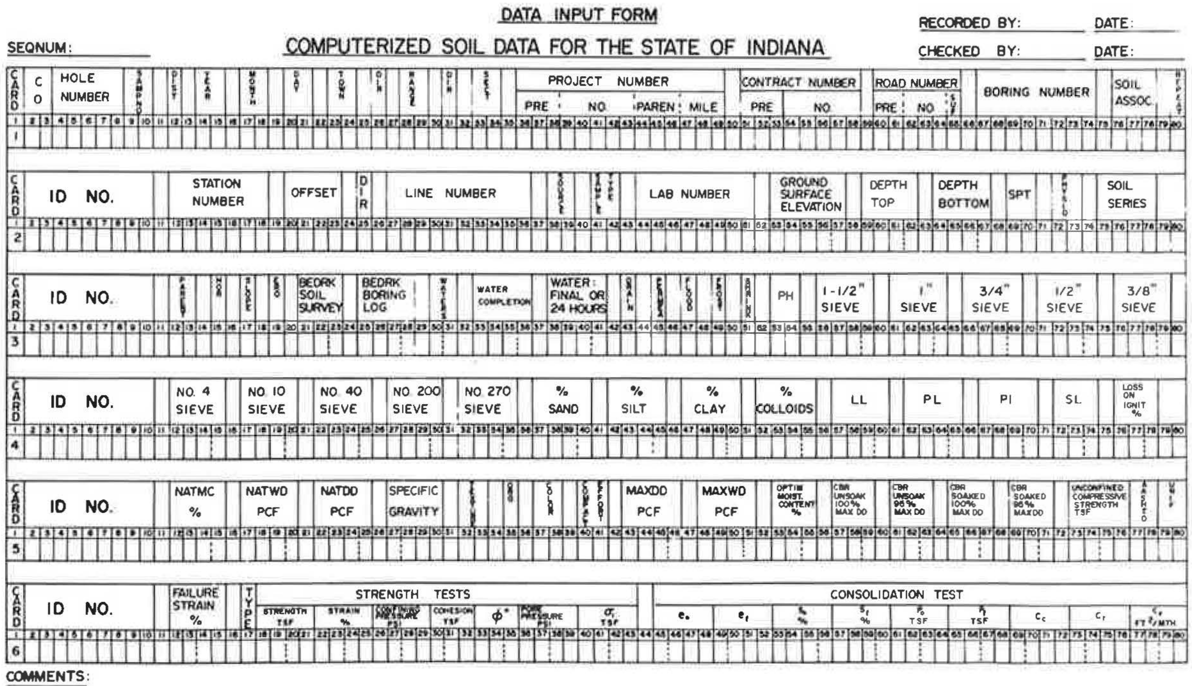


Figure 2. Distributional characterization of natural moisture content.

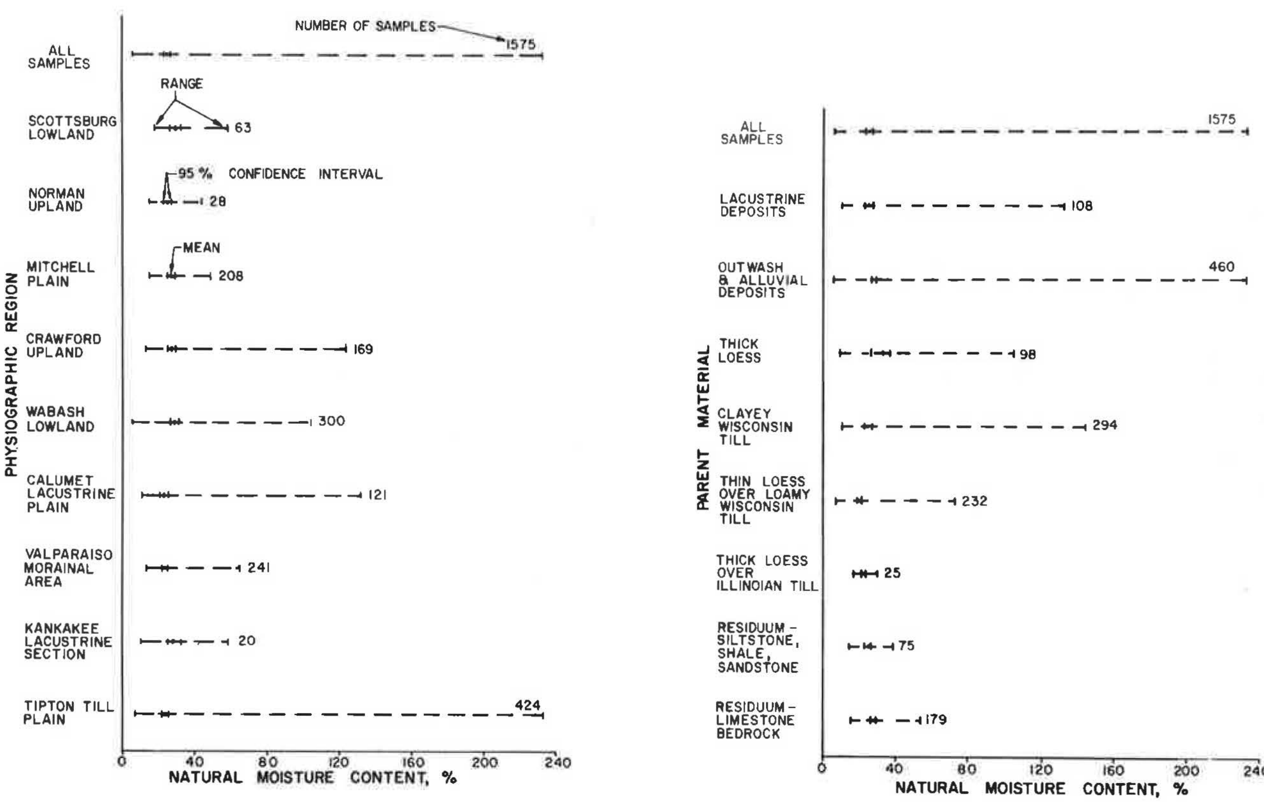


Figure 3. Distributional characterization of plasticity index.

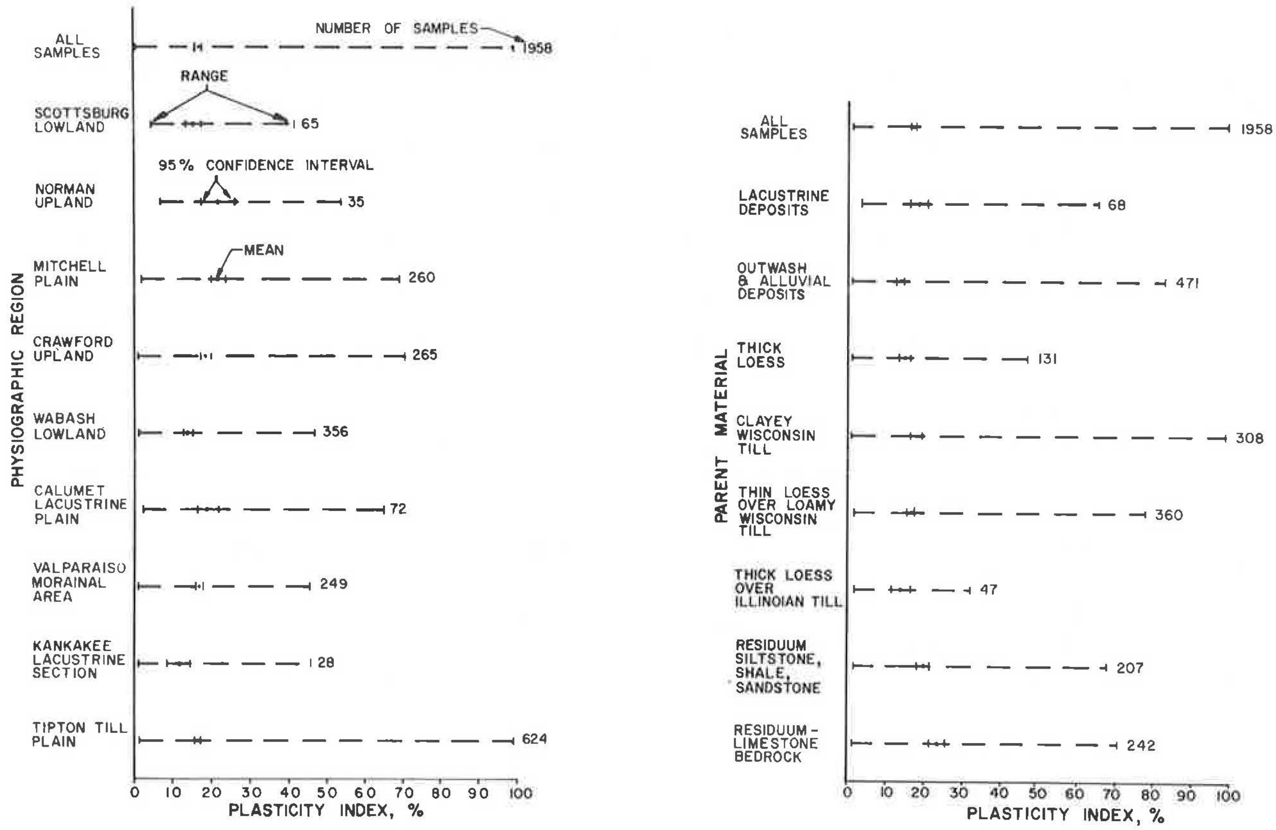
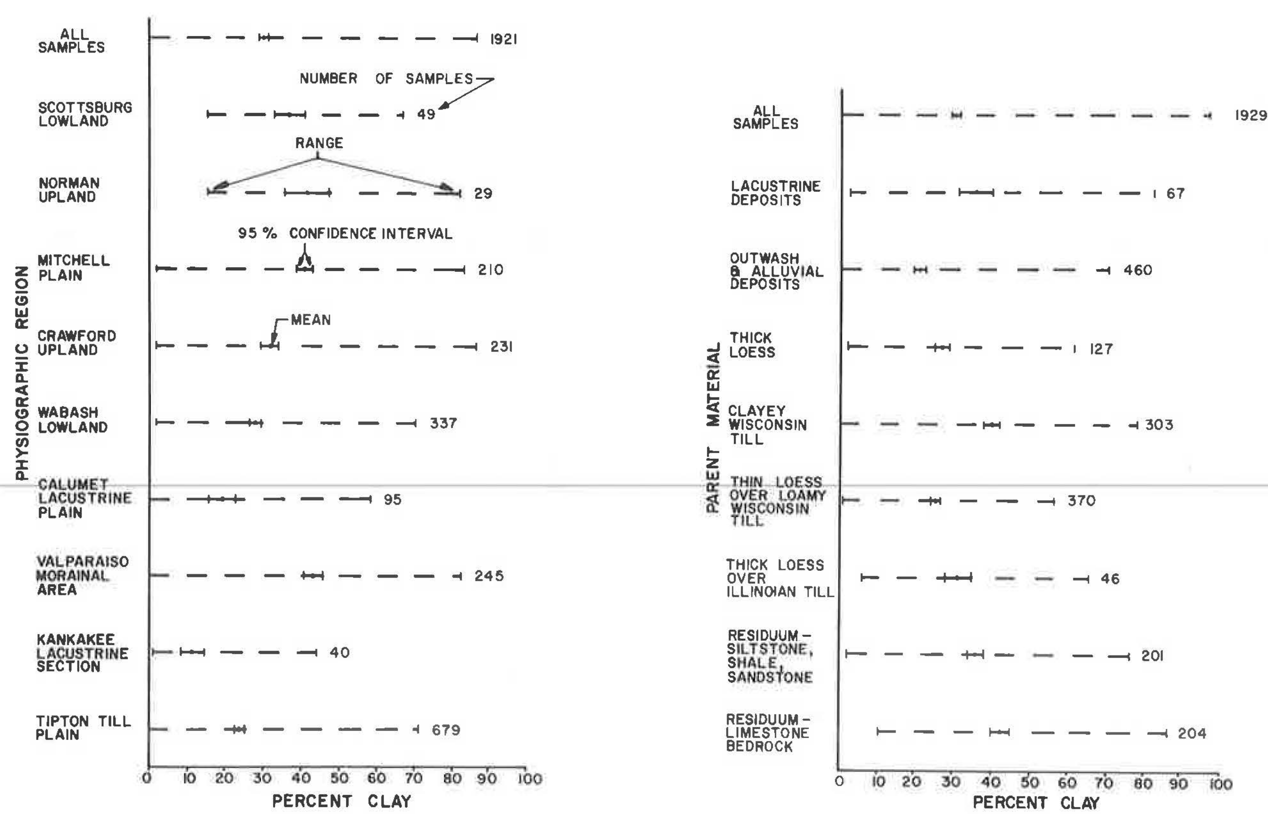


Figure 4. Distributional characterization of percent clay.



UTILIZATION OF SULPHUR-TREATED BAMBOO FOR LOW-VOLUME ROAD CONSTRUCTION

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This paper presents the current research findings on the utilization of sulphur treated bamboo for road construction. Basic engineering properties of bamboo culm (stem) including modulus of elasticity, compressive and tensile strength, as well as bamboo-water interaction are summarized and discussed. Two major applications, namely: bamboo rod used as a substitute for steel bar in structural concrete and bamboo reinforced earth are presented. Performance and durability of bamboo structures for engineering uses are examined. Finally, a field application procedure for bamboo used as reinforced earth for protecting low-volume roads against landslide is proposed.

For road construction, large quantities of materials are generally required. Since economic consideration is one of the major decision factors, it is necessary to examine alternate or low-cost materials for future road construction. In some regions the conventional construction materials, such as steel, not only are costs high but also hard to get. In such regions, the utilization of indigenous and replenishable material is ideal for road material, and it is believed that bamboo is one of such an ideal material (6).

Bamboo is classified as Bambusoideae or as Bambusaceae. It can be found in many parts of the world and has been used as a low-cost construction material for many years. However, bamboo has a major weakness due to high water absorption potential which leads to swelling-skrinking and decay and perhaps these factors are the main reason why bamboo is not widely used in today's modern construction. Recently, a low-cost simple treatment technique using a sulphur-sand method to overcome the above mentioned problems has been developed (7,10). The purpose of this paper is to present the additional research findings on bamboo rod used as substitute for steel bar in structural concrete and to reinforce earth with a bamboo mat and/or bamboo-lime column for new or existing highway embankments. Basic engineering properties of bamboo and durability of bamboo structures together with the field application for low-volume road construction are also presented.

Engineering Properties of Bamboo Culm

In using bamboo for engineering application, it is necessary to examine the basic characteristics of bamboo culm (stem). Bamboo consists of two distinctive layers of fibers. The inner portion is soft and spongy-like with a white color containing 15% to 30% of the fibers. The outer portion is stiffer with a light green or yellowish color and contains 40% to 60% of the fibers. As indicated by Fang and Fay (9), the fiber content and its behavior are mainly related to the bamboo age, type of bamboo and its sampling location within the culm. Figure 1 shows the variations of fiber contents versus sampling location (11,17,22). It indicates that the bottom portion has more fiber content than the top portion. The aged bamboo (3 to 6 years old) has more fiber than young green bamboo. The general strength characteristics of bamboo fiber and culm include tensile strength and compressive strength, modulus of elasticity, modulus of rupture, specific gravity, and others as summarized in Table 1. Based on previous and present research findings, it can be concluded that the aged bamboo gives higher strength of all kinds than green (young) bamboo. The bottom portion of the bamboo has higher strength, higher specific gravity than the top portion of the bamboo culm. To consider bamboo for comparison with other materials, Table 2 lists the strength-weight ratio of various materials among which the strength-ratio of bamboo shows an excellent condition. Since bamboo is a fiber material, the longitudinal and transverse directions are different. To examine this point, a Pennsylvania bamboo has been used for study. Two strain-rosettes were installed on the surface of bamboo culm, one on the knob (joint) and the other one between the knob. The typical stress-strain curves of the bamboo culm are shown in Figure 2. Both with and without knobs, the longitudinal directions have higher strain.

Using bamboo rod for reinforcement of concrete, the bamboo-water interaction should be examined. Bamboo will swell or shrink during a wet-dry cycle, and consequently, causes a loosening at the bamboo matrix interface and cracking. Fore reinforced concrete, the bamboo absorbs the moisture from fresh concrete and swelling occurs. If the swelling pressure of the fibers is large enough, it pushes the wet concrete aside. At the end of the curing period when the concrete becomes hard, the bamboo having lost the water shrinks, leaving voids between the bamboo rod and the concrete. The mechanisms of bamboo-concrete interaction during the curing process are discussed in references (8) and (9). Figure 3 shows the water absorption potential of various bamboo culms, and how the sulphur treatment techniques can reduce the absorption potentials.

Substitute for Steel Bar in Structural Concrete

Using bamboo for reinforced concrete was started early in 1914 by H. K. Chu at Massachusetts Institute of Technology (2). Since then, similar studies have been made in China, Germany, France, Japan, the Philippines, India, Egypt, Colombia, and more recently at the Waterways Experiment Station (3). Based on their general conclusions, bamboo has three major weaknesses of low modulus, low bond stress and high water absorption potential which leads to swelling, shrinking and decay. Therefore, using bamboo for a substitute steel bar in structural concrete or other engineering applications creates certain problems. As mentioned previously, a low-cost simple technique using the sulphur-sand treatment of bamboo rod will give better results (10). The procedure can be briefly described as follows: all bamboo specimens were sand blasted and thin wire wrapped around the bamboo culm. Then the specimens were impregnated with molten sulphur at 280° to 300°F for approximately one hour and then air dried. Before the sulphur-impregnated bamboo is completely dry, a coating of sand is applied. The main reasons for treating bamboo with molten sulphur are: to increase the confined pressure and to reduce the cracks during loading conditions, to waterproof the bamboo to minimize its swelling-skrinking potential as shown in Figure 3, and to aid the uniform coating of sand adhere to the bamboo surface making a rough surface in order to increase the bond strength between the bamboo and concrete. A series of experiments using 6"x6"x30" (15.2cm x 15.2cm x 76.2cm) and 6"x6"x72" (15.2cm x 15.2cm x 182.9cm) bamboo-concrete beams for examining the load-deflection characteristics of the beam were conducted. Various water-cement ratio, size of bamboo culm, and position of bamboo culm in the concrete beam were studied. The curing periods were all reached in 28-days. A typical load-deflection curve is shown in Figure 4 and the failure mode of the bamboo-concrete beam is shown in Figure 5. In all cases, the concrete portion reached failure first (see points A,B,C, and D in Figure 4) and the bamboo fiber in the concrete held together showing the ductility characteristics of concrete structures.

It is understandable, that for bamboo reinforcing concrete there is much less strength than in steel reinforced concrete in compression, bending, and rupture, however, bamboo shows high strength-weight ratio and a unique behavior of ductility which is very useful in earthquake regions or in developing countries (15,19) especially for low-volume road construction.

Reinforced Earth with Bamboo

The purpose of reinforced earth is mainly to increase the bearing capacity of weak subgrade soils, or to increase the resistance for retaining structures. The original concept developed by Vidal (21) was used with thin metal strips placed horizontally in layers held together by internal friction between the reinforcing strips and the material. This technique has been used frequently in various highway projects around the nation for protecting dams, embankments, etc. against landslide potential. Recently, the concept has been extended to the use of other materials to form tensile reinforcement elements of various configurations and sizes, such as: plastic membranes, fabrics, timber or corduroy mat, bamboo mat or strips and paper grid cells (4,5,11, 13,16).

As previously mentioned, road construction requires large quantities of materials. Since the economic factor plays an important role for road construction, using inexpensive bamboo mat or strips is one of the most attractive materials for reinforced earth construction.

Seismic responses of bamboo-reinforced earth also show good results in comparison with non-bamboo reinforced embankments, earth dams, and adobe walls (8,11,14). Figure 6 shows the proposed field installation layout for bamboo used for reinforced earth. For the existing earth embankment or dams, the vertical-type bamboo-pile is recommended, as shown in Figure 6(a). The length of bamboo culm should be larger than the depth of the theoretical failure plane. This theoretical failure plane can be determined by conventional slope stability analysis which is available in any standard soil mechanics and foundation engineering textbook. Since bamboo culm cannot take large impact loading, it is suggested to first excavate a boring hole and then install the bamboo culm. The boring hole should be slightly larger than the diameter of the bamboo culm to be used. The voids between the bamboo and soil wall in the boring hole are then filled with bentonite or quicklime as shown in Figure 6(b). The small diameter type of bamboo (varying from 1cm to 5cm) with lengths varying from 3m to 5m is preferred. The bamboo culm should be placed with the top part first as shown in Figure 6(c). The reason for this is to utilize the bamboo's natural branches in order to increase the skin friction of the bamboo culm. For long-term use, the bamboo culm should be treated with molten sulphur as described in a previous section. Using quicklime as a filler between bamboo culm and soil wall in the boring hole has shown many advantages due to the natural characteristics of water absorption and volume expansion of quicklime (1,12). It also indicates that bamboo-lime composite pile not only increases the bearing capacity, but also has high earthquake resistance. This type of material is useful for reinforced earth in earthquake regions where the conventional construction materials are hard to get. It is also useful in low-volume roads, such as forest roads and many roads in developing countries (6,15,19).

For reinforcing new embankments or dams, the horizontal type of bamboo mat or strip is suitable, as shown in Figure 6(d), 6(e), and 6(f). Standard AASHTO compaction criteria should be applied on each layer, in order to minimize the voids between bamboo mat and soil layers. The bamboo strip or mat also should be sulphur treated. A suggested bamboo mat is shown in Figure 7.

Durability of Bamboo Structures

The durability of bamboo is one of the major objects for our concern. A review of the literature on bamboo durability indicates that there is very little factual information available. However, a survey was made on the performance of existing bamboo structures throughout the world indicating that bamboo structures could stand for a long period of time. Figure 8 shows one of the many case studies carried out by the Geotechnical Engineering Division at Lehigh University. It shows a non-treated bamboo culm used in reinforced concrete wall built in 1940 and found in the vicinity of Taichung Harbor, Taiwan in 1976. Based on observation, the bamboo culm inside the concrete block still is in excellent condition despite the ocean environment and long period of exposure. Laboratory study carried out comparisons between sulphur treated and non-treated bamboo culm used in concrete beams (6,7). Of course, more research should be done on the long term performance study of bamboo structures.

Summary and Conclusions

1. Bamboo is a fast growing replenishable biological material and can be found in many places around the world. It can be grown in temperatures between 7°C below zero to 50°C provided it has water.

2. All bamboo used for engineering purposes should be seasoned (3 to 6 years old).

3. Sulphur treatment of bamboo is low cost and easy to apply. This procedure can reduce the water absorption potential significantly during the concrete curing process. It also will increase the modulus of elasticity and bond stress.

4. Using bamboo mat or strip for reinforced earth is a low cost ideal construction material, especially useful in low-volume road construction, forest roads, and some developing countries where the conventional construction material is hard to obtain.

5. Using bamboo-lime composite pile for reinforcement of existing earth embankment or dams shows many advantages in earthquake regions because the bamboo culm will take lateral seismic loading effectively.

6. Bamboo-reinforced structural concrete also shows good ductility characteristics and useful in earthquake regions.

7. The strength-weight ratio of bamboo shows excellent potentials in comparison with any other construction material currently used.

8. Utilization of modern geotechnical engineering techniques, such as field moisture control, compaction methods or combination with other soil stabilization, such as bentonite or lime improves the stability of the bamboo reinforced earth systems.

9. Further study is needed in long-term performance of bamboo structures, improvement in the durability of bamboo with other additives, and engineering identification, classification and standard test procedures.

Acknowledgments

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Table 1. Summary of strength characteristics of bamboo culm (stem).

		Range	References
1	Tensile Strength		
	U.S.A.	485 to 1,760 kg/cm^2	(3) (9)
	Japan	390 to 3,789 "	(25)
	Philippines	Ave. 1,674 (with knob) Ave. 2,371 (no knob)	(3)
	India	Ave. 1,547 (inner layer) Ave. 3,199 (outer layer)	(18)
	Egypt	Ave. 1,920 (with knob) Ave. 2,600 (no knob)	(24)
	China (Taiwan) Ma-bamboo Makino-bamboo	 2,546 to 2,751 3,061 to 3,452	(17) (22)
2	Compressive Strength, kg/cm^2		
	U.S.A. and others	460 to 610	(3) (20)
	Taiwan	404 to 929	(6) (17) (22)
3	Modulus of Elasticity, kg/cm^2		
	U.S.A. and others Taiwan	54,300 - 149,100 45,313 - 176,988	(3) (6)
4	Modulus of Rupture, kg/cm^2		
	Taiwan	649 - 2,180	(17) (22)
5	Specific Gravity	0.30 to 0.80	(3) (22) (17) (25)
6	Poisson's Ratio	0.25 to 0.409	(3)
7	Coefficient of Thermal Expansion		
	Across Fiber ^{o}F Parallel to Fiber ^{o}F	Ave. 23.70 x 10^{-6} Ave. 1.42 x 10^{-6}	(3)
8	Creep - Stress-strain-time relationships see Ref. (11)		

Table 2. Average strength-weight ratio of various construction materials

Material Types	Compressive Strength q, kg/cm^2	Specific Gravity G	Strength/Weigth Ratio q/G	Strength/Weight Index with respect to Steel
Bamboo	742.0	0.72	1030.06	1.26
Steel,hot rolled	6,400.0	7.80	820.51	1.00
Zelkova wood	558.5	0.700	797.86	0.97
Cast Iron	5,637.0	7.20	782.92	0.95
Quartz	2,000.0	2.65	754.72	0.92
Wrought Iron	4,227.0	7.70	548.96	0.67
Dolomite rock	1,500.0	2.80	535.71	0.65
Bronze	3,946.0	8.17	482.99	0.59
Copper	3,170.0	8.92	355.38	0.43
Sandstone	750.0	2.30	326.09	0.40

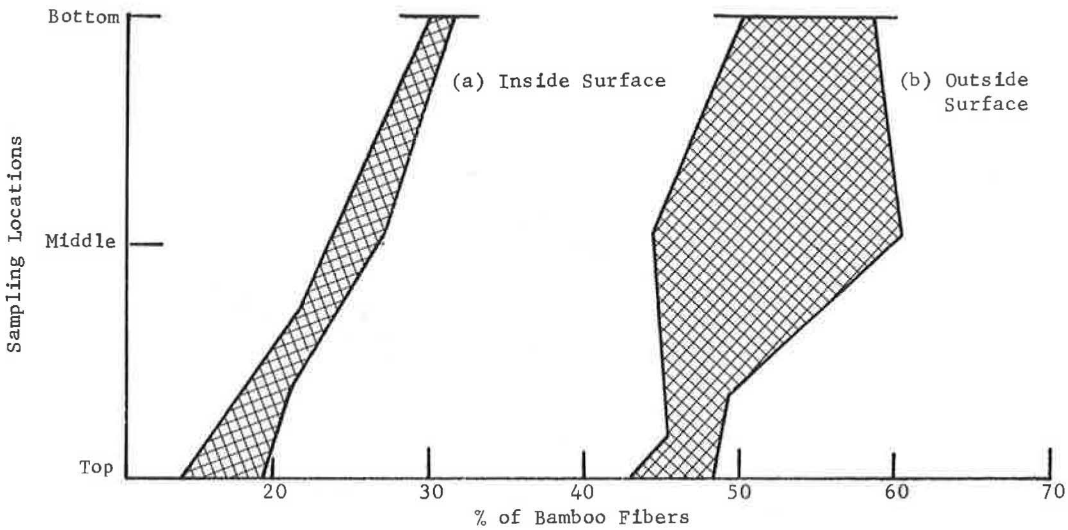

Figure 1 Variations of Bamboo Fiber Content at Various Parts of Bamboo Culm

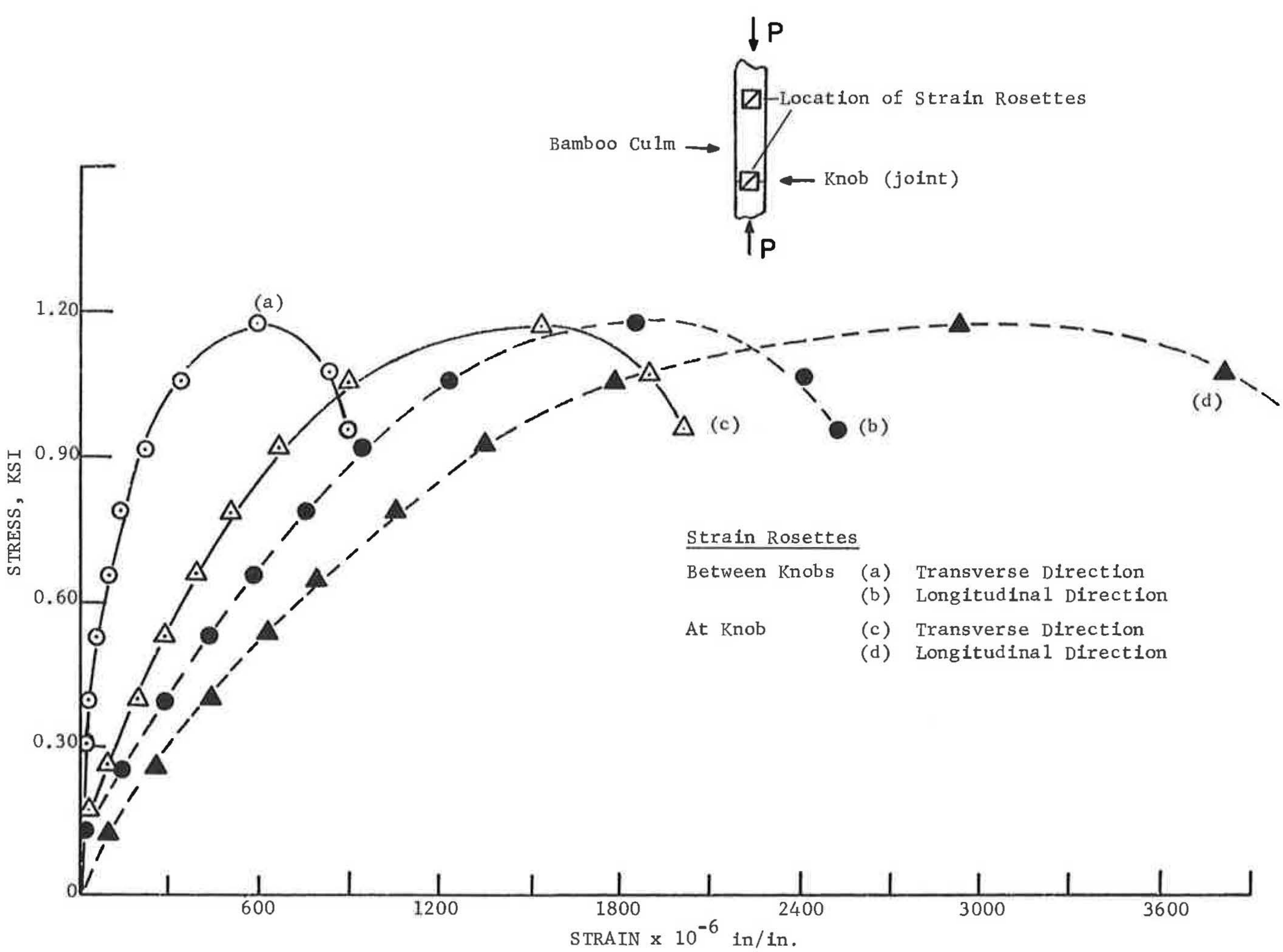

Figure 2 Stress-Strain Curves of a Pennsylvania Bamboo Culm

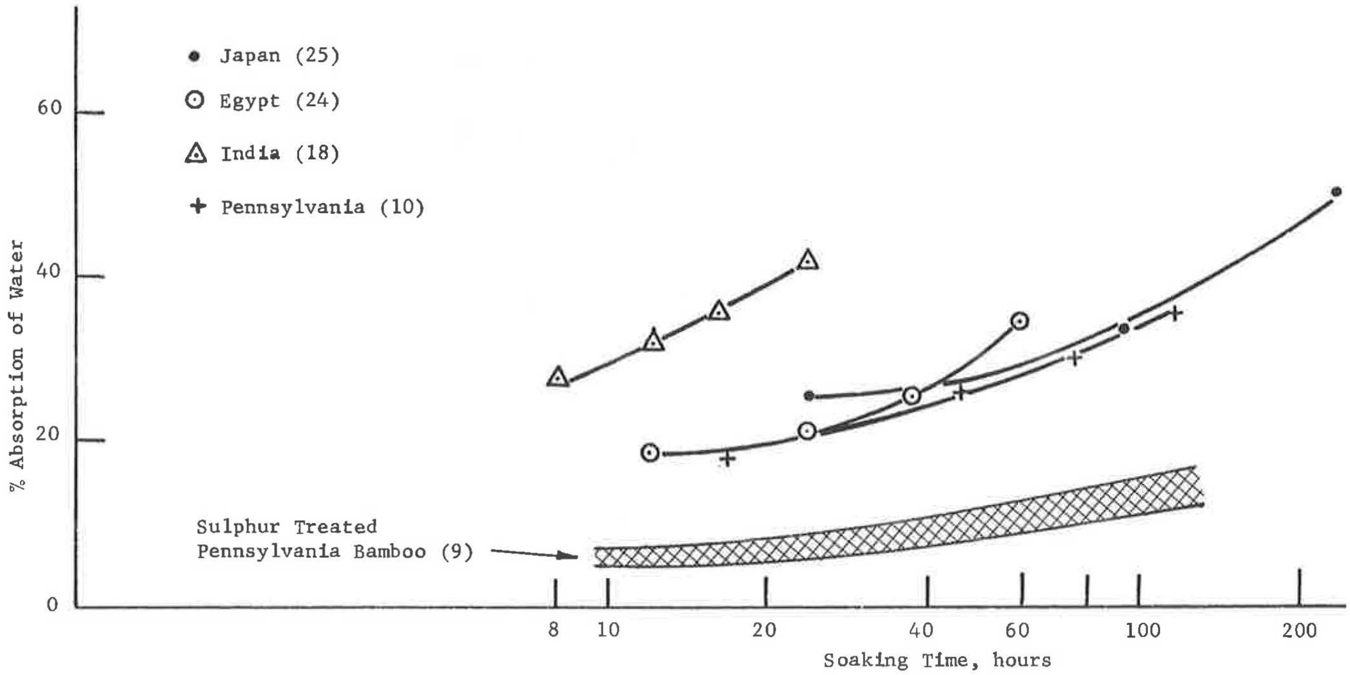

Figure 3 Absorption Characteristics of Various Bamboo Culms

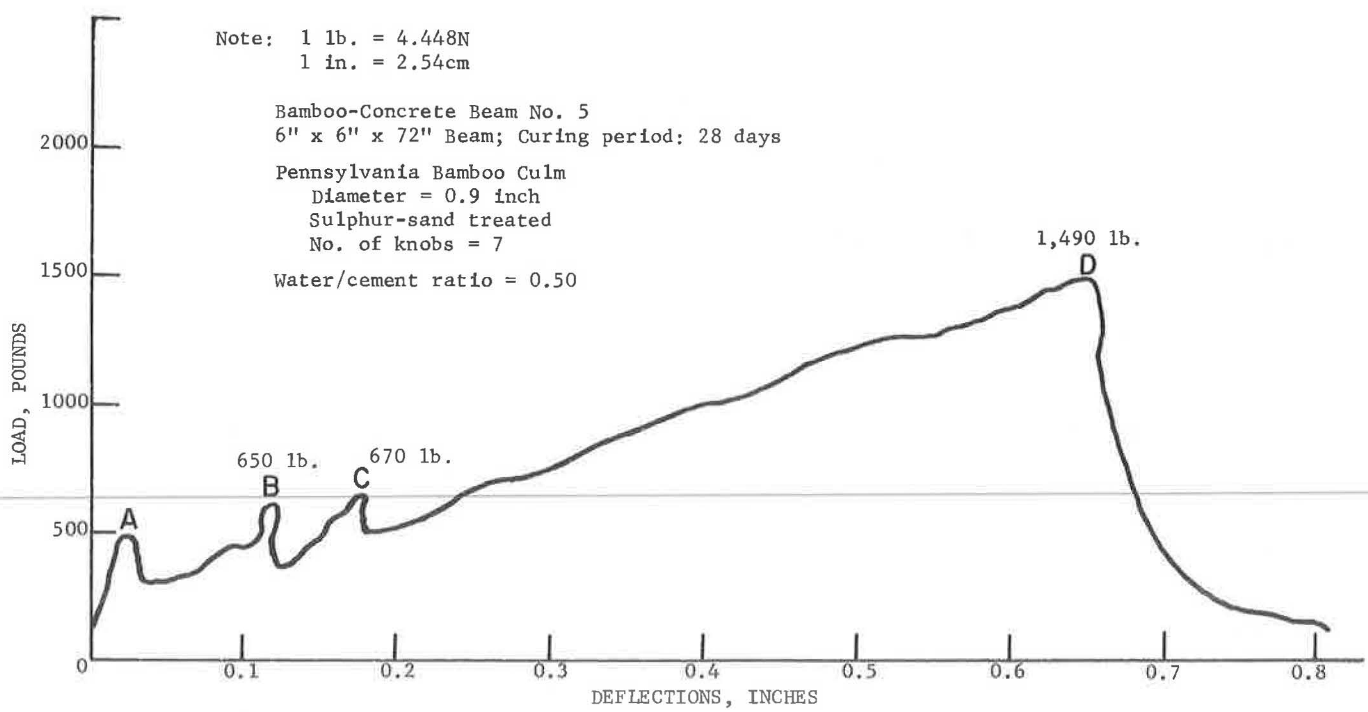

Figure 4 Load-Deflection Curve of Bamboo Reinforced Concrete Beam

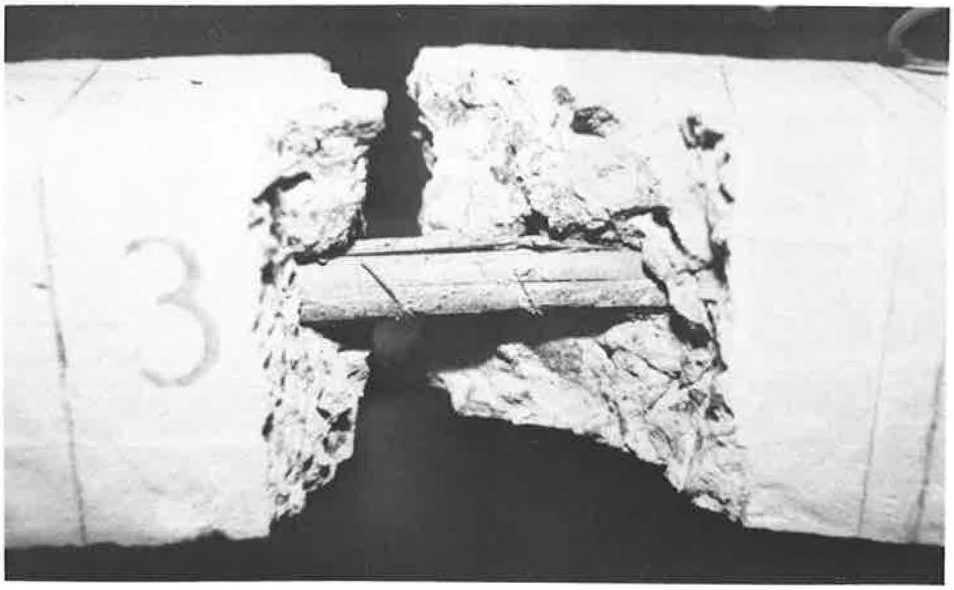

Figure 5 Failure Mode of Bamboo Reinforced Concrete Beam

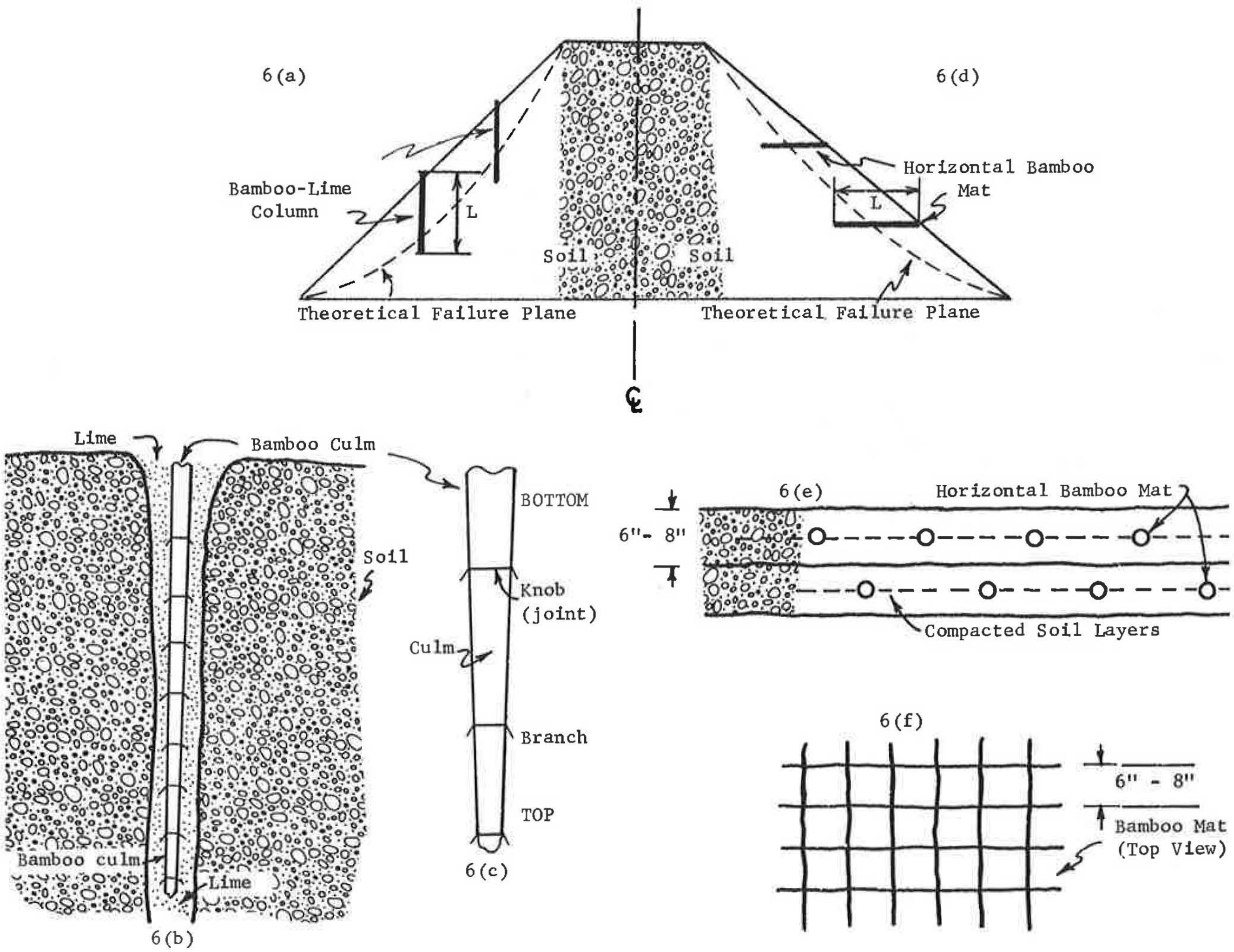

Figure 6 Proposed Field Installation Procedures for Bamboo Used as Reinforced Earth

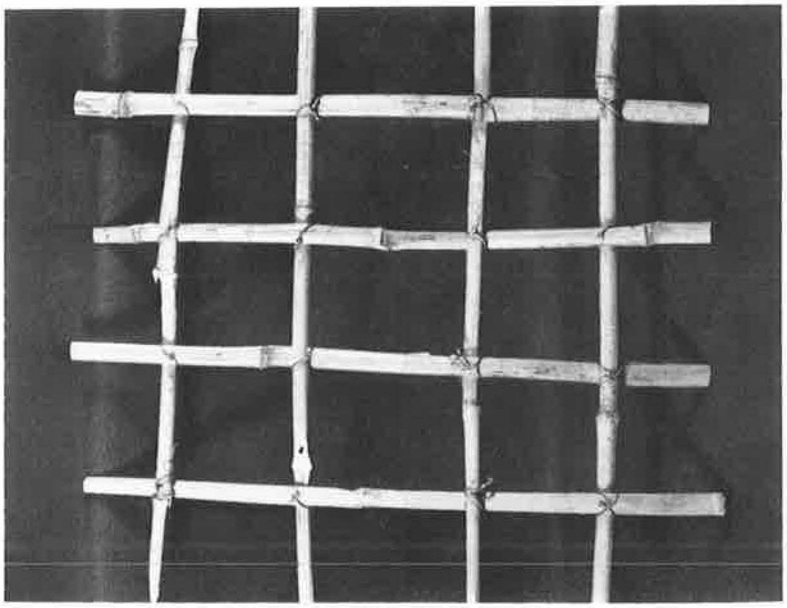

Figure 7 Bamboo Mat Used for Reinforced Earth.

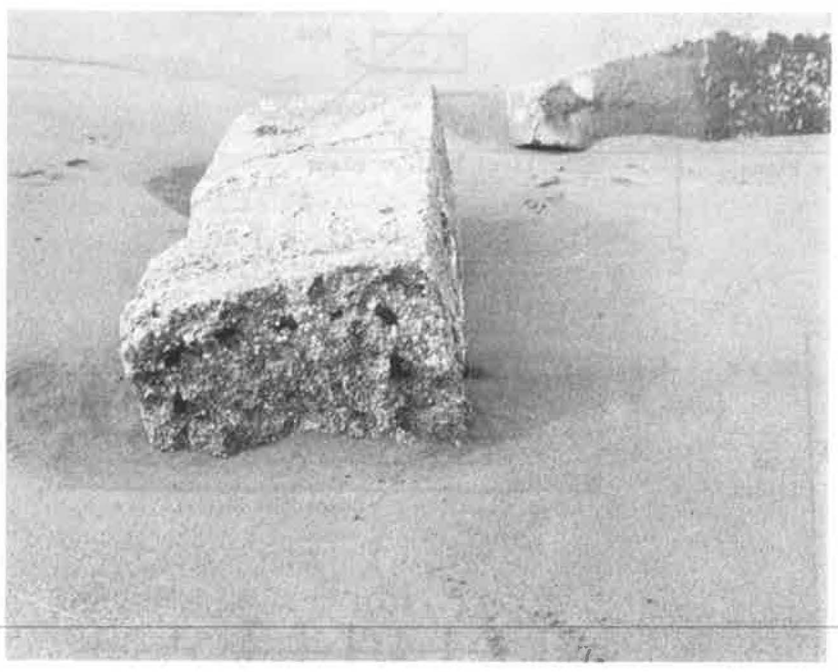

Figure 8 Bamboo Reinforced Concrete Wall Built in 1940 Found in Vicinity of Taichung Harbor, Taiwan, in 1976. Shows Bamboo Culm Inside the Concrete Wall Still in Excellent Condition. (Courtesy of H. C. Chiu, Taichung Harbor Construction Bureau)

THE OPTIMUM USE OF NATURAL MATERIALS FOR LIGHTLY TRAFFICKED ROADS IN DEVELOPING COUNTRIES

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The planning, construction and maintenance of lightly trafficked roads are discussed, with particular reference to the optimal use of local materials and resources. Attention is drawn to the importance of terrain evaluation techniques for improved road location and construction material surveys. The importance of an intimate knowledge and appreciation of local conditions and terrain for optimum serviceability is stressed. Foreign aid often covers only the planning and construction phases of development and loses interest during the maintenance after completion. This aspect is of particular importance to the financing institutions to expand the export of expertise and equipment for maintenance. The importance of, and examples illustrating procedures to promote labour intensiveness in highway construction in developing countries is mentioned. Examples of materials standards for gravel surfaced lightly trafficked roads, and the experience on which they are based, as well as the novel applications of certain natural resources are discussed. Geometric standards and drainage applications that have been found to be particularly practical are highlighted. Approximate cost estimates are included for the various grades of roads discussed.

In most countries in the world, and especially in Africa, trade routes have been established over the centuries. These routes form the basic backbone to the infrastructure as such, and should be considered as a fixed system.

Industrial and agricultural development, as well as the creation of new towns and villages, and the change from a subsistence-economy to a market economy requires encouragement through the provision of an infrastructure, consisting mainly of a tertiary road network, to germinate economic advance.

The obtaining of untied capital funds for developing countries poses problems, and consequently it is essential that the available capital resources be used in the most appropriate socio-economic manner. Road development funds which are provided in the form of "package deals" with constricting conditions are not in the best interest of developing countries; they do not lead to self development of the inhabitants of the countries concerned since they are normally conditional upon the use of expertise and equipment from the highly developed donor country.

The optimum use of local labour forces, and of local materials and construction techniques is of prime importance in the development of the infrastructural system. It is also essential that adequately trained personnel and suitable resources be developed during the course of construction so that continued maintenance of the facilities can be effected. All too often the mistake is made of providing over sophisticated facilities which quickly deteriorate due to inadequate and ill-advised maintenance. Since the roads provided as the tertiary network for developing countries generally develop into major routes, and also since they normally generate heavy axle loadings (trucks and buses) rather than the lighter loading associated with the extensive use of cars, it is essential that these roads be designed so as to easily permit later upgrading, with minimum disruption to traffic both geometrically and structurally, into arterial facilities.

For this reason stabilisation of the pavement layers using in-situ, or nearby occurring natural gravels is from a cost benefit viewpoint normally the best approach, since these layers will then form an ideal subbase for the heavier pavements which will follow with time.

Since the geological history and road building conditions in many of the world's developing countries in South America and Africa is very similar to that of South Africa, and also because South Africa has during the last 30 to 40 years experienced the socio-economic conditions now pertaining to these countries, it is considered that South African experience in respect of the development of a system of low cost roads will be of considerable benefit.

Planning

Terrain Evaluation for Location and Design

In order to ensure that from both geotechnical and materials usage aspects the proposed road is located in the optimum economic position, it is essential that terrain evaluation studies be carried out. The terrain over which the proposed road is to be constructed is generally variable, but the variables are the results of recognisable geological processes, where the interaction between geology, climate and time produces a myriad of landscapes, all of which are interpretable in terms of materials properties.

The basic objective of terrain evaluation is to communicate the relevant materials terrain data to the design engineer and consequently only those attributes relevant to the purpose in hand are evaluated and presented in a form intelligible to the engineer.

The actual requirements vary according to the project, but are covered broadly by the following:

1. location of construction materials,
2. assessment of centre line in-situ materials,
3. assessment of relative quantities and distribution of hard and soft materials,
4. definition of drainage and slope stability problems,
5. definition of areas of sub-surface moisture and erosion problems,
6. definition of areas of expansive or collapsible soils,
7. definition of climatic and weather conditions.

As many of these criteria can only be qualitatively described at the early design stage during which the evaluation of the terrain occurs, it is essential that the design engineer possesses a deep sympathy and understanding of the native geology and of the influence of local geological processes upon the terrain.

Terrain evaluation procedures generally lean heavily on air photo interpretation and it has been found in South Africa that the amount of information obtained from interpretation, and the degree of reliance that can be placed on the information so produced, is proportional to the degree of the interpreter's experience beyond his basic training, and also to the extent of confirmatory field work undertaken.

In addition the degree of communication between the interpreter and the design engineer has an effect upon the success of the work. Because of terrain evaluation's essentially practical origin in South Africa, it has been found that from the design engineer's viewpoint only limited detail regarding the terrain attributes is necessary in the early stages of a project, and that as the project progresses, different, and more detailed information becomes necessary. A continual channel of communication between the engineer and the interpreter is thus essential.

Complete integrity is required on the part of the interpreter and it is his duty to inform the engineer of possible weaknesses in the information, which may lie either in the lack of sufficient evidence on which to base his supposition, in his own inability to adequately interpret the photography, or in the lack of clarity as to what the engineer requires.

A mutual understanding of the requirements and limitations of the method is essential.

Road geotechnical engineering is an art - which depends for a large measure for its success upon the exercising of sound judgement; and sound judgement comes from long and tried experience, based on acute observation.

Of nothing is this more true than in the field of geotechnical engineering and it is in this aspect of the project where the study of, and decisions based on an appreciation of ground conditions can have major economic consequences that there should be a combined effort with mutually accepted responsibilities between the terrain evaluator and the highway engineer.

Optimum Procedures for Prospecting Road Building Materials

About seventy percent of the cost of road construction is associated with the use of materials. This is especially true for low cost roads where it becomes economically essential that all available sources are located and taken into account for design purposes. It is also important that the search for construction materials be undertaken on a logical basis and not as is often the case, in a haphazard random fashion.

The use of air-photo interpretation has long since been recognised as an aid in the search for construction materials. In addition the establishment of data banks for the storage of data pertaining to road materials usage for particular region has been encouraged, e.g.,

"It is desirable to establish a permanent exchange of 'know-how' and information between developed and developing countries with a view to the adaptation and utilization of project techniques by the latter countries, particularly as regards terrain evaluation, use of computers and automatic drawing of road projects as well as the establishment of road data banks and the gathering of geological data" (1).

During the last 8 years a system for the storage and retrieval of information pertaining to the occurrence of road building materials as well as its uses, including problems and construction hazards, has been operated in South Africa as an aid to materials prospecting. The procedure is based on the extrapolation of data from previously prospected areas to new areas with similar physiography.

The more important soil-forming factors which are considered responsible over a period of time for the technical properties of any material or soil class are the natural condition i.e. parent material, climate and relief. If any of these conditions change the materials types and problems change as well; on the other hand, one may expect the same technical properties in materials which are found in different localities but have the same parameters. Such a group of related materials has been termed a material analogue and is defined as "a number of material classes with the same parent rock, climate and relief". (2)

Consequently the standard references indicated below have been introduced for the coding of material data and the description for storage and retrieval when required, of material analogues:

1. Parent rock, described in terms of lithostratigraphy as applicable, and according to the legend of the 1:1000 000 geological map.
2. Weathering, based on the 30 year mean annual rainfall.

3. The relief of the area.

The general principle described above is not sufficient however to provide a field team with directives in their search for materials and a further reference is required indicating where materials with specific engineering properties might be located within an area of the same material analogue. It has been found that the most acceptable and practical reference is an indication of the relevant land surface forms. The relevant land form has therefore been chosen as a sub-unit for materials data banking. It is described with respect to surface shape and is defined as a term which describes the surface form of the locality of road construction materials reasonably uniform properties or soil classes.

Information regarding the properties of construction materials is coded as the probability of locating suitable materials within any particular analogue. It is thus possible to conduct the search for road building materials in a logical fashion and at a saving in cost, both for the time spent in prospecting and more especially because most usable sources are located.

Design and Construction Standards

It is of importance, from an economic viewpoint that a flexible approach be adopted towards the setting of design and construction standards for low cost roads, whilst at the same time recognising that geometric standards will remain for the life of the road, which may be considerable, since the initial tertiary roads will develop into the future main communication links of the country.

In addition local knowledge regarding the performance of pavements using the naturally occurring materials, as well as the construction and maintenance capabilities of the local populace, must be assimilated in determining construction specifications. Various aspects should be kept in mind when making decisions in this respect. They include, inter-alia, topography, geology, drainage, traffic and subgrade characteristics together with other aspects such as the socio-economic climate, prospects for obtaining future finance and maintenance capabilities.

The original network of National Roads for South Africa constructed in the late thirties and during the forties has provided very good service, and in many parts is still doing so following many reseals, nominal widening, the addition of climbing lanes and the application of thin bituminous overlays. Based on this experience over 40 to 50 years the following categories in the development process are suggested for low cost roads in developing countries.

1. Graded and shaped roads (in-situ materials)
2. Graded and shaped gravel roads (imported natural gravel)
3. Bitumen surfaced, natural and stabilised, gravel roads.

Typical Design Standards and Applications

The following considerations must be taken into account in formulating design proposals:

Administrative.
1. Economic considerations.
2. Alignment and services (land use).
3. Construction planning.
4. Axle loadings.

Traffic.
1. Data sources
2. Growth projections
3. Functional classification

Climate.
1. Rainfall and run-off
2. Temperature extremes

Geometrics.
1. Design speed
2. Horizontal, and especially vertical alignment.

Availability of Natural Road Building Materials
1. Data collection procedures
2. Special problems (eg swelling clays, swamps, shifting sand dunes)
3. Treatment of problem areas.
4. Treatment of problems and low quality materials.
5. Evaluating existing road foundations
6. Treatment of existing road foundations where problems occur
7. Pavement design

Drainage.
1. Type of culverts
2. Surface drainage
3. Subsurface drainage
4. Prevention of ponding

Graded and Shaped Soil Roads (in-situ materials)

Design Considerations. This is the first formal step in forming a road from the common veld-track, and it is important for it to comply with the following basic requirements:

1. It should follow the shortest possible route, using the most effective and economical alignment. Consideration can be given to a so-called watershed alignment in order to limit drainage structures. Particular attention should be paid to the fixing of the alignment to alleviate false rise and excessive gradients since this appreciably affects traffic capacities, particularly in situations where the percentage of heavy vehicles is expected to be large.
2. Considerations for drainage and reasonable all weather trafficability should be made.
3. Gravel should be placed only in difficult or rough sections (usually over pipe culverts).
4. Light traffic: not more than 30 vpd.

Construction. Construction of these roads is straightforward and the equipment required can be anything from a mule-grader or an ox cart to the latest equipment available. Hand labour can form an important phase in the construction of these roads. Construction costs can, depending on circumstances and equipment used, vary from a few hundred dollars per kilometre to about 3 000 dollars per kilometre. The local populace and their inherent skills should be used to the maximum extent and as much recource as possible should be had to handwork.

Maintenance. Maintenance will be dependant on the volume and type of traffic as well as the climate (seasonal). Occasional shaping with a grader - often by request of the user - seems to be the most economic solution. In the sandy conditions found in South West Africa (Namibia) a so called sand track grader has been found to be particularly useful in straightening the winding furrows in the road caused by the passage of vehicles.

In some instances, the maintenance and construction of these roads is left to the users or a system of subsidized maintenance is adopted. In the Orange Free State, a province of the Republic of South Africa, $325 000 and $360 000 were spent in the fiscal years 1970/71 and 1971/72 respectively on approximately 25 000 km of tertiary roads, ie about 12 dollars per kilometre per annum (3).

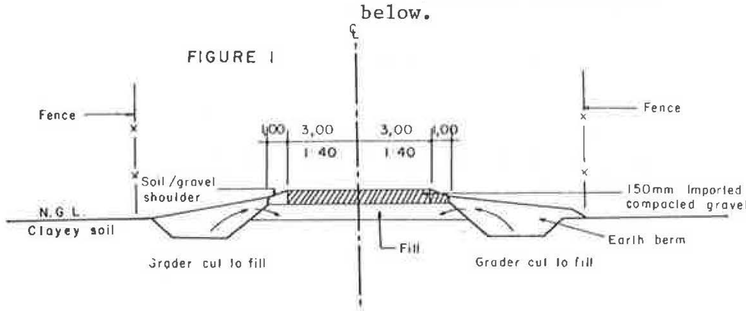
Shaped and Graded Gravel Roads

Roads in this category provide an important service and can link smaller rural communities or tie into the major road network serving the area. Unpaved roads remain an important part of the South African road scene: in 1974, 143163 km (77 percent) of the Republic's rural road network was unpaved, while in Rhodesia 26322 km (78 percent), and in South West Africa 28815 km (90 percent) remain unpaved (4).

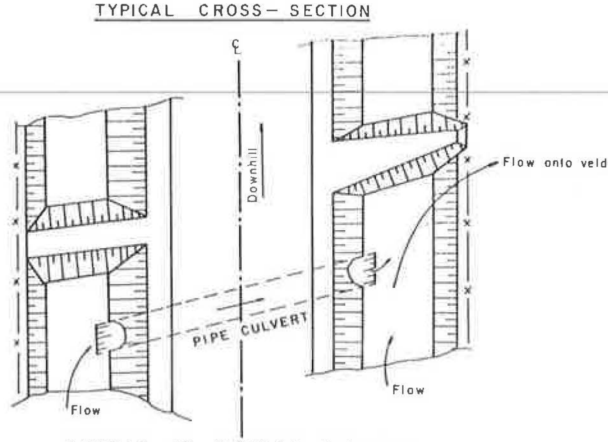
Design Considerations.

Traffic volume	:	50 - 150 vehicles per day.
Design speed	:	80 - 90 kilometres per hour, depending on terrain.
Gravel wearing course	:	5 to 7 m wide, 150 mm compacted thickness.
Crossfall	:	1 in 40.
Shoulder width	:	1 metre minimum (fill material).
Materials		
Fill	:	in-situ materials mostly from side drains or sources as near as possible.
Gravel wearing course	:	material and compaction standards to be controlled, and specified as detailed below.

FIGURE I



TYPICAL CROSS-SECTION



DETAILS OF SURFACE DRAINAGE

Construction. Fill material is obtained from sources as near as possible to the road, generally from within the proclaimed road reserve. The latter method also facilitates drainage of the road. It is advisable and economical to allow construction equipment and traffic to compact the fill prior to placing of the gravel wearing course. Often the earthworks will have been constructed and used by traffic as a lower category road. The so-called 'Dig, load and haul' contracts for placing of the gravel wearing course have been carried out with great success by the provincial roads authorities in Natal and the Orange free State, two provinces of the Republic of South Africa.

Maintenance. Depending on the type and volume of traffic, the life of the gravel wearing course is greatly influenced by the regularity of grader maintenance and patching with gravel in places where the wearing course becomes thin.

An important factor is the replenishing of the fines in the wearing course from the shoulders or sides of the road, where a good grader operator will always leave a windrow for this purpose.

It has been found that dust settles onto the veld next to an average gravel road at a rate of 32 kg per hectare per month, creating an environmental hazard for crops and stock (wool sheep) next to such roads (5). In South Africa the potential dangers of thick clouds of dust from traffic is well known and create a limit to the volume of traffic that can be handled on these gravel roads, particularly in dry climates.

The life expectancy of roads in this category before major maintenance is required, can vary from 2 to 7 years with traffic volumes of about 200 to 50 vpd respectively. The first gravel wearing course usually has a shorter life owing to consolidation and compaction under traffic, and it has been found that a second course added after 2 to 3 years considerably increases the life of the road.

Costs. Approximately 500 - 600 dollars per kilometre can be spent annually on the maintenance of a secondary road; construction costs vary from 5 000 to 12 000 dollars per kilometre with construction units consisting of one or two graders, medium bulldozer, front end loader, and 6 to 8 tip trucks, Compaction equipment is limited to the absolute minimum or where conditions necessitate special attention. Production can reach up to 10 km of completed gravel road per month using a small construction unit.

Selection of Materials for the Wearing Course

The selection of materials for gravel road construction is predicated on a basis of experience rather than structural pavement design concepts and the criteria are related to rate of loss of gravel and the ability to permit the safe passage of traffic in wet weather. An investigation into the capabilities of various wearing course materials, used over many thousands of kilometres and related to the more common road materials tests has led to the following standards being derived for materials used as wearing courses 150 mm thick, compacted to an in-situ density of 95% of the Modified AASHTO effort maximum density, and constructed directly over a prepared subgrade of local in-situ materials (6):

Abrasion. One of the most important criteria has been found to be the resistence to abrasion measured as the loss after 500 revolutions in the Los Angeles Abrasion Machine. In order to provide a reasonably smooth ride in dry weather, and at the same time not to cut up in wet weather, experience has shown that a value between 30% and 60% provides the best results. Materials with a value of less than 30% generally do not provide sufficient fines to meet grading requirements.

Strength. The California Bearing Ratio value should not be less than 20 when determined at the in-situ density.

Atterberg Limits. The following limits are recommended:

Lower Liquid Limit : 20 - 35 (ideally near 30)
Plasticity Index : 5 - 15

It has been found that in the drier regions of Southern Africa, a plasticity index as high as 22 has given satisfactory results especially in controlling corrugations. The rainy season is usually of relatively short duration and slipperyness does not create undue problems. Relaxations of this nature can also be considered when pedogenic materials such as calcretes are used.

Grading. The following grading envelopes shown in Table 1 (after 95% Mod AASHTO compaction effort) have been found to be suitable for gravels: (Sand Clay mixtures do not necessarily have to meet these envelopes but should comply with the other requirements).

TABLE 1 : GRADING REQUIREMENTS FOR NATURAL GRAVEL WEARING COURSE MATERIALS

Sieve Size	Percentage Passing by Mass		
	37,5 mm max size	19,0 mm max size	13,2 mm max size
37,5 mm	100		
19,0 mm	70 - 100	100	
13,2 mm	60 - 85	75 - 100	100
4,75 mm	40 - 60	50 - 75	60 - 100
2,00 mm	30 - 50	35 - 60	45 - 75
0,425 mm	15 - 40	18 - 45	25 - 50
0,075 mm	7 - 30	7 - 30	7 - 30

The grading of material should run parallel to the limits of the relative grading envelope which applies to material subjected to the 95% Mod. AASHTO compactive effort and 4 days soaking. When the grading of the material is too coarse and lies outside the envelopes correction of the deficiency in fines by the blending in of a suitable binder from the in-situ earthworks is often of assistance.

Performance. Table 2 indicates the behaviour tendency of wearing course materials.

TABLE 2 : PERFORMANCE PROPERTIES OF NATURAL GRAVEL WEARING COURSE MATERIALS

Performance	Lower Liquid Limit	Plasticity Index	Coarse plus Coarse fine sand content	% clay
Corrugates	less than 20	-	greater than 55	-
Dusty when Dry	less than 20	-	less than 30	-
Ravels when Dry	less than 20	less than 6	-	less than 6
Potholes when Wet	greater than 35	-	less than 30	-
Slippery when Wet	-	greater than 15	-	-
Cuts up when Wet	-	-	less than 25	greater than 10

In general the following characteristics apply to the gravels more commonly used in South Africa:

Dolerites	: High coarse sand content	- Tend to corrugate
	Low coarse sand content	- Tend to pothole and become slippery
Laterites (Ferricretes)	: Low Plasticity Index	- Tend to corrugate
	High Plasticity Index	- Tend to pothole
Sandstones	: Low plasticity Index	- corrugate and erode easily
	High plasticity Index	- Tendency to pothole
Shales	: Low plasticity Index	- If soft - dusty
	High plasticity Index	- Tend to pothole

Blending of Natural Materials

Blending of naturally occurring materials can be highly beneficial in improving strength and maintenance free life. In the semi-arid regions of Southern Africa natural gravels tend to be low in fines content and with plasticity indices of around 10. This combination of properties is not desirable for gravel wearing courses since it leads to a tendency to corrugate. If such a gravel is blended with a sandy clay soil (15 to 20% by mass) a much more substantial wearing course results.

On the other hand where naturally occurring gravel deposits are at a premium it has been found that blending (5 to 20% by mass) of natural gravel with sandy soils improves compactability dramatically.

Bituminous Surfaced Roads

The next stage, usually dictated by traffic growth and general development of a region, is the upgrading of gravel roads into roads with a bituminous surfaced pavement, constructed with sufficient structural capacity to carry the increasing traffic loads.

The additional costs required to upgrade a natural gravel wearing course road into a bituminous surface road as described below is of the order of fifteen to twenty thousand dollars per kilometre.

The philosophy proposed for developing countries is that such roads should be constructed using equipment which is as simple as possible, and which in addition can be used for other purposes, such as agriculture. Consequently as much use as possible should be made of equipment such as disc harrows and trucks, both of which are able to play their part in the other sectors of the economy.

There should be as little recourse as possible to sophisticated operations such as crushing of rock to conform to the tight gradings commonly specified for crushed stone base, or for the multi-stage crushing operations required to produce good quality aggregate for surface treatments.

By the time it becomes necessary to provide a bituminous surfacing to the pavements the existing gravel roads will generally have been subjected to traffic and the environment for a number of seasons. Areas of water seepage will have been identified and rectified and a stable ground water regime will have been established. In addition most of the settlement in embankments will have taken place and soft spots in the subgrade will have been repaired. The subgrade which is one of the most important factors influencing pavement performance will by now have "bedded down".

Recognising the great economic advantages in respect of performance attendant upon providing a degree of stiffness in the pavement by means of cementitious stabilisation, practice in respect of structural road improvement in South Africa has generally been to stabilise the top 150 mm of the existing wearing course materials on gravel roads using either lime or cement, as a first step in the upgrading of the pavement. The most appropriate equipment for the adequate mixing in of the stabilising agent is the disc harrow, drawn by a sufficiently powerful tractor to operate at the speed required to turn the material under the discs.

Assuming that the wearing course material conforms to the standards previously given, CBR values in excess of 60 percent should easily be attained by the addition of 3 to 4 percent by mass of either lime or cement, the agent used depending upon the plasticity of the wearing course material. For plasticity indices of the order of 8 or more lime is preferable because of the greater time available to complete the mixing and compaction process.

It is common to differentiate between two classes of composition of traffic in determining the necessity or otherwise for additional pavement layers. Where traffic consists predominantly of motor cars it would generally be sufficient to immediately provide a surfacing to the stabilised wearing course. However the more normal case in rural areas is where traffic consists largely of heavy vehicles, such as buses and agricultural haulage vehicles. In such instances it is advisable to provide an additional 150 mm layer of stabilised base material - particularly in low lying areas or areas of poor subsoil drainage.

Pavement Standards. For stabilised base materials the following standards after stabilisation are considered appropriate:

Minimum CBR	: 80 at 95% mod AASHTO (or 0,75 MPa, Unconfined Compression Strength)
Maximum Plasticity Index	: 6
Maximum Linear Shrinkage	: 3
Grading	: A grading envelope is not considered necessary for stabilised layers.

For unstabilised base materials the following standards should be sought:

Grading : Minimum grading modulus 1,5

$$(GM = \frac{\%\text{ ret on 2 mm}+0{,}425+0{,}075}{100})$$

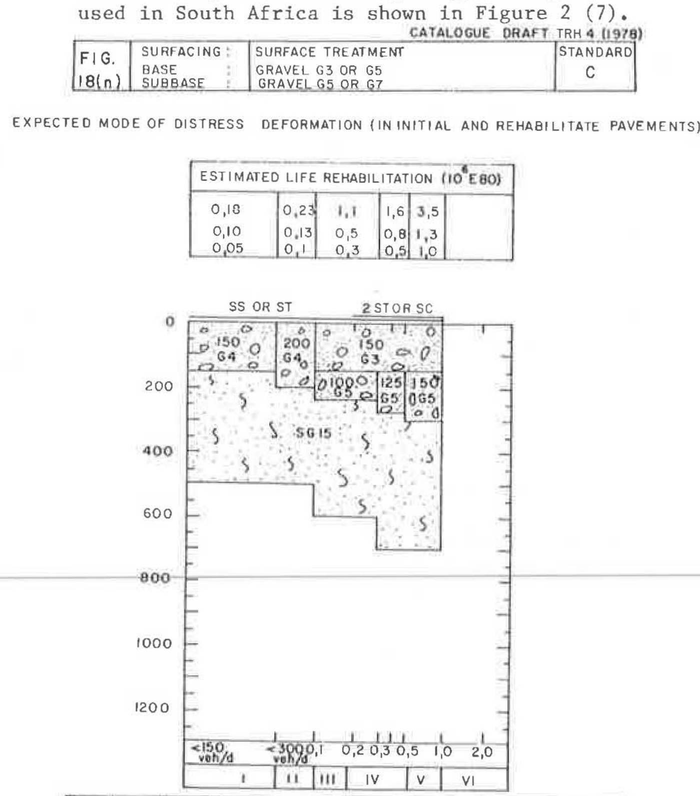
Plasticity Index:	Not greater than 12 (pedogenic materials 15)
Minimum CBR :	For traffic consisting mainly heavy vehicles: 60 - 70 percent. For traffic consisting mainly of motor cars: 50 - 60 percent.

The above standards may be relaxed when subgrades with a CBR value in excess of 25 are encountered.

When two layers of stabilised material are provided, the lower layer should have a minimum CBR value of 35 to 45 percent.

Appropriate surfacing widths are 5,5 to 7,0 metres, with unsurfaced shoulder widths of 0,5 to 1,5 metres. Generally, the shoulder material would not be stabilised and should conform of the standards given for wearing course materials.

An example of a catalogue type design as used in South Africa is shown in Figure 2 (7).



Symbol	Code	Material	Abbreviated specifications
	G3	Gravels including modified and processed gravels	PI ⊁ 6, CBR ≮ 80
	G5		PI ⊁ 10, % passing 2 mm ⊁ 30, CBR ≮ 45
	G7	Selected gravel	PI ⊁ 12, CBR ≮ 20

FIGURE 2

Bituminous Sand Seal Surfacings

Once traffic volumes on gravel surfaced roads increase, it becomes essential for economical maintenance and safety to have an all weather surfacing. Sand seal surface treatments have found wide application in many parts of Southern Africa and this type of treatment has proved itself over many years of usage over many thousands of kilometres.

A sand seal consists of a cutback bitumen, emulsion, or tar tack coat, and a suitably graded sand applied to a base course with or without a prime coat. The use of a prime coat depends on the type and quality of gravel used as well as traffic volumes.

It must be understood that with a sand seal the actual surface of the base forms the contact surface with the traffic. If the availability of good quality gravel is a problem or only available at great cost, because of long hauls, modifying the top 75 mm of the base by adding a good quality natural gravel or crushed aggregate to "reinforce" the top of the base prior to applying a seal coat is recommended.

Where good quality natural quartzitic sand is available, a sand seal presents an extremely economical solution to a surfaced road for light traffic and can safely be used for traffic volumes of the order of 300 vehicles per day to 500 vehicles per day in drier climates. It is essential to note that sand seal treatment is particularly dependant on the quality of the base course and will fail disastrously when -

1. the base is of a poor standard,
2. the road foundation is poorly drained and,
3. if the road has to take a substantial volume of heavy truck traffic at any stage.

Prior to spraying the tack coat a prime coat consisting of tar or cutback bitumen may be applied to improve the quality of the seal, sprayed at approximately 1,00 litres per square metre. A tack coat, consisting of an emulsion or cutback bitumen, may be used sprayed at 1,5 to 2 litres per square metre.

The sand can be applied manually with spades from a truck or by using the conventional push type chip spreader modified to allow a uniform sand carpet to be spread onto the road surface. The sand can be over applied in order to create a layer of sand of 3 - 5 mm in thickness on top of the base. If no prime coat is used allowance must be made for the tack coat to establish cohesion with the base surface with initial penetration, and an over application of the sand will tend to retard or stop this bond to the base. Each layer of sand must be spread at approximately 170 square metres per cubic metre. Grading requirements for sand are shown in Table 3.

TABLE 3 : SAND SPECIFICATION FOR SAND SEAL SURFACINGS

Sieve Size	Cumulative percentage by mass through sieve
6,7 mm	100
1,18 mm	40 - 65
600 um	10 - 35
300 um	0 - 15
150 um	0 - 2

Initial rolling by means of a 5 ton pneumatic roller is advisable, however if a roller is not available a lightly loaded tip truck, immediately behind the sand spreader is essential for setting the sand in the tack coat.

It is recommended that the excess sand whipped off by traffic be broomed back once or twice, particularly if bleeding is encountered. After a period of say 3 months it is recommended that a second seal be applied to rectify any deficiencies shown by the initial seal coat.

The construction of a "thickened edge", roughly 50 mm deep and 80 mm wide cut into the base and filled with clean (screened) natural gravel, graded 7 mm to 2 mm and saturated with bituminous binder creates an effective cut-off to moisture that may enter the base from the natural gravel shoulder.

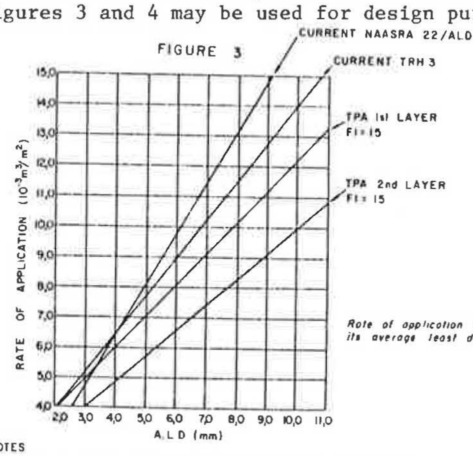
The estimated life between reseals, before general surface deterioration, potholing and edge ravelling sets in, is between 4 and 5 years. The cost of a sand seal treatment has been found to be in the region of one and a half dollars per square metre.

Bituminous Surface Treatments using crushed or natural Rock

Although bituminous surface treatments (generally referred to as "chip and spray" treatments) are not generally considered for this category of road, mention is made of them because suitably crushed or screened rock may be available as bye-products of other engineering activities. In some instances a lack of suitable sand may dictate the use of imported rock chips.

This process is the most common surface treatment for primary roads in Southern Africa and has been developed to a high degree of sophistication. If considered for a lightly trafficked road however, a so-called "single seal" will be applied on the primed base (as for sand seal) sprayed with a first spray of bitumen (penetration grade, cut back or emulsion) followed by 7 mm or 13,2 mm chips and a second application of binder.

The maximum amount of stone that can be retained, is contained in the single layer that comes in contact with the binder film; any over application of stone is whipped off and may be a hazard to traffic (windscreen breakages resulting). The rate of binder application must be related to the average least dimension (ALD) of the stone. It is common practice in South Africa to split the binder application by applying half the initial spray prior to the spreading of the stone chips and then spraying the remainder as a seal coat. This results in much improved stone retention. Figures 3 and 4 may be used for design purposes (8).



Rate of application of stone related to its average least dimension (A.L.D.)

NOTES

NAASRA : NATIONAL ASSOCIATION OF AUSTRALIAN STATE ROAD AUTHORITIES

FI : FLAKINESS INDEX

THE LINES ON THIS GRAPH REPRESENT THE DESIGN USED BY THE VARIOUS AUTHORITIES MENTIONED AND AS SUCH INDICATE THE UPPER AND LOWER LIMITS OF STONE APPLICATION RATE THE CURRENT TRH 3 IS THE ONE RECOMMENDED FOR USE

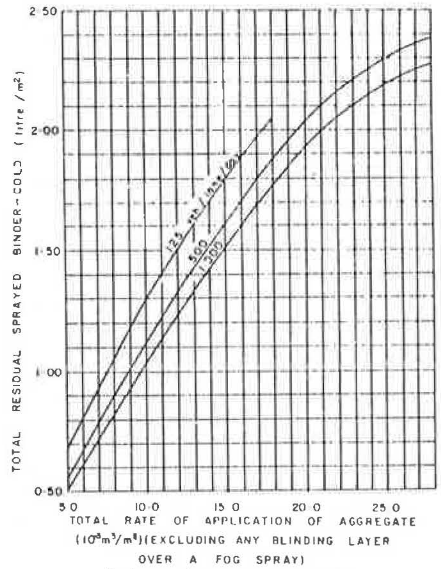


FIGURE 4

Total rate of application of binder for single and multiple surface treatments with stone

Drainage of Lightly Trafficked Roads

The reduced geometric standards generally associated with low cost roads allow better utilisation to be made of watershed alignments and in this way minimise the number of drainage structures required as well as reducing the necessity for surface drainage facilities. The watershed is also normally located on the better class materials and this approach therefore has the added advantage of a resulting better subgrade condition.

The process of shaping the road, using the in-situ materials within the road reserve to provide adequate drainage has resulted in improved performance of many kilometres of roads in Rhodesia and South Africa.

Careful consideration should be given to the sophistication of the structures used for the type of road. It has been found that the local manufacture of concrete pipes is advantageous and preferable to box culverts, and much easier to install.

Causeways or semi-low level bridge structures render roads servicable for the greater part of the year provided that flood levels are calculated to allow fordable water depths over the structures for average type vehicles (approximately 450 mm). Another approach, especially in flat marshy or vlei areas, with only the odd seasonal flood, but with permanent slow flowing water, is to provide a battery of concrete pipes adjacent to a fairly long (approximately 100 m) coarse gravel causeway at a slightly lower-level, with grouted stone pitching, particularly on the down stream face.

Special Problems, Techniques and Materials

It is on projects in this category that engineering judgement and ingenuity is challenged with each road providing its own host of special problems; this underlines the fact that the designer must have an intimate knowledge of local or similar conditions.

Damage to Roads caused by Soluble Salts

In the construction of low cost roads, it is desirable that the most economical use of materials and particularly of water sources in arid conditions be made. Soluble salts in the natural gravels and free water is common in many areas of Southern Africa and is often deleterious to bituminous binders if used for thin seal coats.

Netterberg and Maton reported to the Sixth Regional Conference for Africa and Soil Mechanics and Foundation Engineering in 1975, (9), that damage due to soluble salts had been experienced on at least 16 road construction jobs in South Africa during the past 10 years and that similar problems have also been reported from Botswana and South West Africa (Namibia).

The earlier symptoms of future distress generally occur during or soon after construction and take the form of powdering of the surface of the newly compacted base or of the prime layer itself and/or the deposition of whitish salt in blisters, cracks on the surfacing or along the edge of the surfacing. In the most severe cases heaving and cracking of the surfacing occurs and the whole base may become loose.

Damage can be prevented by limiting the content of very soluble salt, either by identification through testing, or in certain cases by the addition of high calcium content lime. The three most important factors affecting the occurrence of salt damage appear to be:

1. the presence of soluble salts in the base, subbase or subgrade,
2. the length of time an unprimed or primed base is left unsurfaced,
3. the permeability of the surfacing.

High calcium lime treatment should only be used when the salts present in a material are known and furthermore this action appears necessary only if the pH is less than 6,0 and the total soluble salt content is more than 0,2 per cent.

Sea water has been used successfully in the coastal regions of South West Africa for base compaction and reasonable success has been attained with an early application of the bituminous seal coat (within two weeks). Generally however the use of sea water for compaction should be restricted to layers deeper than 0,5 metre below the surfacing.

The Use of Calcretes in Road Foundations

The occurrence of pedogenic rock and in particular calcretes, is common to many countries in the world and Southern Africa is fortunate in having a variety of types occurring over an extremely wide area. In the RSA calcretes are the third most commonly used road building material (10). Calcretes vary from a fine powdery substance to crushable boulders and can be classified into 6 basic types, easily recognised in the field by untrained personnel, viz, calcified soils, powder calcretes, nodular calcretes, honeycomb calcretes, hardpan calcretes and boulder calcretes (10).

Prospecting Methods

Air-photo interpretation is a very useful preliminary tool, because calcarious deposits usually show up well. Another method is infra red aerial photography because the hygroscopic moisture content of calcarious materials is higher than that of the surrounding ground. (Borne out by the fact that small burrowing animals usually dig their holes in these sources - cooler). Ground checking of aerial photography is essential in order to ascertain the quality of the source. For this purpose a valuable tool has been developed by a road inspector (see fig 5).

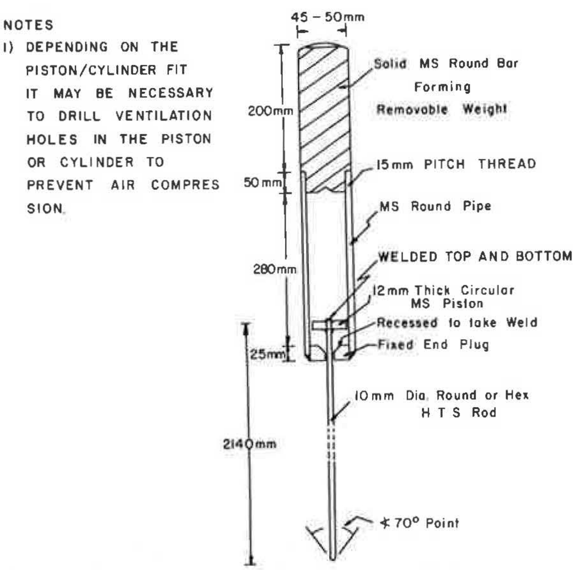


FIGURE 5

RAPID CALCRETE PROBING DEVICE

The tool is simply hammered into the ground to its full reach or to refusal and then withdrawn. Any calcrete encountered will stick to the paint and with practice the quality can be deduced. This probe is about ten times faster than any probe or auger in these conditions.

Engineering Properties of Calcretes

Particle strength of calcretes is usually low and normal grading analysis must be regarded as unreliable. The normal Atterberg-limits of soils must also be applied with extreme care, because practice has shown that calcretes with PI's as high as 15 - 20 can still be used as natural base courses for highly trafficked roads in arid areas.

Two other simple field tests have been devised and reported by Netterberg (10) i.e. the Aggregate Pliers Test that can be used by any field worker using a pair of normal pliers and good judgement in order to establish an approximate 10% FACT value. Similarly softer calcretes can be crushed by the worker's fingers - Aggregate Finger Value (AFC).

It is also well known that a substantial increase in strength of calcareous layers is experienced with time, ascribed to self-cementation.

The successful uses of calcrete in road foundation layerwork have been proven over many years in Southern Africa. In many instances the material used would not comply with the usual AASHTO, British and South African standards for the particular application, however, perfectly satisfactory flexible bases for light and medium trafficked surfaced roads under relatively thin seal coats have been constructed. In the light of these findings it seems reasonable to relax certain materials standards in order to provide a more economically locally suited facility.

Conclusion

The secondary and tertiary legs of the road infra-structure in many developing countries are often neglected thus impeding development and the national economy. In order to utilise natural materials as well as manpower to its fullest possible extent, knowledge of local conditions is essential.

This has the effect of reducing costs to a minimum and consequently the dependance of developing countries. Donor countries offering this aid generally have well developed economies and infrastructural facilities, hence appropriate expertise is not always available within these countries. In such instances it is advisable to consult with countries who have recently passed through this phase of development.

Acknowledgements

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DESIGN OF EMULSIFIED ASPHALT-AGGREGATE BASES FOR LOW VOLUME ROADS

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This paper briefly summarizes procedures that have been developed for both mixture and structural design of emulsified asphalt-aggregate bases for low volume roads. The procedures are based on laboratory, analytical, and field studies. The Marshall equipment, resilient modulus, and a capillary moisture soak test are used for determining structural and durability properties. The mix design procedure determines the following: (1) suitability of aggregate and emulsified asphalt, (2) compatibility of emulsified asphalt and aggregate (including acceptable range of premix aggregate water content), (3) optimum moisture content at compaction, (4) optimum residual asphalt content, and (5) adequacy of mixture structural and durability properties. These procedures are developed specifically for dense graded cold mix base courses of low traffic volume roadways. The use of local aggregates has been particularly emphasized in this study. A method to relate the cold mixture structural properties, as determined from laboratory tests, to the "structural coefficient" of the base course was developed. Data obtained from specially cured specimens during the mixture design are used to determine the material's structural coefficient. The structural coefficient of the base is then used to select the required structural thickness using the AASHTO Interim Guide or similar design procedure. The procedures were evaluated over a range of actual in-service pavements and materials, and found to give satisfactory results.

Stabilization of granular base materials, particularly substandard aggregates, with emulsified asphalt has increased in recent years in Illinois and other states because of economical and environmental advantages. Field performance has been generally good, but because of lack of knowledge of mixture and structural design, and construction criteria, many problems have occurred.

Mixture design for cold EAMs has largely been based on field experience and simple laboratory mixing tests to estimate proportions of materials. Some emulsion producers have over the years developed simplified mix design methods such as McConnaughay (23) and Armak (18). A few more detailed methods have evolved in the past few years including Chevron (7, 17), U. S. Forest Service (24), California (13), Asphalt Institute (8), Chevron (9), the Asphalt Institute Pacific Coast Division (20), and Purdue University (22). Most of these procedures are very similar to each other and based mainly on hot mix design procedures. These procedures do not necessarily optimize the material proportions, but only attempt to meet certain minimum criteria such as percent coating, or stability. Since many additional emulsified asphalt-aggregate mixture (EAM) projects will be constructed on low volume roads in the future, there is a great need for standardized and verified design procedures.

A major effort has been underway at the University of Illinois, sponsored by the Illinois Department of Transportation and the Federal Highway Administration, to develop practical mixture and structural design procedures that could be used by various governmental agencies and others for low volume roads. These procedures have been completed and field tested. It is the objective of this paper to briefly describe the design procedures and practical results obtained from laboratory and field studies. Detailed descriptions of the research results and design procedures and background on their development are contained in References 1, 2, 3, 4, 5, and 6.

EAM Mixture Design

The mixture design procedure involves the following major parts:

1. Aggregate quality tests.
2. Emulsified asphalt quality tests.
3. Estimate of amount of asphalt emulsion content.
4. Compatibility of emulsion and aggregate.
5. Optimum water content at compaction.
6. Selection of optimum asphalt content.

Aggregate Quality Tests

Tests are conducted to determine aggregate properties and general suitability. Field experience has shown that a wide variety of aggregates can be successfully used in EAM bases. These aggregates include crushed stone or gravel, pit or bank run gravel, slag, sand, and silty sand. Use of substandard aggregates (that do not pass current specifications) have made satisfactory EAM bases on low volume roads in Illinois. Only aggregates containing excessive amounts of clay and certain hard to coat aggregates have caused problems in cold EAMs. Excessive clay results in difficulty in mixing due to severe ball-up of the emulsified asphalt, a longer time period required to gain strength, and a relatively large amount of residual asphalt content required for strength and durability requirements. A washed sieve gradation is required to determine the actual amount of fines in the aggregate.

Perhaps the most useful and simple test is the Sand Equivalent, which is a good indicator of the amount of excessive clay present. A Sand Equivalent value above 35 percent is predominantly granular and no excessive clay contents exist. Most pit run gravels in Illinois are well above this level, provided the soil overburden has not been mixed into the pit. Aggregates having a Sand Equivalent from 20 to 30 percent are much more difficult to stabilize and the amount of free mixing water and mixing procedures are more critical. Aggregates having a Sand Equivalent of less than 20-25 percent are usually not considered suitable for EAMs (7, 8, 9, 10, 11, 12). However, some aggregates have been used in Illinois with Sand Equivalent values of 17-22 with apparent success. Laboratory tests on the compacted mixtures show potential problems, and a large amount of residual asphalt is required (13).

Emulsified Asphalt Quality Tests

Standard ASTM or AASHTO tests and specifications are required:

Anionic	ASTM D 977	AASHTO M 140
Cationic	ASTM D 2394 or	AASHTO M 208

In some cases, additional specifications are required for other emulsion types such as High Float Emulsions (HFE)(13) which have been used in Illinois.

Estimate of Asphalt Emulsion Content

The determination of pre-mixing water content, compatibility of asphalt and aggregate, and optimum water content at compaction requires mixtures containing approximately the optimum residual asphalt content. Based upon several emulsified asphalt mixture design data, a regression equation was derived that gives an approximate optimum residual asphalt content. The information required to use this method is obtained from the washed sieve aggregate gradation.

$$R = 0.00138AB + 6.358 \log_{10}C - 4.655 \qquad (1)$$

where

- R = trial residual asphalt content by weight of dry aggregate, %
- A = percentage of aggregate retained on #4 sieve
- B = percentage of aggregate passing #4 sieve and retained on the #200 sieve
- C = percentage of aggregate passing on the #200 sieve

(Note: Gradation based only on washed sieve gradations.)

The R is rounded off to the nearest half percent to yield the trial residual asphalt content.

Example:

Retained on #4 sieve = 35%
Passing #4 and retained on #200 sieve = 57%
Passing #200 sieve = 9%

$$R = 0.00138 \times 35.0 \times 57.0 + 6.358 \log_{10}(8.0) - 4.655)$$
$$= 3.84\%$$

Thus, the trial residual asphalt content, R = 4.0% by weight of dry aggregate. To obtain an emulsified asphalt content, it is necessary to divide the trial residual asphalt content, R, by the fraction of residual asphalt contained in the emulsion. The following is an example for a CSS-1 emulsion:

Trial residual asphalt content = 4.0
Residual asphalt in CSS-1 emulsion = 65%

$$\text{Trial emulsion content (by wt. dry aggregate)} = \frac{4.0}{.65} = 6.15\%$$

Compatibility of Emulsion and Aggregate

The compatibility between asphalt and aggregate is a major mix design consideration. Two criteria that can be used to judge the compatibility are (1) coating achieved after mixing, and (2) the compacted EAM resistance to moisture (i.e., stripping). Several factors affect coating: aggregate/asphalt electro surface charge, free moisture existing in the aggregate before mixing with emulsion, temperature of materials, and aggregate surface texture. It is believed that coating is an important variable in providing mixture water resistance and strength. Both anionic (including HFEs) and cationic emulsions have been used successfully in Illinois with pit-run gravel and crushed limestone. The HFEs, however, contain up to 7 percent oil distillate that aids in the coating. Emulsion producers of both cationic and anionic (including HFEs) emulsions claim that they can adjust the emulsifying agent to provide satisfactory coating for most aggregates. For particular problem aggregates, however, one type of emulsion may clearly provide superior coating.

The amount of coating achieved depends upon the amount of premix free water existing in the aggregate. The optimum range of premix moisture in the aggregate can be determined by preparing laboratory bowl mixes at various premix moisture contents. A quantity of air dried aggregate is placed in the mixing bowl, and a desired amount of water added and mixed thoroughly with the aggregate. The asphalt emulsion is then added and mixed for a specified time period. To simulate cold mixing, none of the materials should be heated. After the EAM is mixed adequately it is placed on a flat pan until it breaks (as noted by a gradual change in color from brown to black). The coating can be judged visually by several persons and their estimates averaged, and a range of acceptable premix water content determined. In areas where the addition or removal of water is impossible or uneconomical, mixtures should be prepared at the in-situ aggregate water content.

Cationic and anionic emulsions behave differently with regard to coating and the amount of free premix water in the aggregate. Cationics generally require additional free water over anionic emulsion for good mixing, thus cationics can be used with aggregates containing relatively high water content.

Curves in Figure 1 illustrate results for a typical Illinois limestone aggregate using anionic (HFE) and cationic (CMS) emulsions. The anionics frequently do not require much free mixing water for good coating, whereas the cationic type requires some free moisture in the aggregate and it tends to ball up resulting in poor coating. If the anionic mixtures contain too much water, they begin to strip asphalt. Each aggregate/emulsion mix has its own characteristic curve, that must be determined through actual testing. Based on the coating tests, a range of acceptable premix aggregate water content is recommended for construction, considering that 50 to 100% coating is acceptable. If this coating cannot be achieved, the emulsion is rejected.

The determination of asphalt/aggregate compatibility with regard to its resistance to moisture effects is determined by subjecting compacted EAM Marshall sized specimens to free moisture in a test that simulates an in-service pavement base.

Water Content at Compaction

The total moisture content that exists in the CAM at time of compaction has significant effects on the resulting density, voids and stability. Several design procedures recommend that optimum compaction water content should be selected to maximize density, similar to granular materials (8, 9, 20).

There are, however, other factors that must be considered, including the breaking of the emulsion, mixture stability and the residual asphalt content. The water content obtained for maximum density when only water is used may not give the best liquid content to be used for mix design and construction. Results show that both residual asphalt content and water content at compaction affect the resulting stability as shown in Figure 2 for gravel EAMs. There is a specific residual asphalt content and moisture content at compaction that gives optimum stability. If residual asphalt content is held constant, typical results such as Figure 3 are obtained in the field and laboratory. Both show a characteristic peaking curve with an optimum moisture content of 4-6 percent. The loss of stability as moisture content decreases below about 3 percent is very rapid. Observations of mix color indicate that the mix is beginning to break at approximately 3 percent moisture as indicated by a change in color from brown to black. The field mix was initially compacted at about 3.3 percent moisture. Other field and laboratory observations indicate that moisture content for maximum stability and the beginning of breaking of the mix occurs at about the same time. EAMs that are allowed to break significantly before compaction become difficult or impossible to compact in the field. Addition of water will not bring the mixture back to a workable condition.

If the EAM is being placed as a road mix, the mixing process can continue until breaking begins (at optimum water content), and then immediately compacted. If a laydown machine is used, the total lift will begin to break at the top before the bottom because of surface dyring. The best solution is probably to limit lift thickness to 51-76 mm (2-3 in.) so that the entire lift will break at about the same time.

The typical effect of total liquid content on density and stability are shown in Figure 4 for specimens prepared with Marshall equipment. A maximum stability and density can occur at differing total liquid contents. The maximum dry bulk density typically occurs at a higher total liquid content than occurs for stability. These results are similar to hot mix where usually maximum bulk density corresponds to a higher asphalt content than does Marshall stability. The liquid content resulting in maximum density using Proctor compaction is higher than that obtained using the Marshall compaction equipment.

In summary, field and laboratory experience indicates that there is an optimum moisture content at compaction for a given residual asphalt content at which both stability and density will be near maximum. At moisture contents just below this optimum, the emulsion begins to break, and compaction must begin before much additional moisture is lost. If additional moisture is lost the EAM will be difficult to compact, and density and stability will be reduced greatly. Also if compaction is begun wet of optimum, the mix will not compact since all voids will be filled.

Selection of Optimum Residual Asphalt Content

Using the required mixing water and optimum compaction water content, mixtures are prepared at varying residual asphalt contents. If the optimum compaction water content is lower than the minimum required mixing water content, aeration is required before compaction. The mixtures are then compacted into Marshall specimens and air cured for three days. The specimens are tested for bulk density, modified Marshall stability, and flow. Moisture susceptibility of the mixture is evaluated by subjecting a series of specimens to a special capillary water soak test for four days (referred to as soaked tests). The typical effects of residual asphalt content on mix properties are shown in Figure 5. The test property curves shown have been found to vary considerably between aggregate types and gradations. General trends are described as follows:

1. The one day dry stability will generally show a peak at a particular moisture content at compaction for a given residual asphalt content. Sometimes this curve is very flat and no peak is apparent, indicating a wide range of possible compaction moisture contents.
2. Soaked stability will generally show a peak at a particular residual asphalt content while dry stability will generally show a continually decreasing curve with increasing residual asphalt content. Some mixes may show a continual increase in soaked stability over the range of asphalt content evaluated, which indicates the increased beneficial effect of additional asphalt content on soaked stability.
3. Percent stability change is computed by (dry stability - soaked stability)100/dry stability. The amount of loss of soaked as compared to dry stability decreases as residual asphalt content increases.
4. Dry bulk density generally peaks at a particular residual asphalt content.
5. Percent moisture absorbed during the soak test decreases with increased residual asphalt content.
6. Percent total voids (air plus moisture) decreases as residual asphalt content increases.

The capillary absorption (or soak) test is believed to be the most realistic test available that represents field moisture conditions of an EAM base course. Extensive use and evaluation of the test has shown it to be very simple, convenient, and realistic test (13, 3). The only disadvantage is the relatively long soaking time required. Based upon experimental testing, a 4-5 day soak is believed adequate to provide a realistic indication of the moisture durability of the EAM. This test is shown in Figure 6.

Based upon results from field and laboratory studies (1, 2, 3) in Illinois and other studies (14, 15, 16, 17, 18, 9, 19, 4, 21, 11) the following design criteria are considered important in selecting the optimum residual asphalt content:

1. EAM must provide an adequate stability when tested in a "soaked" condition to provide adequate resistance to traffic load during wet seasons. There is considerable free moisture available in Illinois. Most subgrade soils are poor draining and most low volume roads are constructed with poor drainage characteristics (i.e., no side ditches, high watertable).

2. The percent loss of stability of the EAM when tested "soaked" as opposed to "dry" should not be excessive. A high loss is indicative of the EAM having high moisture susceptibility and may cause softening and disintegration during wet seasons.

3. The total voids within the EAM should be within a specified range to prevent either excessive permanent deformation and moisture absorption (for too high void content), or bleeding and excessive cost of the residual asphalt from the EAM (for a low void content).

4. Moisture absorption into the EAM should not be excessive to minimize the potential of stripping or weakening the bond between residual asphalt and aggregate.

5. Residual asphalt should provide adequate coating of the aggregate and should be resistant to stripping.

The basic design philosophy is that a residual asphalt content should be selected that meets all of the criteria, and maximizes the soaked stability. Specific design criteria are summarized in Table 1.

Structural Design

Procedures were developed for use in design that related cold mix base structural properties to pavement performance (4). Thus, the structural properties can be measured in the laboratory and the results used in structural design of the pavement using the AASHTO Interim Design Guides, or similar design procedure. The resilient modulus (M_R) of the cold mix base was correlated with the structural coefficient of the base. A stress dependent finite element pavement structural analysis program along with performance data and results from the AASHO Road Test were used to develop the approximate correlation as described in Reference 4.

The structural coefficient for cold mix bases (a_2) is believed to fall between that of non-stabilized granular materials ($\simeq$ 0.11) and hot mix asphalt stabilized ($\simeq$ 0.35). The M_R for these materials ranges between approximately 68,940-206,820 KPa (10,000-30,000 psi) for non-stabilized granular materials to 689,400-6,894,000 KPa (100,000-1,000,000 psi) for hot mix asphalt stabilized granular materials. It is within these bounds (and only for asphalt stabilized materials) that an approximate correlation exists. A correlation curve between the base structural coefficient (a_2) and the base resilient modulus is shown in Figure 7.

The measurement of the resilient modulus of the cold mix requires expensive equipment which many laboratories may not have available. Hence, it is highly desirable that a simpler test such as Marshall Stability be correlated with the M_R test. During the experimental laboratory phase of this study the M_R and modified Marshall Stability tests (at 22.2°C) were conducted on many of the same specimens. A reasonable correlation was found to exist between the two tests (4). Using this correlation a relationship can be established between the modified Marshall Stability (at 22.2°C) and the a_2 coefficient. Such a relationship is shown in Figure 9. It should be emphasized that this is only approximate and that the best procedure is to measure the M_R directly on the specimens.

The structural coefficient of asphaltic cold mixtures used in design is determined as follows:

1. Conduct laboratory testing on compacted asphaltic cold mixture specimens containing the recommended residual asphalt content and compacted at the recommended moisture content. The following alternative tests may be conducted:

Alternative A. (preferred)	Diametral resilient modulus as described in Reference 4.
Alternative B.	Modified Marshall stability at 22.2°C (same standard test procedures except conducted at 22.2°C).

The following sequence of testing should be followed for either alternative:

3 compacted specimens retained in mold and dry cured for 3 days in laboratory at 22.2°C, then tested by alternative A or B;

3 compacted specimens retained in mold and dry cured for 3 days in laboratory and then placed in the capillary soak test for 4 days (specimens are rotated after 2 days) at 22.2°C, then tested by alternative A or B.

This data can be obtained routinely from the mixture design tests previously described.

2. Using the data obtained routinely from the mixture design tests, the resilient modulus or modified Marshall stability values are converted to "design" values using one of the following expressions.

Alternative A. Resilient Modulus

$$M_{R_{design}} = M_f \left(\frac{M_m}{M_d}\right)$$

where

M_f = final average resilient modulus of mixture after long term curing, psi
= M_d x CF

M_d = resilient modulus determined after 3 days of dry cure at 22.2°C

CF = a curing factor (2.0 for construction May-Sept. in Illinois and not sealed for >7 days)

M_m = resilient modulus determined after 3 days of dry cure and 4 days of capillary moisture cure at 22.2°C

Alternative B. Modified Marshall Stability

$$MS_{design} = MS_f \left(\frac{MS_m}{MS_d}\right)$$

where

MS_f = final maximum modified Marshall Stability of mixture after long term curing at 22.2°C lbs
= MS_d x CF

MS_d = modified Marshall Stability determined after 3 days of dry cure at 22.2°C

CF = a curing factor (2.0 for construction May-Sept. in Illinois and not sealed for >7 days)

The CF may range from 1 to over 4 depending on time of construction and when the base is sealed. For general design purposes in Illinois a CF = 2.0 is recommended if the base is constructed during the May-September period and is allowed a few days (>7 days) to cure before an overlay or seal is placed.

MS_m = modified Marshall Stability determined after 3 days of dry cure and 4 days of capillary moisture cure at 22.2°C

3. The $M_{R_{design}}$ or MS_{design} is then used in either Figure 7 or 8.

Base Design Application

A county road in Illinois which has an existing granular surface is being up-graded by stage construction. The first stage includes placing an emulsified asphalt stabilized base course over the existing granular surface, which will be compacted and used as a 101.6 mm (4 in.) subbase.

The aggregate is from a pit located near the project and, if acceptable, would provide considerable economic advantage over hauling in other aggregate. The aggregate properties are given in Table 2 and the emulsion properties in Table 3. The washed gradation reveals a relatively high fines content, which was apparently due to the failure to strip overburden at the pit. A relatively high amount of clay is indicated by the low Sand Equivalent value of 22. The water absorption is excessive and asphalt absorption may become a problem during later pavement life, as asphalt is absorbed into the aggregate and film thickness is reduced. This aggregate would normally be rejected because of the gradation and low Sand Equivalent value.

The optimum residual asphalt content is estimated to be 5.4 percent as computed by Eq. 1. Coating tests were conducted by preparing several laboratory bowl EAM mixtures over a range of premixing moisture contents (i.e., moisture contained in aggregate before adding emulsion) at the estimated optimum asphalt content. The best aggregate coating was obtained at pre-mix moisture contents of 3-5 percent (excluding water contained in the emulsion). Marshall sized specimens were prepared with 5.0 percent residual asphalt (actually 5.4 percent should have been used since that is the estimated optimum asphalt content), and compacted over a range of moisture contents. The specimens were air cured 1 day on a laboratory shelf, extruded from their molds, and tested in the Marshall stabilometer at 22.2°C (72°F). A curve shown in Figure 9 was obtained, with maximum stability occurring at 3.5 percent total moisture content (by weight of dry aggregate). This optimum moisture content at compaction was used for compaction of all other specimens.

Compacted EAM specimens were then prepared over a range of residual asphalt contents. Specimens were tested after 3 days of laboratory air curing, called dry curing, for Marshall stability at 22.2°C (72°F). Other specimens, after the 3 day dry cure, were subjected to 5 days of the capillary soak test (2.5 days on each side of specimens), called soaked curing, and then tested for Marshall stability. Dry bulk density and moisture contained in the specimens were also determined as described in Reference 5. Total voids were computed utilizing these data (13). Results are plotted in Figure 9. Maximum soaked stability occurs at approximately 5.3 percent residual asphalt. There is a large loss of stability between the dry cured specimens and soaked specimens, but the amount of difference decreases with increasing asphalt content. A large loss such as this has only occurred with aggregates having a low Sand Equivalent (<25) and a large amount of fines (minus No. 200 sieve)(i.e., >15 percent).

The residual asphalt content at peak soaked stability is 5.3. The following values of other parameters are obtained from the graphs for this content:

Mix Parameter	Value at 5.3% Asphalt	Limiting Criteria
% Stability Loss	57	50 max
% Total Voids	6.7	2-8
% Moisture Absorption	3.6	4 max
Modified Marshall Stability, lbs	550	500 min
% Aggregate Coating (3-5% premix moisture)	60-70	50 min

All of the criteria except percent loss stability are achieved at a residual asphalt content of 5.3%. A residual asphalt content of 5.6% is required to meet the 50% loss requirement. At 5.6% asphalt all other requirements are achieved. However, the soaked stability and moisture absorbed are very close to the limiting criteria, and thus the mix is questionable.

The following mixture design and construction recommendations are obtained:

1. Residual asphalt content = 5.6% by weight of dry aggregate.
2. Asphalt emulsion content (for an asphalt residue of 70%) = 5.6/0.70 = 8.0% by wt. of dry aggregate, or approximately 19.4 gal emulsion/ton dry aggregate.
3. Pre-mixing water content = 3-5% by weight of dry aggregate.
4. Optimum water content at compaction = 3.5% by weight of dry aggregate (that is, total water content in EAM).

Structrual design of the base makes use of data from the mix design tests at optimum asphalt content. The Marshall stabilities at 22.2°C and 5.6% residual asphalt are as follows:

3 day dry cure = 590 Kg (1300 lbs)
3 day dry cure and 4 day capillary soak = 250 Kg (550 lbs)

The loss of stability after soaking is very significant for this aggregate. The structural coefficient of the base material is determined as follows:

$$\text{Design Marshall Stability} = (1300 \times (2.0)(\frac{550}{1300})$$
$$= 499 \text{ Kg } (1100 \text{ lbs})$$

Using Figure 8 the a_2 = 0.17.

Traffic over the 15 years design life is estimated for the average year as follows:

Passenger Cars	= 413/day
Single Unit Trucks	= 94/day
Multiple Unit Trucks	= 20/day
Total Vehicles	= 527/day

The total 18-kip ESAL over the 15 year period is computed to be 69195 80 KN (18-kip) ESAL. Subgrade soils along the project are A-6 classification and have measured CBR values of approximately 3. The required structural number for the 15 year period is determined using the AASHTO Interim Guide with a Regional Factor of 1.5 to be 2.7. Thus, the required thickness of base is computed as follows (the surface will be 3 in.):

$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

$$2.7 = 0.22(3 \text{ in.}) + 0.17\ D_2 + 0.11 \times 4 \text{ in.}$$

$$\text{Base thick } (D_2) = 239 \text{ mm } (9.4 \text{ in.})$$

This large thickness is required because of the low quality of mixture.

The project was constructed in 1976 using road mix procedures. The actual construction did not exactly follow the recommended design. Three lifts were used: 64, 64, and 51 mm (2.5, 2.5, 2.0 in.) thickness, for a total of 178 mm (7 in.) which is less than that recommended. The water content of the gravel prior to mixing was 5.9 percent (which is greater than the optimum range for coating). The field mix observed just before compaction was estimated to have about 60 percent coating. Water content at the first pass of the roller was about 3.9 percent, which is near the recommended 3.5 percent. Field mix was obtained just before compaction and brought to the laboratory in sealed containers for compaction into Marshall sized specimens and testing. Some results obtained are given in Table 4. These data indicate that the field obtained mix has less stability and greater moisture content than the lab mixtures. This coupled with the low residual asphalt content may cause serious problems for the EAM base. After one year of service the pavement does not show any significant distress.

Conclusions

The construction of cold mix bases has been based on field experience and/or simple mixing tests. This has led to field performance problems in several pavements. Standardized procedures have been developed to improve the design and construction procedure so that acceptable performance can be assured. The procedures have been field tested and found to give reasonable results. Economically, this research can result in a reduction in the cost of the construction of roads carrying low-volume traffic. This will occur when substandard aggregates can be stabilized successfully with emulsified asphalts for the construction of quality bases for economical use of low-traffic roads. A second benefit, that may become quite important in the future, is the conservation of rapidly disappearing, quality aggregate sources. When substandard aggregates are used instead of quality aggregates, the demand for quality aggregates is reduced and thus more quality aggregates are available to be used in construction of higher-type roads.

Acknowledgments

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Table 1. Emulsified asphalt-aggregate mixture design criteria.

Test Property	Minimum	Maximum
Stability, N (lb) at 22.2°C (72°F)		
Paving Mixtures	2224 (500)	
Percent Total Voids		
Compacted Mix (granular mixes, no requirement for sand)	2	8
Percent Stability Loss		
After 4 days soak at 22.2°C (72°F)	--	50
Percent Absorbed Moisture		
After 4 day soak at 22.2°C (72°F)	--	4
Aggregate Coating (%)	50	--

Table 2. Properties of gravel pit aggregates.

	Sieve Analysis		
	Sieve	Dry	Washed
	1 1/2	100	100
	1	98	98
Specific Gravity - Surface dry 2.36	3/4	96	96
Absorption % - 5.0	1/2	90	92
Abration, Los Angeles % Loss 24.4	3/8	86	87
	#4	67	70
	#16	27	39
	#200	4	17
Sand Equivalent (-#40 sieve)	22		
Standard Proctor			
Dry Density 123.7 pcf			
Water Content 9.8%			
Natural Water Content of Pit ≈4%			

Table 3. Emulsified asphalt properties for HFE-300 grade used.

Specific gravity at 15.5°C (60°F)	0.990
Viscosity, Saybolt Furol, at 50°C (122°F)	110.4 secs.
Sieve test, retained on No. 20 sieve	.016%
Settlement	1.4%
Coating test 3 minutes	Passed
Float test at 60°C (140°F)	1200+ secs.
Distillation test to 260°C (500°F)	
Residue from Distilation	70%
Oil distillate, by volume	1.5%
Characteristics of residue from distillation test to 260°C (500°F)	
Specific Gravity at 25°C (77°F)	0.980
Penetration at 25°C (77°F), 100 g., 5 sec.	380+

Table 4. Comparison of field and laboratory data for gravel aggregate project.

Curing	Laboratory Mix Design Stability at 4% Asphalt (N)	Field Mix Stability at 4% Asphalt (N)	Moisture Content at Testing %	
			Lab	Field
3 Day Dry	9786	6174	2.2	2.6
3 Day Dry + 5 Day Soaked	2002	1299	7.0	7.7
		% Water Absorption =	4.8	5.1

(1 lb = 4.448 N)

Figure 1. Illustration of coatings obtained for a typical Illinois crushed limestone for two emulsions.

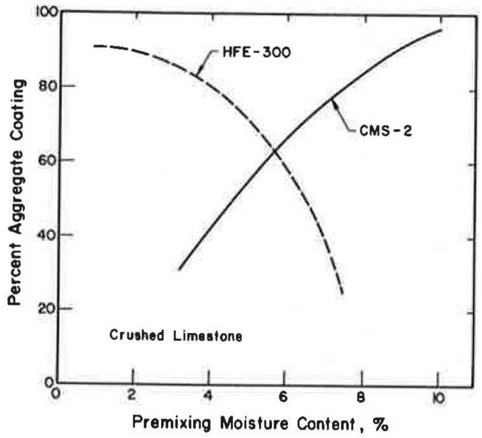


Figure 2. Contours of equal soaked stability for various residual asphalt and compaction moisture contents for gravel aggregate (3-day cure and 75 blows).

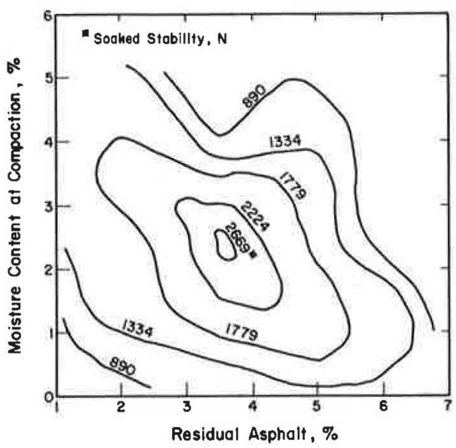


Figure 3. Effect of water content at compaction on Marshall Stability at 25°C (HFE-300, crushed limestone).

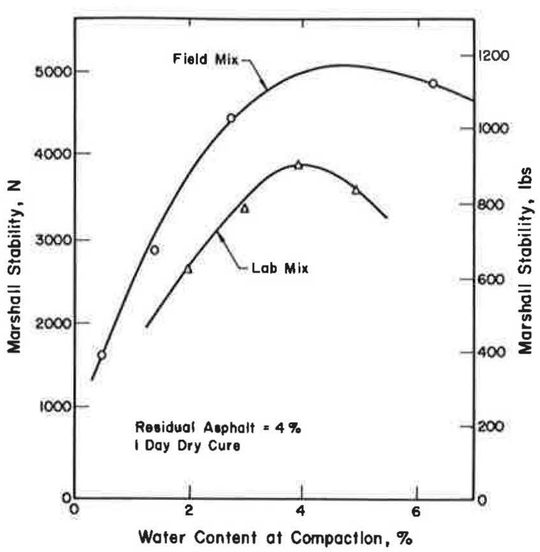


Figure 4. Effect of total liquid (asphalt + water) at compaction of EAM on stability and dry density using Marshall equipment (75 blows)(residual asphalt = 3-6%, water content = 4%).

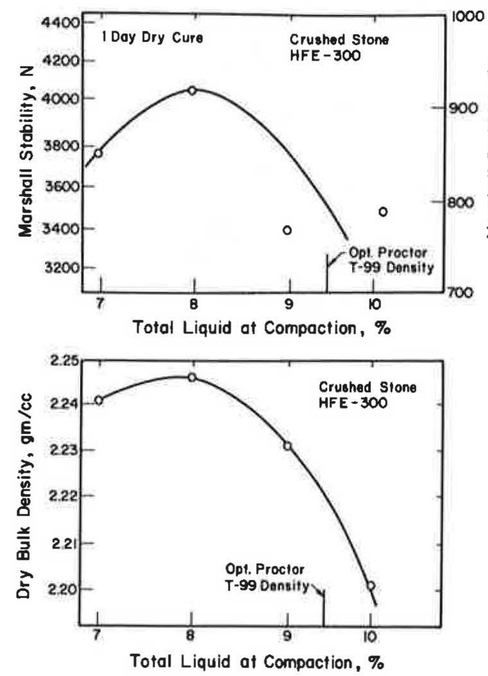


Figure 5. Typical emulsified asphalt-aggregate mixture design plots.

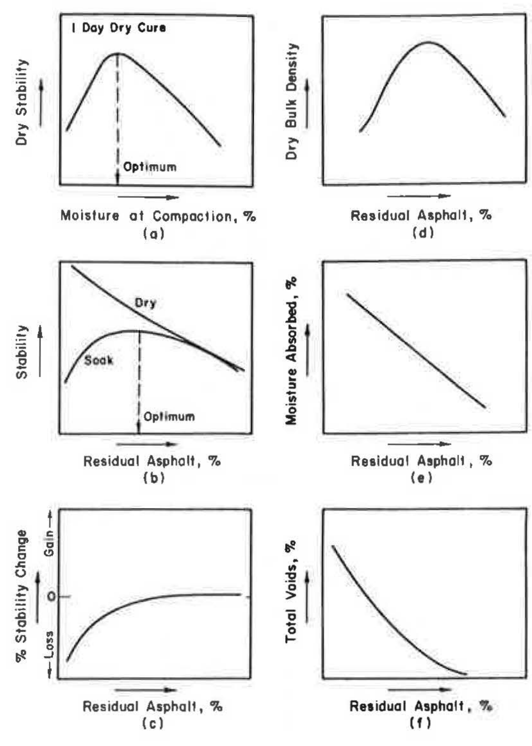


Figure 6. Emulsified asphalt-aggregate mixture soak test equipment (Note: a top cover is required to prevent moisture loss).

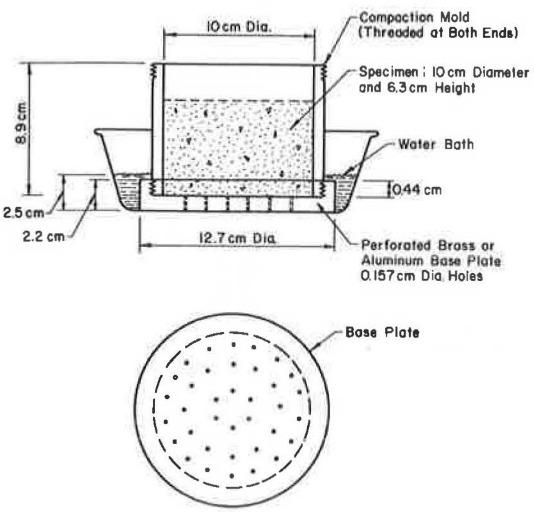


Figure 7. Correlation between base structural coefficient, a_2, and resilient modulus, M_R, at 22.2°C.

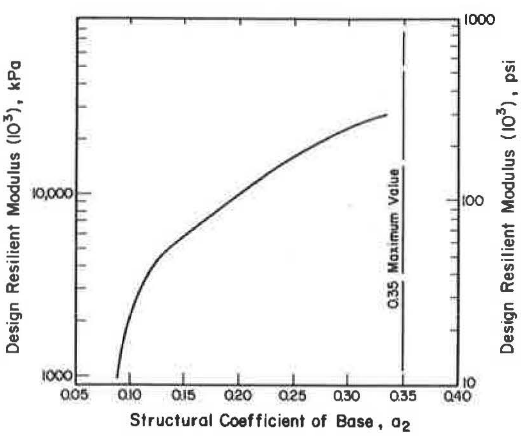


Figure 8. Correlation between base structural coefficient, a_2, and design Marshall Stability at 22.2°C.

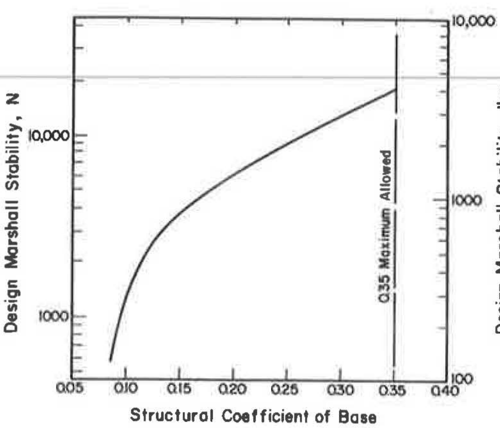
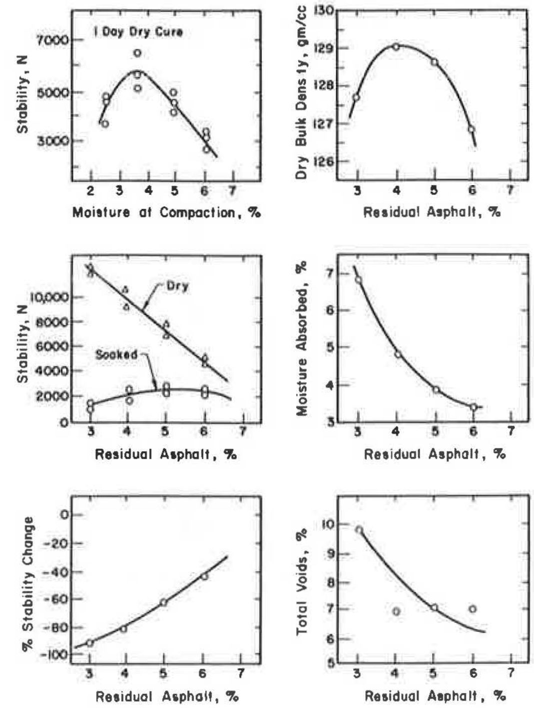


Figure 9. Mix design for pit run gravel and HFE-300 emulsion.



SOIL-CEMENT - A CONSTRUCTION MATERIAL

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The technology of cement-treated materials is summarized. Basic properties of soil - and soil-aggregate - cement mixtures are given to help the reader understand and use the product. The use of soil and cement mixtures as pavement layers is considered. Testing and mix design methods are discussed and approximate cement requirements given. A thickness design procedure is presented. Construction procedures are outlined. Recycling of material with the addition of cement to salvage and strengthen road layers is discussed. It is concluded that cement stabilization can improve the engineering properties of materials and has wide application in pavement layers. In using soil-cement, proper consideration should be given to mix design, thickness design, and construction procedures.

Soil-cement is a compacted mixture of a soil, portland cement, and water. As the cement hydrates, the mixture becomes a hard, durable paving material.

Soil-cement is used mainly as a base for road, street, and airport paving. A bituminous wearing course is normally placed on the soil-cement base to complete the pavement.

Soil-cement was developed in an effort to save road-building money by using soils on or near the construction site.

The basic idea of soil-cement is not new. In the early 1920's, state highway departments built short sections of roads with soil and cement. However, the principles of soil compaction as applied to road building were not yet developed.

Research by the Portland Cement Association led to the development of the basic control factors for soil-cement construction: (1) an adequate cement factor, (2) proper moisture content, and (3) adequate compaction.

Compared with today's practices, the methods used to build early projects were crude at best. Although methods and equipment have been greatly improved, the engineering principles resulting from this early work have been used to build many thousands of kilometres (miles) of soil-cement pavement in the United States, Canada, and other countries.

Some other terms often applied to soil-cement are "cement-treated base", "cement-stabilized soil", and "stabilized aggregate".

The material presented here explains basic information on cement-soil reactions, testing philosophy, test methods, properties of soil-cement, thickness design and construction.

Development of Standard Tests

One of the first significant findings of the early 1935 research was that the moisture-density relationship for soils as discovered in 1929 by Proctor was also valid for mixtures of cement and soil when compacted immediately after mixing and before cement hydration. It was found that optimum moisture content provided sufficient water for cement hydration. The soil-cement moisture-density test was adopted as a standard by ASTM (D558) in 1944. It was adopted as standard by AASHTO (T134) in 1945.

The next step in the 1935 research program was to devise methods of measuring the effect that various cement contents, moisture contents and densities have on the physical properties of compacted soil-cement mixtures. Since the rate and amount of cement hydration would influence final results materially, specimens were permitted to remain undisturbed for 7 days in an atmosphere of high humidity before being tested. This permitted hydration of a significant portion of the cement.

In analyzing possible test methods that might be used for evaluating soil-cement mixtures, tests used in soil and concrete testing were analyzed. Consideration was given to various compression and tension tests that might be modified to simulate the internal forces of expansion and contraction produced by changes in moisture and temperature. It was considered that these tests were not applicable since they do not simulate the nature and magnitude of the desired forces. However, it was found that repeated wetting and drying, and freezing and thawing could induce internal forces similar to those induced by changes in moisture content.

Thus, the wet-dry and the freeze-thaw tests were evolved to reproduce in the laboratory the phenomenon of volume changes. The wet-dry test was designed primarily to simulate shrinkage forces. The freeze-thaw test was designed to simulate internal forces produced by moisture and temperature change.

Because moisture plays a predominant role in the strength of soils and road bases it is essential that water play a predominant part in both the wet-dry and freeze-thaw tests. The wet-dry test is accomplished by submerging the specimens in water during the wetting portion of each cycle. In the freeze-thaw test, specimens are permitted to absorb water by capillarity during the thawing portion of each cycle.

Early in the development of the tests, a brushing procedure was developed to remove the loosened material resulting from alternate wetting, drying, freezing, and thawing. Twelve cycles for each test produced interpretable data.

As part of the research program, data on soil gradation, surface area, physical test constants, compressive strength, organic content, pH, density, and cement-void ratio were correlated with the degree of cement reaction in search of relationships that could be used to determine cement contents for construction (1). These studies produced erratic results.

Thus, the wet-dry and freeze-thaw tests were developed to determine the minimum cement content required to produce a structural material that would resist volume changes produced by changes in moisture and temperature. Because moisture and temperature changes occur in varying degrees in all climates and geographic areas, use of both the wet-dry and freeze-thaw tests assure that a hardened, structural material is produced for any area. The wet-dry and freeze-thaw tests are standards of ASTM and AASHTO (ASTM D559, AASHTO T135; ASTM D560, AASHTO T136).

Test Criteria

Studies of laboratory test data, outdoor exposure of specimens, and field performance were used in selecting the criteria for determining minimum cement content to produce a structural material.

The selected criteria included requirements of volume change, maximum moisture content, soil-cement weight loss, and trend of compressive strength. The criterion of maximum volume increase of the specimens (not more than 2%) was chosen as an indication that the cement was holding the mass intact and preventing volume increases that would otherwise take place. The criterion of maximum moisture content (not more than that required to fill the voids) was selected as a further indication of resistance to disruptive volume changes. The criterion of maximum soil-cement loss from brushing was used as an indication that the forces of expansion and shrinkage resulting from the wetting and drying, freezing and thawing, that disrupt and disintegrate soil specimens, had been resisted. The compressive strength criterion was used because increases in strength due to increases in time and cement content were evidence that the cement is functioning normally and that the soil was not interfering with hydration of the cement.

Today, after conducting thousands of tests, weight loss, together with strength gain, are the primary criteria. The validity of these criteria has been verified by a quarter century of successful field performance of soil-cement projects in service. Invaluable as the tests (2) are, they require considerable time to obtain the factors needed for construction. The Portland Cement Association has developed a special short-cut test procedure for determining cement factors for sandy soils.

Short-Cut Test Procedures for Sandy Soils

Short-cut test procedures have been evolved to determine adequate cement contents for sandy soils (3,4). These procedures do not involve new tests or additional equipment. Instead, data from previous tests of similar soils were correlated with durability to develop charts for the short-cut test procedures. The only laboratory tests required are a grain-size analysis, a moisture-density test, and compressive-strength tests. Relatively small soil samples are needed and all tests, except the 7-day compressive-strength tests, can be completed in one day (Figures 1,2,3,4). The procedures are widely applied by engineers and builders and may largely replace the standard tests as experience increases.

Figure 1. Indicated cement contents of soil-cement mixtures not containing material retained on the 4.75-mm (No. 4) sieve - Method A.

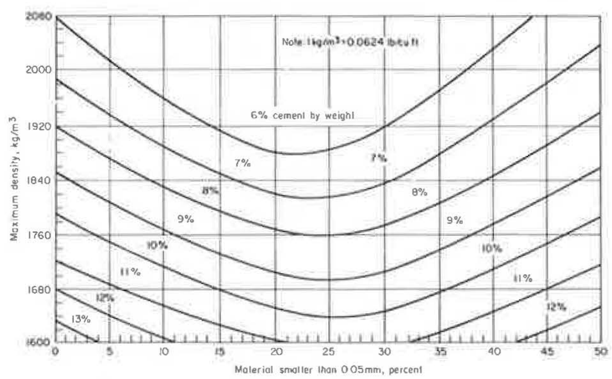
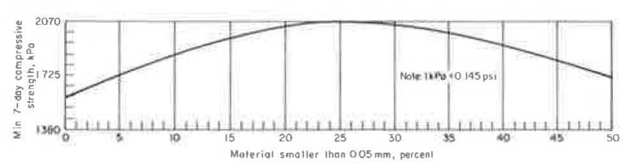


Figure 2. Minimum 7-day compressive strengths required for soil-cement mixtures not containing material retained on the 4.75-mm (No. 4) sieve - Method A.



Two procedures are used: Method A for soils not containing material retained on the 4.75-mm (No. 4) sieve and Method B for soils containing material retained on the 4.75-mm (No. 4) sieve (2).

The procedures can be used only with soils containing less than 50% material smaller than 0.05 mm (silt and clay), less than 20% material smaller than 0.005 mm (clay), and less than 45% material retained on the 4.75-mm (No. 4) sieve. Dark grey to black soils with appreciable amounts of organic impurities were not included in the correlation and therefore cannot be tested by these procedures. This is also true of miscellaneous granular materials such as cinders, caliche, chat, chert, marl, red dog, scoria, shale, and slag.

Figure 3. Indicated cement contents of soil-cement mixtures containing material retained on the 4.75-mm (No. 4) sieve - Method B.

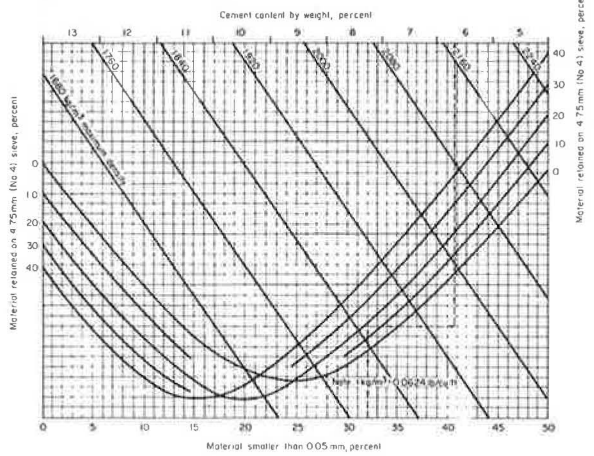
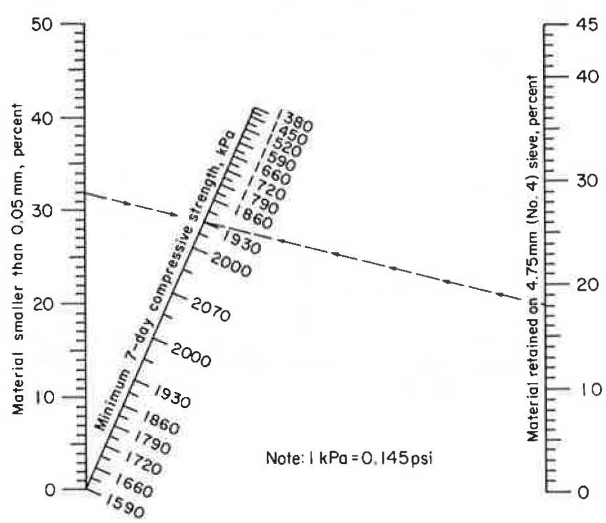


Figure 4. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on the 4.75-mm (No. 4) sieve - Method B.



Step-by-Step Procedures

Short-cut test procedures involve:

1. Running a sieve analysis of the soil.
2. Running a moisture-density test on a mixture of the soil and portland cement.
3. Determining the indicated portland cement requirement by the use of charts.
4. Verifying the cement requirement by compressive-strength tests.

In using the short-cut test procedure, the 7-day compressive strength is usually substantially higher that the minimum allowable value. This merely indicates that the soil is reacting normally. When higher strengths are obtained, it is not correct to reduce the cement factor so that a strength value is close to the minimum allowable. Such reduction invalidates the reliability of the correlation and usually results in a cement content that is not sufficient to meet the ASTM-AASHTO freeze-thaw and wet-dry test criteria. Any reduction in the cement factor can only be made based on freeze-thaw and wet-dry tests at lower cement contents. A compressive strength value below the minimum indicates abnormal reaction and additional tests are needed to establish a cement requirement.

Compressive Strength

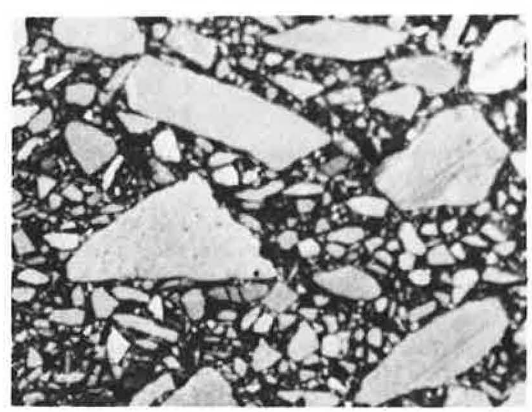
The influence of cement in producing compressive strength in soil-cement mixtures can be analyzed from two viewpoints. The cement influence is evidenced by increases in strength with age and cement content.

The 7-day compressive strengths that represent a durable soil-cement base vary with the physical and chemical properties of the soil and are generally between 2.1 and 5.5 MPa (300 and 800 psi).

Coarse-Graded Aggregates

Excessively coarse-graded aggregates, that is, more than 45% retained on the 4.75-mm (No. 4) sieve, have been used. However, the cement content to make durable soil-cement is generally increased. Up to a limit, an increase in the quantity of coarse material reduces the cement requirement, since the finer particles requiring cement to bind them together are replaced by a coarse particle. The total density of the aggregate material increases as the quantity of coarse aggregate increases, but the density of the 4.75-mm (No. 4) sieve fraction decreases. Too much coarse material interferes with compaction of the matrix of finer particles. Adequate density of the fine fraction is important for it is here that most of the cementing action takes place, forming a matrix that holds the coarser particles together (Figure 5).

Figure 5. The cemented portion of the fine fraction holds the larger particles together.



Durability criteria rather than strength criteria, should be used to establish cement requirements for durable, long-lasting soil-cement. The gradations below are compatible with the short-cut test procedure which requires only a minimum of testing. For materials that do not meet the requirements for the short-cut procedure, particularly aggregate materials that contain more than 45% retained on the 4.75-mm (No. 4) sieve, the standard wet-dry and freeze-thaw tests should be run to insure a durable mixture.

Aggregates of the following gradation limits are suggested to achieve the most economical cement factor for durable soil-cement.

Sieve size	% by weight passing
50 mm (2 in.)	100
4.75 mm (No. 4)	55 - 90
2.0 mm (No. 10)	37 - 67
75 µm (No. 200)	0 - 30
PI	10 maximum

Engineering Properties

During construction soil-cement is compacted to a high density. As the cement hydrates, the mixture hardens in this dense state to produce a structural slab-like material. The strength and elastic properties of soil-cement depend primarily on the type of soil, age, and curing conditions.

As shown in Figure 6, a direct relationship exists between flexural strength (modulus of rupture) and compressive strength--the modulus of rupture being about 20% of the compressive strength (5).

Figure 6. Relationship between modulus of rupture and compressive strength of soil-cement.

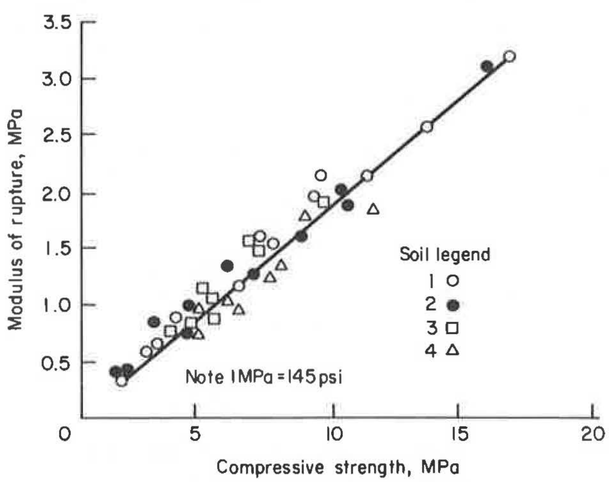


Figure 7. Strength gain with age, projects in service.

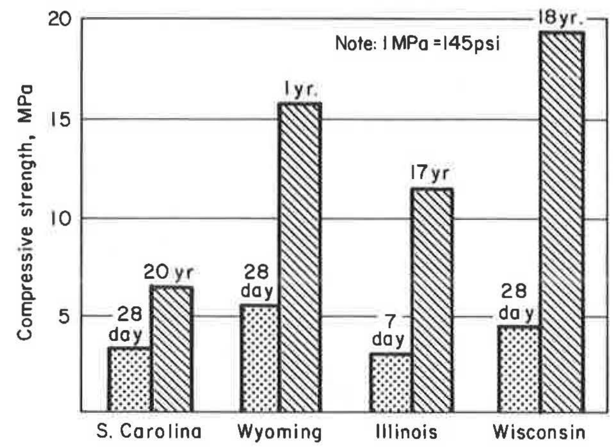
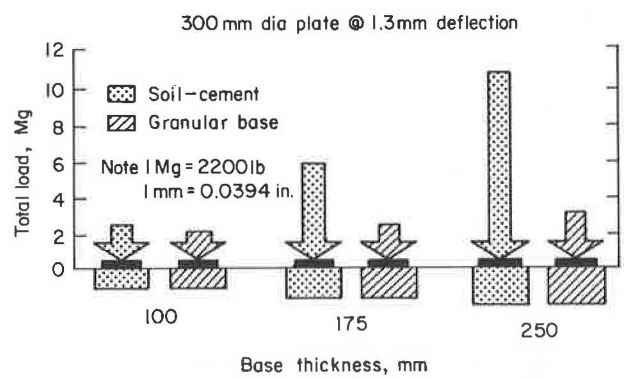


Figure 8. Load-carrying capacity of soil-cement and granular base. Plate-bearing tests on 3x3.7-m (10x12-ft) soil-cement panels made of a sandy soil with 80 kg per cubic metre (5.3% cement by volume). The k value of the clay subgrade was 18 to 33 MPa/m (66 to 122 psi/in.) under soil-cement panels and 27 to 39 MPa/m (98 to 142 psi/in.) under untreated crushed stone.



The cement in soil-cement continues to hydrate for a long time even under traffic. Cores taken from roads after many years of use show appreciably greater strength than samples tested at 7 and 28 days (Figure 7) (6). This means that soil-cement has a "reserve" of strength to accommodate increases in volume and weight of traffic.

Because of soil-cement's slab-like character it has high load-carrying capacity. Results of bearing tests (7,8) (Figure 8) show that soil-cement can support up to three times greater loads than other low-cost base materials of the same thickness.

Cement-Modified Soils

Cement-modified soil is a soil material that has been treated with a relatively small quantity of cement--less than is required to produce soil-cement. Cement treatment changes and improves the soil's physical properties. Cement-modified soils are arbitrarily classified into two groups: (1) cement-modified granular soils with less than 35% silt and clay, and (2) cement-modified silt-clay soils.

One common way of measuring the effect of cement on improving a granular material which contains an excessive amount of clay is by reduction in plasticity characteristics as measured by the plasticity index (PI). Figure 9 shows the reduction in PI produced by the addition of cement to a substandard granular base material. Figure 9 also shows the permanency of the PI reduction as measured over a 10-year period (9).

The Sand Equivalent test (10) used to detect the presence of undesirable clay-like materials tends to magnify the volume of the clay somewhat in proportion to its detrimental effects. Concrete sands and crushed stone have sand equivalent values of about 80; expansive clays have sand equivalents of 0 to 5. Improvement in Sand Equivalent value of a Utah granular soil having a PI of 11 and having 33% passing the 75-mm (No. 200) sieve is shown in Table 1. Three percent cement increased the value from 11 to 59. The PI was correspondingly reduced from 11 to 0 (11).

Figure 9. Plasticity index vs. time.

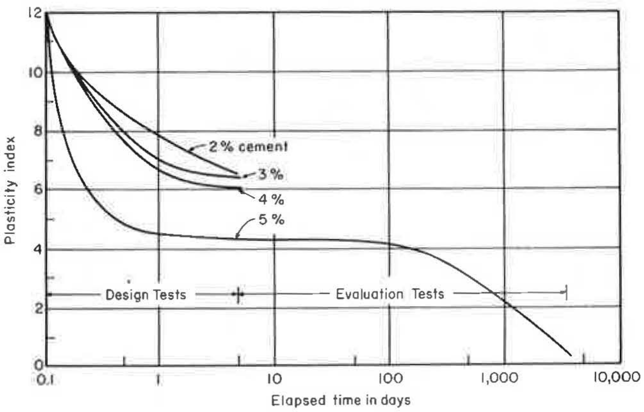


Table 1. Sand equivalent values.

Percent Cement by Weight	Sand Equivalent
0	11
1	18
2	36
3	59

Several types of laboratory strength tests are also used to measure improvement of a substandard granular material. One common test is the California Bearing Ratio test (CBR). Table 2 gives data for an A-1-b(0) disintegrated granite from California. The addition of 2% cement by weight increased the CBR from 43 to 255. The addition of 4% cement increased the value to 485. The permanency of CBR improvement is also shown in Table 2 with results of 60 cycles of laboratory freezing and thawing. Further cement hydration during the freeze-thaw test more than made up for any detrimental effect with the result that the CBR did not decrease. In fact, the CBR of the 4% mixture increased to 574 (11).

Table 2. Permanency of bearing value.

	CBR
Raw soil	43
2% cement by weight, age 7 days	255
2% cement by weight after 60 cycles of freeze-thaw	258
4% cement by weight, age 7 days	485
4% cement by weight after 60 cycles of freeze-thaw	574

Some agencies use the Stabilometer test (12) to determine the stability of a material. A stabilometer value (R value) of about 78 is considered equivalent to good crushed stone. Table 3 gives data for a fine sand. The R value increased from 65 for the untreated sand to 89 with the addition of 3% cement (11).

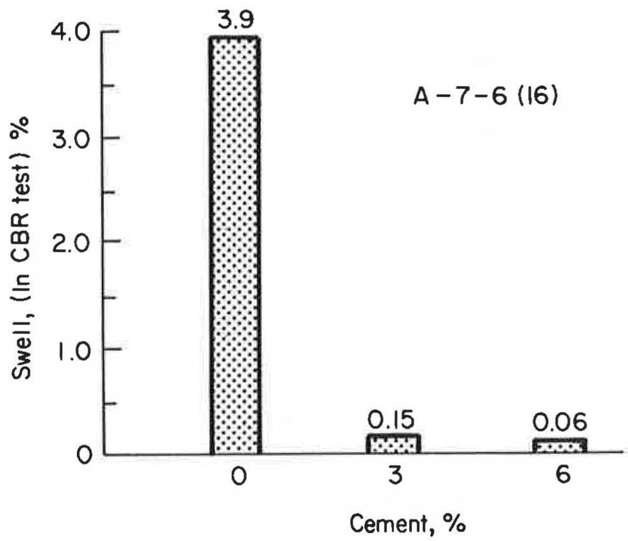
Table 3. R-values.

Raw soil	65[a]
Lab mixture	
3% cement by weight	89
5% cement by weight	93

[a] A-2-4 fine sand

Reduction in swell of an expansive clay due to the addition of cement is shown in Figure 10. The addition of 3% cement reduced the expansion from 3.9% to 0.15%, an insignificant value (8).

Figure 10. Cement treatment of expansive clay.



These examples illustrate that when cement is added to a soil material the chemical and physical properties of that material change, and when sufficient cement is added a strong structural material results.

Thickness Design

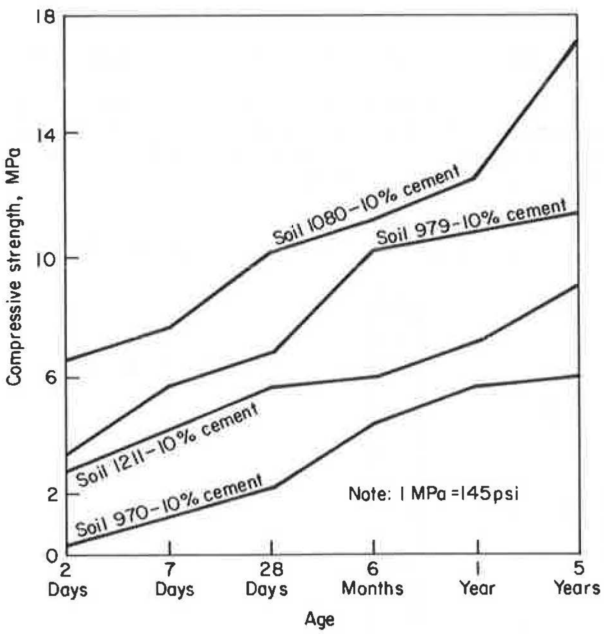
Soil-cement is a material that possesses its own unique structural characteristics.

The structural properties of soil-cement depend on soil type, curing conditions, and age. Typical ranges for a wide variety of soil-cements at their respective cement contents required for durability are:

Property	28-Day Values
Compressive strength, saturated	2.1 - 5.5 MPa (300-800 psi)
Modulus of rupture	0.5 - 1.0 MPa (70-150 psi)
Modulus of elasticity (static modulus in flexure)	4 100 - 13 800 MPa (600,000-2,000,000 psi)
Poisson's ratio	0.12 - 0.14
Critical radius of curvature on 150x150x760-mm beam	100 - 190 m (4,000-7,500 in.)

The average strength of a soil-cement pavement over the design life will be considerably greater than the 28-day values. Figure 11 shows 5-year laboratory strength gains for several soil-cements (6). These strength gains provide a margin of safety in the thickness design procedure.

Figure 11. Strength gain with age, laboratory specimens.



Studies at the Portland Cement Association laboratories (13,14) aimed toward the development of a soil-cement thickness design procedure covered these major phases of research:

1. Basic structural properties
2. Load-deflection characteristics
3. Fatigue properties

The load-deflection research on soil-cement pavements has shown that it is possible to describe the response by a single equation, regardless of soil type and cement content, as long as the final product meets the criteria for fully hardened soil-cement. Soil-cement meets specific mix design criteria (2), and the soil-cement pavements are constructed under definite specifications (15). The thickness design procedure relates to all climate areas when the quality of the soil-cement meets the above requirements.

Fatigue studies revealed that, for a given design, the number of load repetitions to failure was related to the radius of curvature of bending. This relationship proved to be similar to the known fatigue behavior of other materials.

The effect of soil type was significant in the fatigue results (Table 4). It required the division of soils into two broad textural types--granular and fine-grained soils--and the corresponding use of separate design charts (Figures 12,13) for design purposes.

Table 4. Thickness design - fatigue consumption coefficients[a].

Axle load, Mg	Granular soil-cement	Fine-grained soil-cement
Single axles		
13.6	12,500,000.	3,530.
12.7	1,270,000.	1,130.
11.8	113,000.	337.
10.9	8,650.	93.
10.0	544.	23.3
9.1	27.	5.2
8.2	1.0000	1.0000
7.3	0.0250	0.1600
6.4	0.0004	0.0200
5.4	-	0.0018
Tandem axles		
22.7	12,500,000.	3,530.
21.7	3,210.000.	1,790.
20.9	792,000.	890.
20.0	186,000.	431.
19.1	41,400.	203.
18.1	8,650.	93.
17.2	1,690.	41.1
16.3	305.	17.5
15.4	50.4	7.1
14.5	7.5	2.74
13.6	1.0000	1.0000
12.7	0.1200	0.3410
11.8	0.0120	0.1070
10.9	0.0010	0.0310
10.0	-	0.0081
9.1	-	0.0018

[a] These coefficients express the relative fatigue consumption of different axle-load magnitudes for granular and fine-grained soil-cements, respectively.

Note: 1 Mg = 2.2 kips

Design Procedure

In the design procedure, factors analyzed to determine the design thickness are:

1. Subgrade strength
2. Pavement design period
3. Traffic, including volume and distribution of axle weights (single- and tandem-axle loading configurations of conventional trucks)
4. Soil-cement base course thickness
5. Bituminous surface thickness

Subgrade Support. The support given to the soil-cement pavement by the subgrade is a major element in the thickness design procedure for soil-cement pavements. Subgrade support is measured in terms of the Westergaard modulus of subgrade reaction, k, and is determined by plate-loading tests on the subgrade or correlated to simple soil tests.

Design Period. A design period is selected for use with this procedure. Design period is not to be confused with the service life. The selection of the design period is somewhat arbitrary. The design formulation is not particularly sensitive to variations in the design period.

Figure 12. Thickness design chart for granular soil-cements.

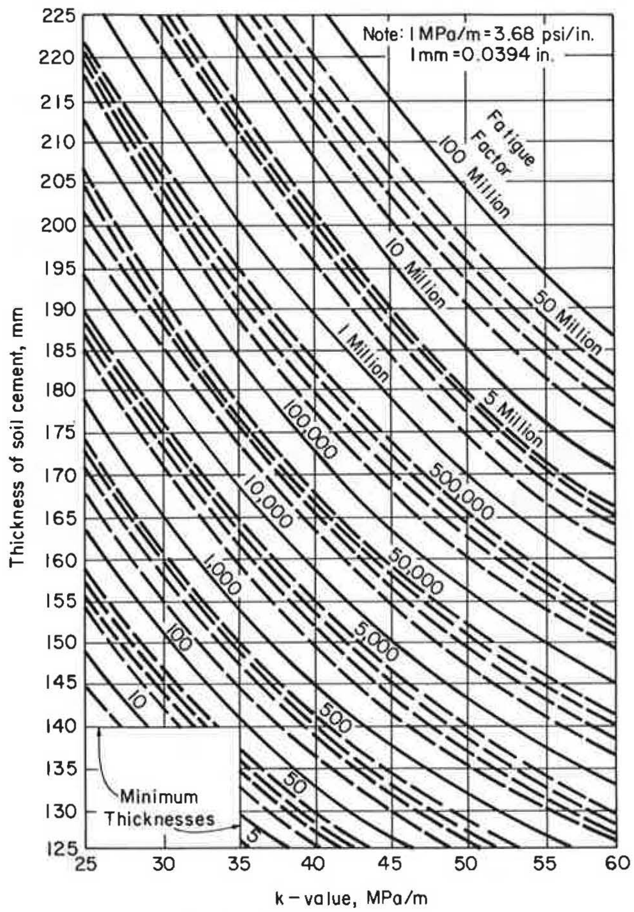
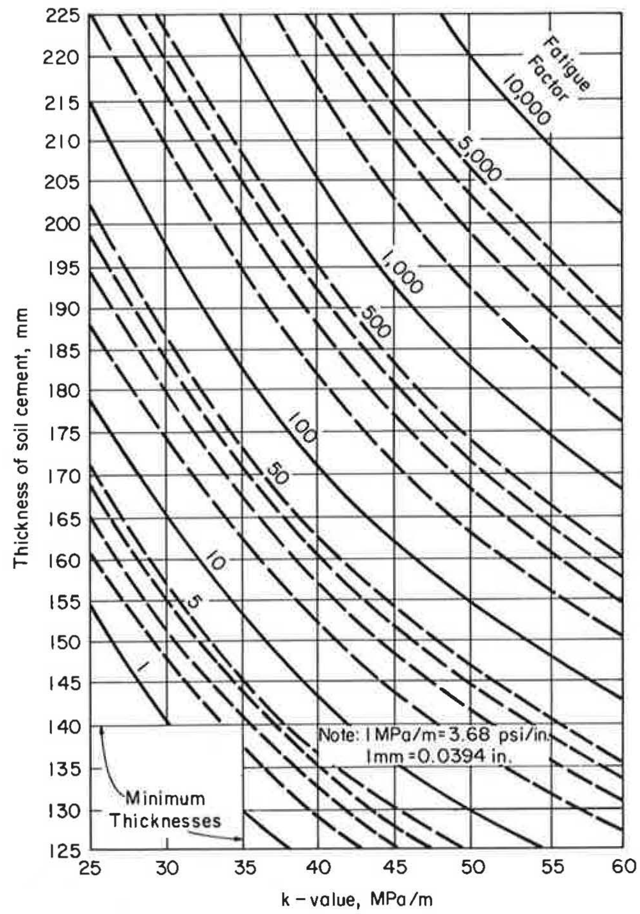


Figure 13. Thickness design chart for fine-grained soil-cements.



Traffic. The weights and volumes of axle loads expected during the design period are major factors in determination of the design thickness. The traffic analysis used in this procedure involves:

1. Determining average daily traffic in both directions (ADT) including the percentage of trucks
2. Projecting the traffic to a future design period
3. Determining the axle-load distribution
4. Computing the Fatigue Factor

Fatigue Factor. A single value that expresses the total fatigue consumption effects of the volumes and weights of single- and tandem-axle loadings for a given design problem is called the Fatigue Factor in this design procedure. It is based on coefficients showing the relative fatigue consumption of different axle-load magnitudes, the Fatigue Consumption Coefficients, which are listed in Table 4. The designer should note that different values are used for granular and fine-grained soil-cements as specified by the two equations developed from the research for the two general soil types.

The Fatigue Consumption Coefficients are multiplied by the numbers (in thousands) of axles in each weight group and then summed to give a single-value Fatigue Factor.

Materials

A proper cement content is the first requisite for soil-cement construction.

Table 5 gives the normal range of cement requirements for soils of the various AASHTO soil groups. Table 6 gives average cement requirements for a number of miscellaneous materials and special types of soil. These average cement requirements may be used for rough cost estimating and then confirmed or revised by laboratory tests (2).

Recycling Old Roadway Materials (16)

The materials usually found in old gravel or stone roads and streets make excellent soil-cement. They are generally friable, easily mixed, and require only a minimum amount of cement. Frequently the old bituminous mat, if present, can be salvaged by pulverizing it and mixing it with the old base course material for processing with cement (Figure 14). The reuse and recycling of these materials with cement is an economical way to strengthen and rebuild worn out granular base pavements. This may be specially beneficial when the level of the top of the pavement cannot be raised from a drainage standpoint.

Table 5. Normal range of cement requirements for B- and C-horizon soils[a].

AASHTO Soil Group	Cement kilograms per cubic metre of compacted soil-cement	Cement percentage by weight of soil
A-1-a	80-110	3-5
A-1-b	110-130	5-8
A-2-4		
A-2-5		
A-2-6	110-140	5-9
A-2-7		
A-3	130-180	7-11
A-4	130-180	7-12
A-5	130-180	8-13
A-6	140-210	9-15
A-7	140-210	10-16

[a] A-horizon soils (topsoils) may contain organic or other material detrimental to cement reaction and thus require higher cement factors. For dark grey to grey A-horizon soils, increase the cement contents 4 percentage points (60 kg/m^3) of compacted soil-cement); for black A-horizon soils, 6 percentage points (100 kg/m^3) of compacted soil-cement.

Table 6. Average cement requirements of miscellaneous materials.

Material	Cement, kilograms per cubic metre of compacted soil-cement	Cement percentage by weight of soil
Caliche	130	7
Chat	130	7
Chert	130	8
Cinders	130	8
Limestone screenings	110	5
Marl	160	11
Red dog	130	8
Scoria containing plus No. 4 material	180	11
Scoria (minus No. 4 material only)	130	7
Shale or disintegrated shale	160	10
Shell soils	130	7
Slag (air-cooled)	130	7
Slag (water-cooled)	140	12

Figure 14. Old bituminous mats can be broken up, pulverized, and incorporated with the old granular base material to make good soil-cement. The same machine can be used to mix the prepared material with portland cement and water.

Deposits of friable or granular materials that require less cement and little pulverizing can often be found nearby and can be used to blanket the existing soil or be combined with it. Selective grading often is used to place the most favorable soils in the top of the grade. Comparative cost estimates will indicate the most economical materials or combination of materials to use.

Construction

Soil, cement, and water can be mixed-in-place using traveling mixing machines, or mixed in a central mixing plant (17). The types of mixing equipment are:

- I. Traveling mixing machines
  - A. Flat transverse-shaft type
    - 1. Single-shaft mixer
    - 2. Multiple-shaft mixer
  - B. Windrow-type pugmill
- II. Central mixing plant
  - A. Continuous-flow-type pugmill
  - B. Batch-type pugmill
  - C. Rotary-drum mixers

Whatever type of mixing equipment is used, the general principles and objectives are the same. Modern mixing machines are very efficient and give high daily production at low cost.

Use of Borrow Materials

From a construction or cost standpoint, it is sometimes advantageous to use a borrow material instead of the soil in place. The existing soil or the soils encountered in cut sections may have a very high clay content and require a relatively high cement factor. Also, considerable effort may be required to pulverize the soils properly.

General Construction Steps

In soil-cement construction the objective is to mix a pulverized soil and cement (Figure 17) thoroughly in correct proportions with sufficient moisture to permit maximum compaction. Construction methods are simple and follow a definite procedure:

A. Initial preparation
   1. Shape the area to crown and grade
   2. If necessary, scarify, pulverize, and prewet the soil
   3. Reshape to crown and grade
B. Processing
   1. Spread portland cement and mix
   2. Apply water and mix
   3. Compact
   4. Finish
   5. Cure

During grading operations, all soft subgrade areas, springs, and frost-heave areas should be located and corrected, and stumps and other debris removed. The roadway should be shaped to approximate crown and grade.

Figure 15. Spreading portland cement in regulated quantities.

Most soil-cement is built from materials that require little or no preliminary pulverizing. If pulverization is required, it is usually done the day before actual processing. Processing operations are continuous. The moist soil-cement mix is compacted and finished immediately.

Compacted and finished soil-cement contains sufficient moisture for adequate cement hydration. A moisture-retaining cover is placed over the soil-cement soon after completion to retain moisture and permit the cement to hydrate. While most soil-cement is cured with bituminous material, but other materials and fog-type water spray are satisfactory.

Summary

When cement is added to a soil material the physical properties of the material are improved. Soil-cement is a hardened structural material possessing definite engineering properties. Outlined here are the basics of soil-cement technology: reaction, testing, properties, design, and construction. By understanding some of the basics of soil-cement an engineer can use it to advantage.

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MIX DESIGN CRITERIA FOR CEMENT MODIFIED EMULSION TREATED MATERIAL

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This paper is the second part of a comprehensive investigation of the stabilization of sands and sand-clay aggregates with asphalt emulsion. The objective here is to develop mix design criteria for emulsion treated soil aggregates. Previous investigations by author and others suggest that cement in trace quantities is indispensible in order to enhance the durability of sand-emulsion mixtures; accordingly Cement-modified Emulsion Treated Material (CETM) only is studied herein. With due consideration to the prevailing distress mechanisms in cold mix bases, several tests are proposed to evaluate CETM. Marshall stability and shear strength tend to exhibit an optimum, respectively, with emulsion content and fines content. It appears feasible to predict the Marshall stability of CETM from a simple soil property such as particle size distribution. Using the test results on five naturally occurring soils and one synthetic aggregate mix design criteria for sands and sandy soils is proposed. Minimum Marshall stability of 4.23 kN (950 lbs) insures that CETM will not undergo shear failure under heavy truck tire pressure. Another criterion to detect and avoid moisture susceptible mixtures is that Marshall cylinders during vacuum soaking should not absorb more than 8.5% moisture. A third criterion to safeguard against stiff mixtures is that the seven day "dry bearing strength" shall not exceed 2760 kPa (400 psi). The recommended design values and test method are presented and discussed in the paper.

Bituminous emulsions are used widely in the construction and maintenance of low-volume rural roads and city streets. Two classes of asphalt emulsion are commonly used. Cationic emulsions (positively charged particles) adhere better to such electronegative aggregates as silica and quartz; anionic emulsions (negatively charged particles) have better adhesion on carbonate aggregates. Because such a wide variety of aggregates is used in pavements, ionic characterization may, however, be of secondary importance.

Because emulsified asphalt in base stabilization has been used on a limited scale only insufficient data are available concerning the response of emulsion to various aggregates; for this reason, select aggregates have, for the most part, been used in roads during the last two decades. For instance, of the thirty projects which Finn et al. (5) surveyed in seven states, only seven of the bases included sandy or fine-grain soils. Kerston and Pederson (11) and Korfhage (12) reported poor performance with SS-1 in Minnesota loess and a poor quality aggregate. Scrimsher et al. (18) reported that two cold aspalt emulsion mixtures - one a dense graded and the other an open graded - placed as a 25 mm (1 in.) overlay on an existing pavement showed noticeable raveling and the surface caused rough riding. Meier (14) recently reported three projects in the Northwest in which fine sand was stabilized with slow setting grade emulsified asphalt. Again, the performance of two of the three projects was less than satisfactory. One problem involved the difficulty of aerating the mixture, a circumstance which was attributed to the finer gradation. Nevertheless, successful use of emulsified asphalt in sand and cohesive graded sand has been reported by Fruedenberg (6). As Bratt (2) remarked, however, numerous problems exist; for example, finding a specification that will guarantee consistent behaviour of emulsions. The numerous failures reported in the literature suggest the lack of a system for evaluating the amenability of a soil to stabilization with asphalt emulsion. Various factors affecting emulsion stabilization of sands and silty sands have been reported in a previous paper (7). That study, as well as others (17,19), shows that portland cement in trace quantities (1-1 1/2%), acting as a stabilizing agent, greatly enhances the soak-stability of sand-emulsion mixtures. In this study, therefore, we are concerned only with Cement-modified Emulsion Treated Material (CETM).

Because of the increased interest in emulsion, due in no small part to the influence of the Federal Energy Administration and the E.P.A. plus the recommendation of the Federal Highway Administration, investigators at the University of Mississippi have embarked on a research program to determine whether local sands and silty sands can be economically used for base stabilization. This report, therefore, focuses on developing mix design criteria for emulsified asphalt bases. This objective will be accomplished in three steps: (1) Choose feasible test methods and procedures for evaluating the desired properties of cold mixes. (2) Use these methods to evaluate the strength, deformation, and moisture absorption properties of CETM. (3) Use these results to propose appropriate mix design criteria.

Materials

Soils

Six sandy soils were selected for study; their physical properties are given in Table 1. For convenience a one letter two digit system is used to identity each soil; for example K38 designates soil #38 with Kaolin as the predominant clay mineral. The percentage fines (percentage fines refers to the amount of material passing through a #200 sieve) of these soils varies widely - namely two of 10%, 12%, 14%, 16% and 17% - as does the uniformity coefficient. All, except K46, are naturally occurring soil aggregates from various locations in Mississippi. Soil K46, however, is a 3:2 blend of a coarse sand and silty clay.

Asphalt Emulsion

Because siliceous aggregates (for that matter, most other highway aggregates) are electronegative cationic emulsion (CSS1) is preferred and is being used in this investigation. The properties of the asphalt emulsion, as furnished by the manufacturer, are listed in Table 2.

Mix Preparation

Air-dried aggregate was first blended with cement and subsequently moistened with water before mixing with the emulsion. The ingredients (aggregate and emulsion) were hand-mixed for one minute, followed by machine mixing until the aggregate was evenly coated. To facilitate even coating, excess moisture (2% to 3%) was added during mixing and subsequently evaporated by a blower.

Organization of the Report

In accordance with the stated objectives, the results of this study are presented in four distinct phases. A brief description of the proposed tests constitutes the first part of this report. Relevant material properties such as Marshall stability, triaxial shear strength, and permanent deformation are presented in the second part of the report. In the third phase of the study, the results are analyzed to propose mix design criteria for CETM. The last section presents a systematic step-by-step procedure (or a methods manual) for design of CETM mixtures in the laboratory.

Selection of Test Methods

The methodology used in making mix-design recommendations was to first determine the significant failure modes in cold mix bases. Then, considering these failure modes, basic required material properties of cold mix bases were identified. Available tests were then evaluated in respect to their effectiveness in measuring these required properties.

A recent study (10) reported that distortion caused by instability is the distress most prevalent in the existing cold mix bases; followed by disintegration and cracking. The survey study currently reported by the writer tends to substantiate this observation (8). Two types of permanent deformations are identitied: the first, consolidation deformation; and the second, plastic deformation, which is due to appreciable vertical and lateral shear failure movement of large masses under wheel loads. Shear strength, therefore, is considered a basic property in a cold mix.

Stability Test. The stability of CETM is the relevant property utilized in proposing mix design criteria. Marshall stability results have generally been considered satisfactory for assessing the overall strength and stability under repeated application of wheel loads. Other factors in favor of the Marshall test are (a) ability of the test method to simulate in-service conditions, (b) reproducibility of test results and (c) simplicity of execution.

Marshall test specimens 102 mm (4 in.) in diameter by 64 mm (2.5 in.) high were prepared according to ASTM D 1559, except for the modification that 75 blows, instead of 50, were applied on both sides. These specimens, wrapped except for the top face, were air dried for seven days at 50% relative humidity (RH) and 25° C (72°F) before testing at a loading rate of 51 mm (2 in.) per minute. This curing procedure is referred as "partial air-cure" in this report.

Shear Strength Test. In considering the principal failure mechanisms observed in ETM bases, one realizes that shear strength is an important property of the mixture. Undrained shear strength parameters (by triaxial test) are obtained from vacuum soaked specimens, 70 mm (2.8 in.) in diameter and 152 mm (6 in.) high. In order to minimize the effect of viscous resistance on shear strength parameters (thereby rendering a very conservative estimate of shear strength parameters), the rate of strain is set at 0.13 mm (0.005 in.)/min.

Repeated Triaxial Tests. Permanent or plastic deformation of pavement materials is especially significant in estimating the rutting of pavements. Each specimen tested, 70 mm (2.8 in.) in diameter by 152 mm (6 in.) high, was subjected to thirty load pulses per minute. The pulse used had a triangular shape and a duration of 0.20 seconds. Specimens were tested to an average of 10,000 load repetitions using constant confining pressures of 35 and 70 kPa (5 and 10 psi). Deviator stresses varying from approximately one to six times the confining pressure were used in the repeated load tests. Permanent axial and elastic deformations occurring at the mid-height of the specimens were measured by means of a pair of linear variable differential transducers (LVDT). It should be noted that elastic or rebound strain is used in modulus of resilience calculation, whereas the plastic or permanent strain is important in estimating the distortional characteristics.

Moisture Susceptibility Test. Of the various moisture susceptibility tests reviewed, the vacuum saturation method appeared to be most appropriate. The advantage of this method over others is that the distribution of moisture in the soaked specimen is nearly uniform within the whole mass of the specimen. When the moisture distribution within the specimen was not uniform, as occurred in the MVS test, strength results were not reproducible. Altrue several variations of vacuum soaking are

TABLE 1. Soil identification and compositional data

Soil No.	Location	Passing #200 Sieve, %	Liquid Limit, %	PI	Unified System Classification	Fines Ratio*	CKE Oil Ratio
K-38	Highway 6, Oxford	10	14	NP	SW-SM	0.133	5.5
K-40	Highway 6, Oxford	17	15	NP	SM	0.200	6.0
K-44	Calhoun Co., MS	14.5	18	NP	SM	0.184	6.5
K-45	Oxford, MS	10.0	--	NP	SP-SM	0.120	---
K-46	Oxford, MS	16.0	16	NP	SM	0.276	---
K-48	Rankin Co., MS	12.0	--	NP	SP	0.122	6.5

*Fines Ratio = material passing #200 sieve/material passing #40 sieve

TABLE 2. Properties of asphalt emulsion

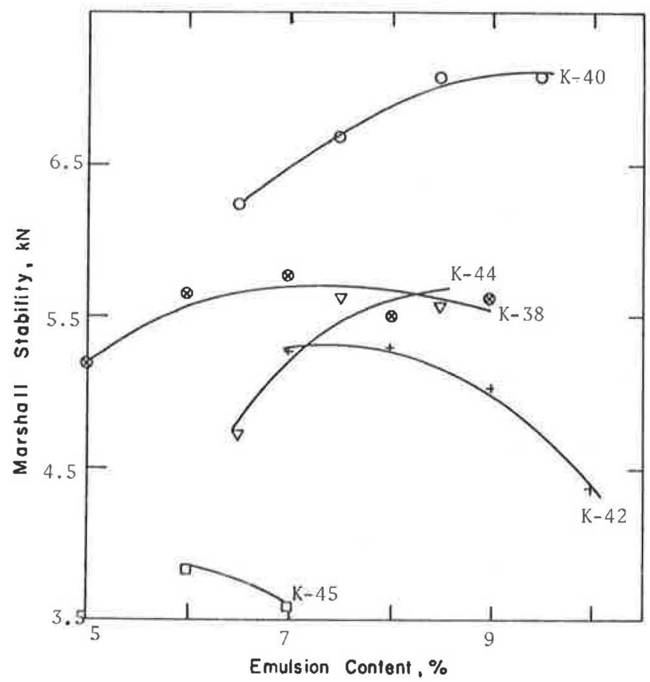
Property	Cationic CSS-1
Emulsion:	
Furol viscosity @ 28°C	35-65
Settlement, 5 days, %	---
Cement mixing, % broken	0.1
Residue (by distillation), %	64.0-68.0
Base Asphalt:	
Penetration at 28°C, 100 g, 5 sec.	+140
Solubility in CS^2, %	---
Ductility @ 28°C 5 cm/min, cms	100+
Ionic charge	positive

presently in use, the water susceptibility test method suggested by the Asphalt Institute was adopted in this study. Marshall test specimens were subjected to one-hour vacuum saturation at 100 mm (4 in.) mercury followed by one hour of soaking at normal atmospheric pressure. A complete description of the vacuum soaking procedure can be seen in reference 16.

Strength Properties of CETM

The investigations, as described in references 8, 17 and 19, show that for ETM mixtures with trace cement (1% to 1-1/2%) would be a satisfactory material for base construction. In order to propose mix design criteria, however, the strength and deformation properties of CETM were determined, and the results are presented herein.

Figure 1-Variation of Marshall Strength (7-day air cured and vacuum soaked) with emulsion content. Temperature 25±1°C.



Note: 1 kN = 225 lbs.

Marshall Stability

The effect of different variables, such as emulsion content and fines content on the stability of CETM mixtures at 25±1°C (72±2°F) was investigated by the modified Marshall test. The seven-day air-dry (air-dried from top face only) stability values generally decrease with an increase in emulsion content from 6% to 10%. Dunn and Salem (4) reported optimum bitumen content of 5% for Leighton Buzzard sand after seven days curing. The soaked stability results (Fig. 1) are more consistent in that they increase with emulsion content; attain an optimum value somewhere between 6% to 9%, depending on the fines content; and then either remains constant or slightly decreases. In other words, in many aggregates it is possible to find an optimum emulsion content giving the most stability.

The effect of fines content is such that both the dry and the soaked stabilities increase with fines, peak at about 15% to 18%, and then gradually decreases (7,8). As the fines content increases, the density also increases; this increase in density is primarily responsible for the increase in strength. On the other hand, a large fines content in excess of the optimum has an adverse effect on the mixture. The fines absorb a large quantity of water which causes swelling in the mixture and negates some or all of the stability gained from increased density.

This brief discussion reveals that numerous factors pertaining to soil, emulsion, and mixture-properties govern the stability of the end product. Therefore, it would be significant if Marshall stability (soaked) could be correlated to the various properties. Those properties thought to have some bearing on Marshall stability are:

1. Percentage fines (PF)
2. Fines ratio (FR = $\frac{\text{percent fines passing sieve \#200}}{\text{percent fines passing sieve \#40}}$
3. Particle index (PIn, Particle index is a measure of geometric characterisitcs which include shape, angularity and surface texture.)
4. Emulsion content (EC), percent
5. Penetration of emulsified asphalt (Pen) (The penetration of the bitumen in emulsion samples varied around 140.)
6. Dry density of compacted mix (γ_d), pcf
7. Cement content (Cm), percent

TABLE 3: Experimental Marshall stability values of CETM after soaking compared with those prodicted by Eq. 1.

Soil No.	Cement, %	Emulsion, Water, %	Dry Density, kg/m^3	Marshall Stability, kN	
				Experimental	Predicted by Eq. 1
K-38	1.5	6 + 4 + 3[a]	1874	6.00	5.49
	0.5	7 + 3 + 3	1895	1.78	1.82
	1.0	7 + 3 + 3	1911	4.76	3.29
	1.5	7 + 3 + 3	1910	6.31	5.93
	1.5	8 + 2 + 3	1895	5.98	5.84
K-40	1.5	6.5 + 5 + 3	1953	9.78	11.15
	0.5	7.5 + 4 + 3	2002	3.44	2.82
	1.0	7.5 + 4 + 3	2019	4.89	5.69
	1.5	7.5 + 4 + 3	2019	10.44	11.51
	1.5	8.5 + 3 + 3	1950	11.02	10.84
K-44	0.5	6 + 6.5 + 3	1921	4.00	3.84
	1.0	6 + 6.5 + 3	--	6.67	5.53
	1.5	6 + 6.5 + 3	--	8.00	7.98
	1.5	7.5 + 5.8 + 3	1948	9.38	8.69
K-45	1.5	5 + 7 + 3	1828	3.53	4.62
	1.5	6 + 6.5 + 3	1844	5.29	5.47
	0.5	7 + 5.3 + 3	1850	1.78	1.82
	1.0	7 + 5.3 + 3	1841	3.47	3.29
	1.5	7 + 5.3 + 3	1847	4.98	5.93
K-46	1.5	5 + 5.5 + 3	--	9.69	8.13
	0.5	6 + 5 + 3	2046	3.42	4.64
	1.0	6 + 5 + 3	2065	8.44	6.69
	1.5	6 + 5 + 3	2031	9.87	9.64
	1.5	7 + 4.5 + 3	--	10.49	10.44
K-48	0.5	7.5 + 5.6 + 2	1871	1.33	1.75
	1.0	7.5 + 5.6 + 2	--	4.00	3.55
	1.5	7.5 + 5.6 + 2	--	6.00	7.15
	1.5	8.0 + 5.4 + 2	--	7.29	7.07

Note: 1 kg/m^3 = 0.0624 lb/ft^3, 1 kN = 225 lbs

[a]Legend 6 + 4 + 3, respectively, Emulsion, %+Compaction Moisture, %+Excess Moisture during mixing, %

Data from six soils were used to develop a functional relationship between these variables and the Marshall stability of seven-day cured, vacuum-soaked CETM mixtures. A sub-program named SSP-stepwise regression was used for this purpose. After a series of trials, properties two, three, five and six from the list were deleted because they showed little influence on stability. The functional relationship thus derived is given below:

$$\ell n(MS) = 6.7420 + 0.0943\ (FC) - 1.9866\ (Cm) - 0.0462\ (EC)^2 + 0.4529\ (EC \times Cm) \quad (1)$$

in which Marshall stability (MS) is in pounds. (1 lb = 4.448N). Using Eq. 1, the Marshall stability values of six soils were predicted, and they were reasonably close to the experimental values, as can be seen in column 6 of Table 3. An important advantage of Eq. 1 is that in order to determine the stability values of a given soil, one needs to know only the particle size distribution of the soil.

Triaxial Shear Strength

Effect of emulsion content on shear strength parameters of mixture. The results show that the ϕ_u (friction angle) values of all soils tend to decrease with emulsion content. Cohesion values, however, increase slightly at low emulsion contents attain an optimum, followed by a gradual drop with further increase in emulsion.

Effect of fines content of soils on shear strength parameters of mixture. The cohesion of CETM increases with an increase in the fines content of the soil (Fig. 2a). This phenomenon may be attributed to the fact that on addition of emulsion to a soil, silt and clay size particles preferentially absorb emulsion, which in turn spreads and tends to coat the bigger sized particles. The surface area of contact is thus increased owing to the presence of such asphalt covered fine particles in the bituminous matrix thereby augmenting the cohesive characteristics of the mix. This hypothesis has been corroborated by microscopic examination of CETM mixtures, where the asphalt coated fine particles were seen sticking on to the surface of the larger ones (see Fig. 1 of reference 7). The increase in c_u value can also be attributed to the increase in density brought about by the fines.

The angle of internal friction, however, decreases with increasing amount of fines (Fig. 2b). As pointed out earlier, the fines, after absorbing the emulsion, spread and stick to the larger grains. Fines sticking to the bigger grains tend to diminish the grain-to-grain contact among the larger grains and thereby the angle of internal friction.

Using the c_u and ϕ_u values, it is possible to calculate the bearing strength of CETM mixture in accordance with the following equation, which is due to McLeod (13):

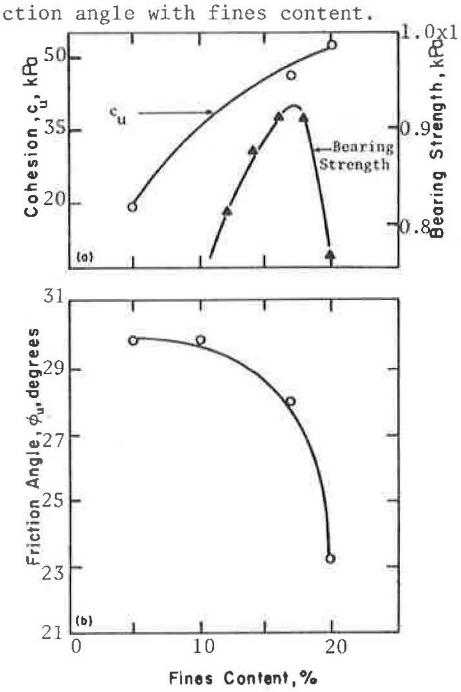
$$V = 2c_u \left(\frac{1+\sin\phi_u}{1-\sin\phi_u}\right)^{\frac{1}{2}} \left(\frac{2}{1-\sin\phi_u - 0.2\cos\phi_u}\right) \quad (2)$$

The variation of bearing strength with fines content is plotted in Fig. 2a along with the c_u and ϕ_u values. The observation that a mixture with 17% fines exhibits optimum bearing strength is in excellent agreement with the Marshall stability values where the optimum is also approximately 17% (7). Considering the fact that uniformity of mixing is greatly hampered by a large amount of fines, the researcher recommends that the optimum fines content be one or two percentage points below 17%.

Permanent Deformation in CETM

The deformation data from repeated triaxial test show that both resilient strain (recoverable or rebound strain) and permanent strain increase with the deviator stress. The fact that the curvilinear relationship is concave upward suggests that the permanent strain increases at a faster rate beyond a resilient strain of approximately 0.02%.

Figure 2. Variation of cohesion, bearing strength and friction angle with fines content.



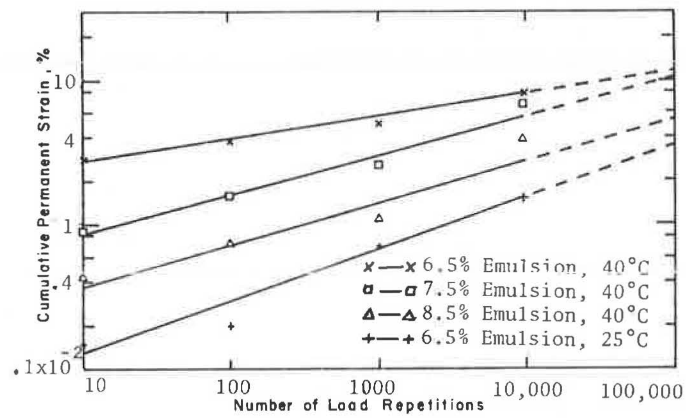
Note: 1 kPa = 0.145 lbf/in.^{2}

The relationship between cumulative plastic strain and the number of stress applications for varying emulsion contents is shown in Fig. 3. The permanent strain is seen to accumulate logarithmically with the number of load applications. Assuming a linear relationship between permanent strain and load repetitions, the anticipated strain at 40°C (105°F) after 100,000 cycles is extrapolated to be somewhat below 0.15%. In view of the tentative criterion that a rut depth of 6.3 mm (0.25 in.) is tolerable (20), we conclude that evidence is lacking to indicate that CETM will undergo such permanent deformation as to result in objectionable rut depth.

Development of Design Criteria

The overall design problem from the standpoint of stability consists of preventing detrimental shear within any one of the three elements of the composite structure - the subgrade, the base course and the wearing surface. In this discussion, the author assumes that an adequate thickness of base and surface has been provided to prevent subgrade failure. The fundamental problem, therefore, is to design CETM mixtures having sufficient stability to support the wheel loads to which they will be subjected. The discussion that follows attempts to provide a rational answer to this problem on the basis of test results provided by the triaxial test, Marshall stability test and vacuum soak test.

Figure 3. Influence of number of load repetitions on permanent strain. Deviator stress 275 kPa (40 psi) and confining pressure 69 kPa (10 psi). Soil K40.



Mix design criteria will be developed on the basis of strength and durability considerations as expressed by (1) stability of the mixture and (2) resistance to moisture intrusion, respectively.

Minimum Required Stability

Because shear failure is most crucial in CETM mixtures, the first step is to estimate the maximum shear stress or the equivalent vertical pressure that the pavement base is called upon to withstand. Considering that truck tire pressure varies from 551 to 689 kPa (80 to 100 psi), the pressure at the base level, in a typical pavement with 51 to 76 mm (2 to 3 in.) surfacing, shall be taken 517 kPa (75 psi). Assuming a load factor of 1.20 we may arrive at a design pressure of 620 kPa (90 psi).

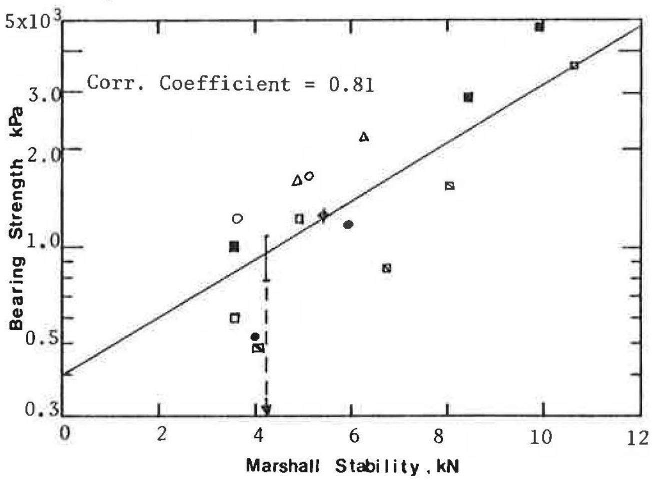
Field Methods are different from the laboratory methodology of mixing, compacting and curing. The writer's field experience with CETM suggests that 80% of laboratory strength is achievable in the field. Accordingly, the required equivalent soaked laboratory bearing strength will be 620/0.8 = 780 kPa (113 psi).

The analysis at this point shall focus on selecting a minimum bearing strength value using either the triaxial or Marshall test. If the triaxial test is chosen, Eq. 2 may be used to arrive at a combination of c_u and ϕ_u values required of a mixture to withstand a pressure of 780 kPa. A criticism often levelled against the triaxial test is that it is difficult to perform on a routine basis. Encouraged by the simplicity of the Marshall test procedure we may inquire how this test result could be used for mixture design. Using the Marshall stability and corresponding flow values, one can calculate the ultimate bearing strength in accordance with the following equation (6).

$$\text{Bearing Strength (psi)} = \frac{\text{stability}}{\text{flow}} \times \frac{120\text{-flow}}{100} \quad (3)$$

where stability is expressed in pounds and flow in units of 1/100 in. The accuracy of bearing strength calculation depends primarily upon the accuracy with which strength, and especially flow, is determined. The flow value determination during Marshall test has been subjected to criticism in that consistent reproducible flow values are difficult to obtain (7,10). Accordingly, the writer asserts that Marshall stability alone be used as a criterion. The question now is what value of Marshall stability in general corresponds to a bearing strength of 780 kPa. By plotting Marshall stabilities of several soils against the bearing strengths, as calculated by Eq. 3., a relation between these two quantities is established (Fig. 4). A safe stability value— defined as Marshall stability to withstand a vertical pressure of 780 kPa is obtained from the correlation in Fig. 4. For a Marshall stability of 4.23 kN (950 lbs) the 95% confidence interval for the mean estimated value of bearing strength is 780 kPa – 1082 kPa (113 psi– 157 psi). Stated differently a CETM mixture which exhibits a laboratory soak stability of 4.23 kN (950 lbs) would insure a bearing strength in the field of 620 kPa (90 psi) at 5% level of significance.

Figure 4. Bearing strength related to Marshall stability.



Note: 1 kPa = 0.145 lbf/in.², 1 kN = 225 lbs

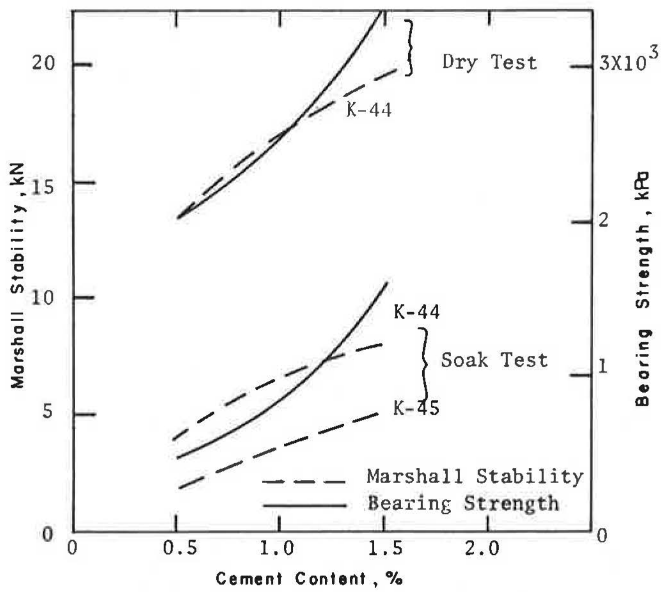
The question now arises as to whether a safety factor of 1.20 is acceptable for design purposes. Considering the test procedure and other design parameters, one can show that the present criterion will tend to give an actual safety factor greater than 1.20. For example, the pavement base will never be subjected to as severe water intrusion as is simulated in the vacuum soak test. The fact that the CETM mixture gains strength for a period of 120 days or more would tend to make the criterion based on seven-day strength conservative. If, in any particular case, a number of the above factors were operative and their effects additive, it can be shown that the safety factor of 1.20 in accordance with 4.23 kN soak strength may, in actuality, be as high as 1.5 to 2.

The minimum acceptable Marshall value of 4.23 kN suggested here appears to be in order, considering that the Asphalt Institute recommended a minimum value of 3.34 kN (750 lbs) for cutback asphalt paving mixtures (16).

Cement Requirment

Cement treatment of ETM is shown to be very effective with sands (7,17,20). Terrel and Wang (19) recommended up to 1% cement in selected aggregates primarily as a measure to overcome the detrimental effect of adverse curing conditions. Schmidt et al. (17) reported that the effectiveness of cement diminished as the cement approached 3%; accordingly, they favored 1.3% cement in ETM.

Figure 5. Variation of Marshall stability and bearing strength with cement content.



Note: 1 kN = 225 lbs, 1 kPa = 0.145 lbf/in.²

The question not yet addressed in these studies is whether one can prescribe an optimum cement content for a given sand emulsion mixture. Marshall stability, flow and, thereby, bearing strength of all six soils with cement 0.5%, 1% and 1.5% were determined. A typical plot is shown in Fig. 5. A general trend in these results is that although the Marshall stability increment decreases with cement content, corresponding bearing strength increase is somewhat exponential at the high end of cement content, due primarily to the decrease in deformation. The fact that environmental stresses and consequent pavement cracking will be more prevalent in less flexible materials, therefore, led the writer to propose that, in order to be of optimum benefit to CETM, the cement content should be limited to 1.5%.

This result was corroborated in a recent field test program where a CETM base was constructed with K-44 soil at 6% emulsion and 2% cement. Although the drying shrinkage of the CETM was well below what is considered to be critical for pavement cracking, the base developed cracks to the tune of 0.39 m/m² (.12 ft./ft.²). That the core strength has typically increased from 10,340 kPa (1500 psi) in 28 days to 17,230 kPa (2500 psi) in 18 months suggests that cement not only acts as a catalyst to increase the rate of curing of ETM but plays a major role in stabilizing the sand.

When the cement content was decreased from 1.0% to 0.5%, a good many of the soils became so susceptible to moisture intrusion that not only did their soak stabilities drop below the minimum of 4.23 kN, but also their retained strength plunged to somewhere in the neighborhood of 20% to 30%. Cement content of 0.5% is insufficient; therefore, the writer proposes that the optimum cement content should be between 1% and 1.5%. Exceptions to this general rule may be cited; for example, K-46 is sufficiently modified with 0.5% cement. Only a larger cement content of 1.5% has brought the soak stability and moisture absorption of K-45 to acceptable levels.

The writer asserts that the selection of cement content should be governed by the dry strength of CETM. In other words, a dry strength criterion is in order here. This can be accomplished by estimating a dry bearing strength corresponding to the acceptable soak bearing strength of 1082 kPa (157 psi). Anticpating a soaking condition in the field as severe as that in vacuum soak a loss of 60% could be a conservative value. Accordingly, the desirable dry bearing strength would be (1082 x 2.5) nearly 2706 kPa (392 psi) rounded to 2760 kPa (400 psi). In fact, no requirement in the field justifies a bearing strength higher than 2760 kPa.

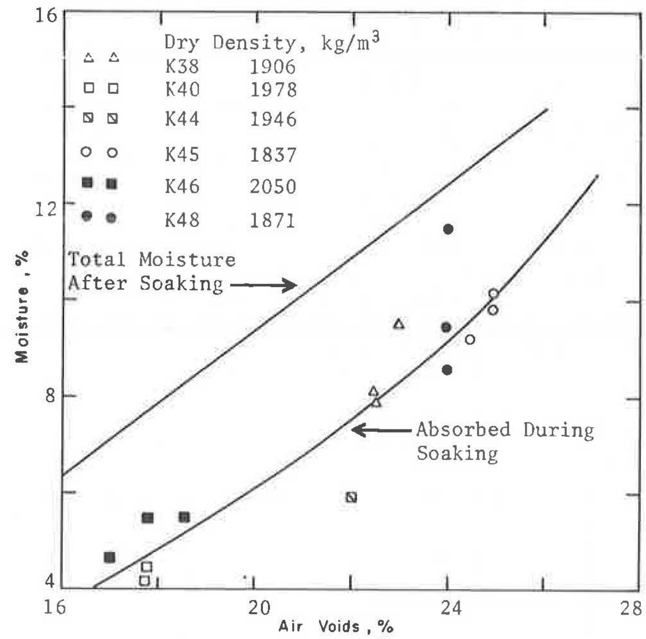
Permissible Moisture Absorption

Since moisture absorbed by the stabilized mixture is highly detrimental to its stability, and since this deterioration is dependent upon the extent of absorption, assigning a limiting value to the moisture absorption is considered important. The results show that the moisture absorption (percentage of moisture absorbed during vacuum soaking after seven-days air curing)increases with percentage of air voids (Fig. 6), which in turn is inversely proportional to the dry density of the mix (see inset of Fig. 6). Furthermore, the total moisture content (retained moisture after seven days plus moisture absorbed during soaking) increases linearly with the air voids. The spread between those two curves gives the moisture retention by CETM which slightly decreases with a decrease in density. When comparing the moisture contents during compaction, after seven-days curing, and after vacuum soak two important results emerge. First, the moisture retention after seven-days partial curing varies from 2.5% to 3.5%, depending on the fines content. Second, soils whose total moisture after soaking is much greater than the molding moisture (optimum moisture) would likely be susceptible to moisture in the field. As a rule of thumb the ratio of the former to the latter should not exceed 1.5; ideally, the ratio should be unity.

The importance of moisture absorption becomes even more subtle as we note that the Marshall stability decreases logarithmically with increasing absorption (Fig. 7). It is apparent from the curve that CETM mixture exhibiting a soaked Marshall strength of 4.23 kN (950 lbs) will normally have absorbed 9.4% moisture during vacuum soaking. Taking into account the 95% confidence interval for this estimate the writer suggests that maximum permissible moisture absorption by Marshall specimens should not exceed 8.5%.

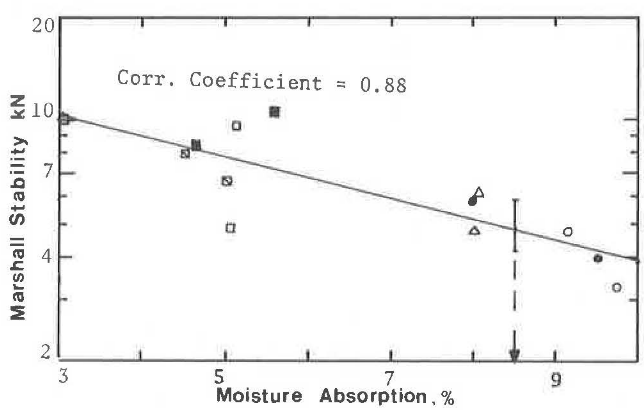
It is significant to note here that the moisture absorption value proposed in this study is compatible with the value sggested by the Asphalt Institute: 5% for selected aggregates. The moisture absorption, according to the Asphalt Institute, should be determined by MVS test. Our tests show that the moisture absorption during MVS test is about 40% to 50% of what would normally be absorbed in a vacuum soaking test. Thus, the permissible moisture absorption of 5% suggested by the Asphalt Institute is comparable to the 8.5% moisture pickup in this study where moisture absorption is determined by the vacuum saturation method.

Figure 6. Moisture absorption related to air voids.



Note: 1 kg/m^3 = 0.0624 lb/ft^3

Figure 7. Marshall stability related to moisture absorption.



Note: 1 kN = 225 lbs.

Conclusions

1. Portland cement in trace quantities, acting as a stabilizing agent, greatly enhances the soak-stability of sand-emulsion mixtures.
2. Soaked Marshall stability of CETM increases with emulsion content; attains an optimum value somewhere between 6% and 9%; and, for all practical purposes, remains constant, The effect of fines content on Marshall stability is such that the stability increases with fines, peaks at about 15% to 18%, and

then gradually decreases.
3. It is feasible to estimate the (seven-day cured vacuum soaked) Marshall stability of CETM from the particle size distribution of the soil (equation 1).
4. The trend of triaxial shear strength result is in agreement with that of Marshall strength in that the shear strength exhibited an optimum value, respectively, with the emulsion content (approximately 7%) and the fines content (approximately 17%).
5. Using the test results on several sandy soils mix design criteria for CETM is proposed. The two-part criteria read as follows:
(i) Seven-day partial air-cured vacuum soaked Marshall cylinders at 25±1°C (72±2°F) (with 1% cement) should exhibit a minimum stability of 4.23 kN (950 lbs.).
(ii) Moisture absorption during vacuum soaking should not exceed 8.5% by weight.

Proposed Mississippi Method for CETM Mixture Design

1. Determine the particle size distribution of soil aggregates (ASTM D1140 and D422).
2. Determine the plasticity index of fine fraction (ASTM D423 and D424).
3. Consider soil aggregates suitable for emulsion stabilization:
(a) if the fines content lies between 5% and 25% and
(b) if the product of the fines content and PI is less than 72.
4. Determine the CKE oil ratio (Reference 16).
5. Determine type and grade of emulsion by coating test (Reference 16).
6. Determine moisture and corresponding density of CETM from moisture density curve; the details of obtaining such a curve can be seen in reference 8. Mixing moisture may be 2%-3% more than that for compaction, depending upon the fines content.
7. Use the equation,

$$\ell n(\text{MS}) = 6.7420 + 0.0943\ (\text{FC}) - 1.9866\ (\text{Cm}) - 0.0462\ (\text{EC})^2 + 0.4529\ (\text{EC} \times \text{Cm}) \qquad (1)$$

to determine the stability value (lbs) at the emulsion content of 1.1 x CKE oil ratio and cement 1%. If the stability predicted by Eq. 1 is greater than 4.23 kN (950 lbs), it would appear that the soil can be stabilized with emulsion and trace cement.
8. (a) Mold Marshall specimens at emulsion contents of 1.1 x, 1.3 x, and 1.5 x CKE oil ratio (three for each emulsion content) and with 1% cement admixture. (b) Air-cure these specimens for seven days at a temperature of 25±1°C (72±2°F) and 55% RH. (while curing they should be kept in the mold or wrapped in such a way that they undergo drying from the top face only— partial air-cure.) (c) Vacuum soak the specimens for two hours and then test for Marshall stability. (d) Weigh the specimens before and after vacuum soak to determine the moisture absorption during vacuum soak.
9. The suitability of a CETM mixture is governed by the following design criteria.
(a) Seven day partial air-cured vacuum-soaked Marshall cylinders (with 1% cement) should exhibit a minimum stability of 4.23 kN (950 lbs).
(b) Moisture absorption during vacuum-soaking should not exceed 8.5% by weight.
10. (a) In the event that the mixture with 1% cement additive do not satisfy the criteria proposed in step 9, increase the cement to 1.5%, repeat step 8, and mold six specimens from each emulsion mixture. (b) Subject the six specimens to partial air-cure for seven days. (c) Test three of these specimens for Marshall stability when dry, and the remaining three after vacuum soak.
11. The selection of a CETM mixture is governed by the following criteria:
(a) Criterion (a) of step 9
(b) Criterion (b) of step 9
(c) Seven-day dry bearing strength (calculated from Eq. 3) should not exceed 2760 kPa (400 psi).
NOTE: The purpose of criterion 11-c is to safeguard against selection of a mixture that beomces highly stiff upon drying. The bearing strength 2760 kPa (400 psi), therefore, should not be construed as an absolute maximum limit but should be viewed as a general guide only.
12. Although criteria of step 9, or alternatively those of step 11, govern the selection of emulsion content, the minimum emulsion in any event shall not be less than 1.1 x CKE oil ratio.
13. As the bitumen content in emulsion varies, the residual bitumen on weight basis should be specified for control purposes.

Acknowledgement

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The opinions, findings and conclusions expressed in this report are those of the author and not necessarily those of the State Highway Department or the Federal Highway Department. This report does not constitute a standard, specification or regulation.

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EFFECTS OF COMPACTION DELAYS AND MULTIPLE TREATMENTS ON THE STRENGTH OF CEMENT STABILIZED SOIL

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The effects of delayed compaction on the strength and durability of cement stabilized soils for base courses and subbases has been investigated. It was concluded that time delay does not adversely affect either strength or durability. However, prolonged time delay does increase the required level of compaction necessary to achieve a specified density so that it may be beyond the capability of ordinary highway compaction equipment. An example is given in the report showing how the results of this work can lead to more rational field compaction specifications for cement stabilized materials, in recognition of the fact that time delays are inherent in the construction process.

Aggregate supplies for road building purposes are becoming increasingly more costly and more scarce in many parts of the United States (10). Alternative road base materials such as cement treated aggregate or cement stabilized subgrade soil, which previously were relatively expensive, are now viable economic alternatives. However, use of such materials in lieu of a conventional aggregate base requires additional considerations to assure that a strong and durable material will result.

It is not uncommon to have a time delay between initial mixing and final compaction in excess of two hours for road-mixed cement stabilized construction. Previous research (1, 2, 3, 4, 5, 6, 7) has shown that time delays of this magnitude will result in the reduction of density, strength and durability of laboratory specimens compacted at constant compactive effort, such as that imparted by the standard AASHTO procedure. The reduction of these properties in a cement stabilized soil is a result of the time-dependent reaction that is occurring between the soil, cement, and water. As these constituents are mixed together the cement hydrates, and over time the products of hydration will effectively aggregate the soil particles. The increased frictional resistance to compaction that this creates requires an increase in the compactive effort necessary to achieve the required density.

A review of the literature (1, 2, 4, 6) indicates that if a high state of density can be achieved in cement stabilized mixtures subject to time delays, a strong and durable product will usually result. Most of the studies to date that have considered the effects of time delays on cement stablized soils have been limited to the use of standard AASHTO compaction effort. Since both dry density (related to void ratio) and moisture content (related to water/cement ratio) will ultimately affect strength gain, these factors should be considered jointly when cement stabilized soils are used.

In terms of field applications, it has generally been found (1, 3, 4, 6, 8, 17, 18) that increased compactive effort is beneficial to the dry density and therefore to the strength of cement stabilized soils. Previous research, however, does not indicate how much compactive effort is necessary to obtain 100 percent of the standard AASHTO dry density over varying time delays. In addition, if a level of compactive effort cannot be applied in the field to achieve 100 percent of the standard AASHTO dry density, it would be desirable to know what a reasonable specification would be for dry density, moisture content, and time delay, such that satisfactory strength and durability could be obtained.

Purpose and Scope of the Investigation

The purpose of this investigation was to study the effects of varying amounts of time delay before compaction on the strength of cement stabilized soils, in order that recommendations for field construction procedures could be made.

The investigation was carried out in three phases. Phase I consisted of determining the influence of moisture content on the strength and dry density of three conventional cement stabilized soil mixtures. Previous research (4, 5, 7) has shown that the optimum moisture content for maximum strength and for maximum dry density may not always be the same. In Phase I the optimum moisture content and dry density associated with the maximum strength of each cement stabilized soil mixture were determined using the standard AASHTO compaction method. These values were used in the two subsequent phases of the research.

Phase II involved an investigation of the influence of time delays on the dry density, strength, and durability of one of the three cement stabilized soil mixtures, compacted at the optimum moisture content for maximum strength. A kneading compactor was used to compact the specimens, using a compactive effort that permitted achieving the standard AASHTO density at zero time delay. These specimens were termed the "constant compactive effort" series, since the same compactive effort was used for all specimens, regardless of the amount of time that compaction was delayed. The data obtained were used to determine whether a direct relationship existed between strength and dry density, or durability and dry density.

Phase III involved compacting cement stabilized soil specimens to approximately 100 percent of the standard AASHTO dry density after the mixtures were subject to time delays of up to 6 hours. These specimens were termed the "constant density" series since the same density was attained, regardless of the amount of time that compaction was delayed. The amount of compactive effort required to obtain the desired dry density was recorded so that it could be compared to the capability of typical field compaction equipment.

For all three phases of the investigation both one-part and two-part cement stabilized soil mixtures were investigated. One-part mixtures are defined as those where all of the soil, cement and water are mixed together at one time. Two-part mixes involve mixing one-half of the cement and one-half of the water initially, followed by a twenty-four hour hermetic curing period prior to the addition of the remaining cement and water. This technique of multiple treatment (two-part mixing) represents the extreme antithesis to the conventional wisdom of minimizing the time for construction of soil-cement mixtures. It occurs occasionally in mixed-in-place construction where mixing is interrupted for a prolonged period or overnight. Earlier research (6) has suggested that some strength benefits were obtained using two-part mixing, however this research did not confirm those findings.

Materials and Methods

Soils

Three soils sampled from sites within Tompkins County, New York were used for testing. Soil A was a silty clay and Soil C was a clayey silt, each typical of the fine-grained subgrade soils found over a major portion of the United States. Soil B, a highly angular gravelly sand, was typical of the coarse-grained dense-graded materials used in the construction of cement treated aggregate bases. The physical properties of these three soils are shown in Table 1.

Portland Cement

The cement used throughout this investigation was Type IA portland cement, having an initial setting time of 100 minutes, a final setting time of 204 minutes, and a 28-day mortar cube strength of 730 kPa (5012 psi).

Mix Designs

Standard procedures for the determination of optimum cement content published by the American Society for Testing and Materials (11) and the Portland Cement Association (14) were used to obtain the mix designs. Freeze-thaw criteria controlled the mix design for Soil A, resulting in a design cement content of 13.5 percent by weight of dry soil. The short-cut procedure for sandy soils (14) was used for Soil B and Soil C, resulting in design cement contents of 5.0 and 12.0 percent, respectively.

Table 1. Properties of unstabilized soils.

	Soil A	Soil B	Soil C
Sieve Analysis			
Percent Passing			
19.1 mm (3/4 in)	100	100	100
13.2 mm (1/2 in)	100	90	100
4.75 mm (#4)	100	58	99
2.00 mm (#10)	100	42	98
0.425 mm (#40)	97	18	95
0.075 mm (#200)	90	2	87
0.050 mm	89	--	78
0.005 mm	57	--	25
0.002 mm	39	--	17
Physical Properties			
Liquid Limit	36		24
Plastic Limit	17		17
Plasticity Index	19	NP	7
Standard AASHTO Maximum Dry Density (kg/m^3)	1805	2195	1842
Optimum Moisture Content (pct.)	16.4	7.1	14.5
Modified AASHTO Maximum Dry Density (kg/m^3)	1919	2288	1942
Optimum Moisture Content (pct.)	13.3	5.9	12.2
Specific Gravity	2.76	2.68	2.71
Soil pH	7.8	--	7.7
Percent Loss on Ignition at 900°C	14.6	--	4.8
Classification			
AASHTO	A-6 (12)	A-1a	A-4 (8)
Unified	CL	SW	CL-ML

($1\ kg/m^3 = 0.0624\ lb/ft^3$)

Mixing

A 12-liter, restaurant-type Blakeslee Model CC20 mixer, was used to mix the soil-cement mixtures. A speed of 250 revolutions per minute was used, and a total wet mixing time of 3 minutes was allowed for all soil-cement mixtures. The cement and dry soil were first hand-mixed to a uniform texture and upon the addition of water the measurement of time delay was begun and wet mixing was initiated. For the two-part mixtures a 1.5 minute mixing time was used for each part. The measurement of time delays for two-part mixes was initiated upon the addition of the second portion of the mixing water.

Moisture Contents

All moisture contents were based on the weight of dry solids, when dried in an oven at 110°C. The moisture contents used in calculating dry densities and plotted on the moisture-density curves were based on the molding moisture content, that is, the amount of water added to the soil-cement mixture plus any hygroscopic moisture in the soil that could be evaporated at 110°C.

Compaction

All specimens used in this investigation were 102 mm (4.0 inches) in diameter by 116 mm (4.58 inches) in height. Each specimen was compacted in three equal layers and was scarified between each layer. The specimens for Phase I, the moisture-density-strength study, were compacted in accordance with ASTM D558-57 (11), using a drop-hammer compaction corresponding to the standard AASHTO effort. In no instance were the compacted specimens broken up to be recompacted.

The soil-cement specimens for the remainder of the investigation were compacted using a kneading compactor. The kneading foot and the load-duration relationship conformed substantially to the test methods of the California Department of Transportation. The compactor was capable of applying contact pressures up to 5000 MPa (700 psi).

Curing

All specimens were cured for 7 days at 23 plus or minus 1°C (73 ± 2°F) in a moist atmosphere near 100 percent relative humidity. Each specimen was sealed in a plastic bag during curing to maintain a constant moisture content.

Strength

The unconfined compressive strength of each specimen was measured. After curing, the ends of each specimen were capped using a sulfur-type capping compound. The tests were run on a universal testing machine having a spherically seated loading head using a loading rate of 1.25 mm (0.050 inches) per minute.

Durability

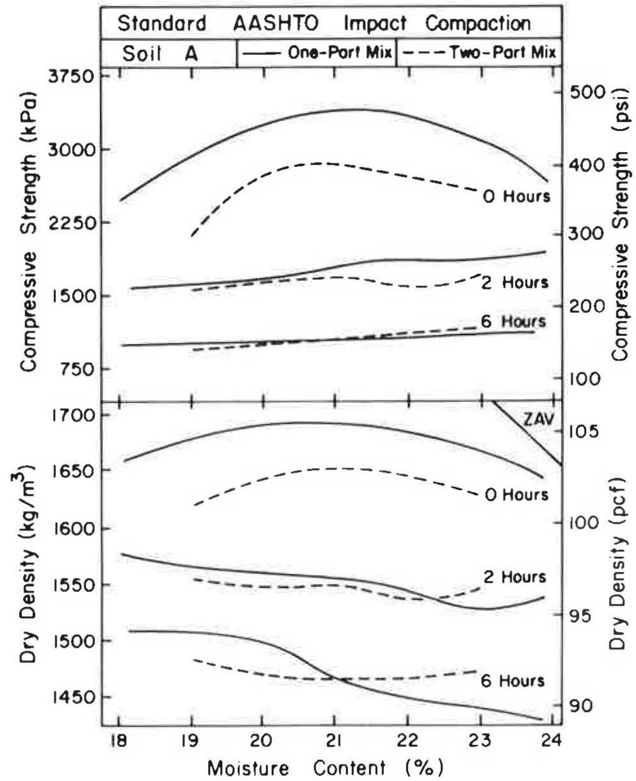
The strength of specimens subject to vacuum saturation has been found to have a high correlation with the strength of samples subject to freeze-thaw cycling (16). After 7 days of curing, the specimens were subjected to vacuum saturation, capped, and tested for unconfined compressive strength.

Discussion of Results

Moisture-Density-Strength Relationships

Figures 1, 2, and 3 illustrate the influence of moisture content on the dry density and strength of the three cement stabilized soils. A minimum of fifteen compacted specimens were fabricated in establishing each of the curves shown in the three figures. The results for the two fine-grained soils (Soils A and C) show that the moisture content for maximum strength of a one-part mix subject to a minimum time delay (0 hours) was the same or slightly greater than that for maximum

Figure 1. Moisture-density-strength relationship for cement-stabilized Soil A.



dry density. However, for the coarse-grained Soil B the moisture content associated with maximum strength was lower than that for maximum dry density. It can be seen in Figure 2 that deviations in moisture content from the optimum for maximum strength of the coarse-grained material will cause large decreases in strength.

The dependence of strength on moisture content is a result of the manner in which the type of soil obtains strength when stabilized with cement. Strength gain in coarse-grained soils is similar to that of concrete. The function of the cement paste in concrete is to fill the voids within the aggregate matrix and bond the particles together. In cement stabilized soils however, the voids are not completely filled with paste (18). Strength in a cement stabilized coarse-grained soil can therefore be seen as a function of both water-cement ratio (a specific moisture content associated with a given cement content) and degree of compaction (dry density).

For fine-grained soils, strength gain is obtained through cementation as well as through the chemical combination of the individual soil particles with the products of cement hydration. While the dry density and moisture content are primary factors influencing strength gain, so are the mode of compaction (19), cement content, soil chemistry, temperature, and mixing technique, along with many other variables that affect the soil-cement reaction (23, 24, 25).

Table 2 summarizes the results of the moisture density-strength tests for zero time delay. The moisture content for maximum strength and the associated dry density, for each of the three cement stabilized soils, were used as the controlling parameters for Phase II and Phase III of the research: the constant compactive effort study and

Figure 2. Moisture-density-strength relationship for cement-stabilized Soil B.

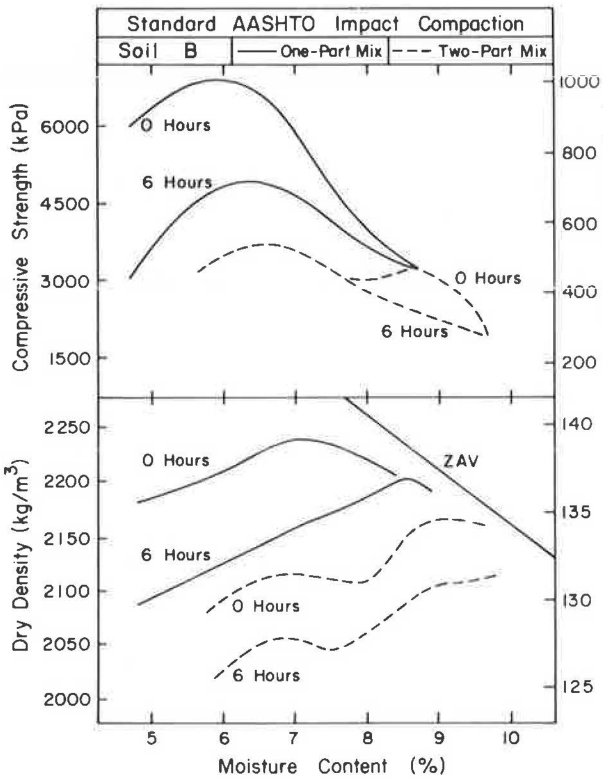
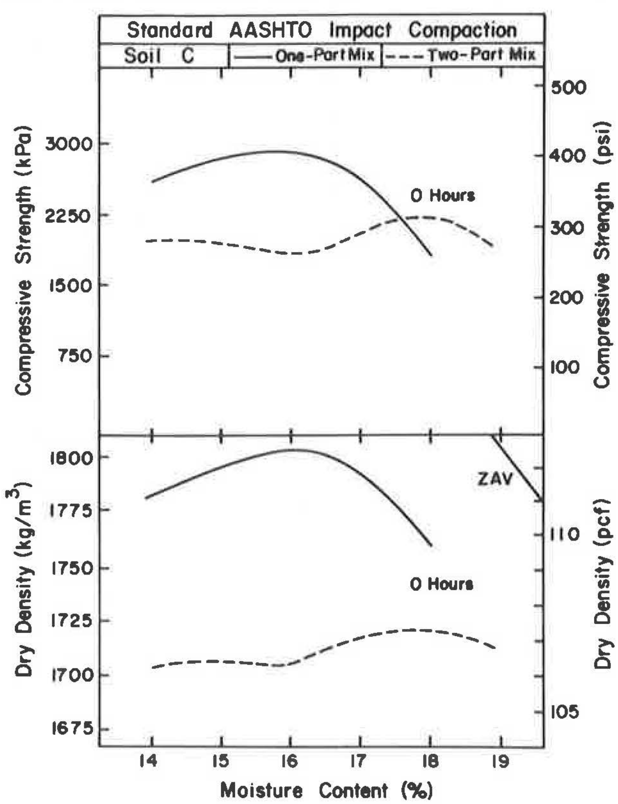


Figure 3. Moisture-density-strength relationship for cement-stabilized Soil C.



the constant density study. Since kneading compaction rather than drop-hammer compaction was used in the subsequent phases of the research, the compactive effort (i.e., the hydraulic pressure setting) necessary to achieve the desired density at the given moisture content for zero time delay was determined for each of the stabilized mixtures. Because the standard AASHTO density was attained at this pressure setting, it is termed the standard AASHTO compaction pressure. In a similar manner the pressure setting necessary to achieve modified AASHTO density was determined. These pressures are reported in Table 3 for each soil-cement mixture.

Table 2. Moisture-density-strength relationships for cement stabilized soils using standard AASHTO impact compaction, zero time delay, and one-part mixing.

		Soil A	Soil B	Soil C
Maximum 7-day Unconfined Compressive Strength	(kPa) (psi)	3310 480	6960 1010	2790 405
Maximum Dry Density	(kg/m^3) (pcf)	1689 105.4	2220 138.6	1804 112.6
Moisture Content for Maximum Density	(percent)	20.5	7.0	16.0
Moisture Content for Maximum Strength	(percent)	21.0	6.3	16.0
Cement Content	(percent)	13.5	5.0	12.0

Table 3. Kneading compaction pressure required to achieve standard and modified AASHTO densities for cement-stabilized soils at zero time delay with one-part mixing.

		Soil A	Soil B	Soil C
Standard AASHTO Compaction Pressure	(kPa) (psi)	1170 170	1520 220	1100 160
Modified AASHTO Compaction Pressure	(kPa) (psi)	2620 380	2410 350	2620 380

Constant Compactive Effort Study

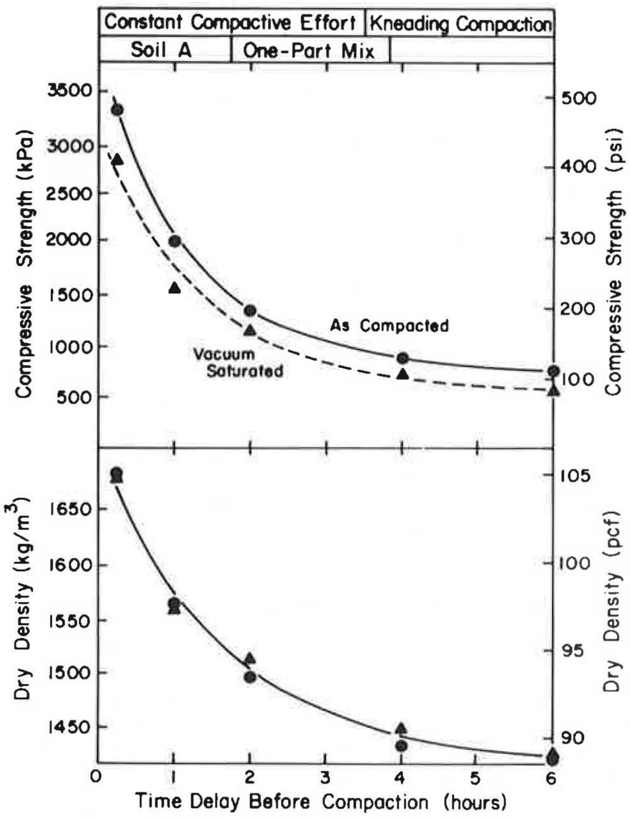
The influence of time delay upon the dry density, strength, and durability of cement stabilized Soil A was investigated in Phase II of the research. Specimens compacted with the kneading compactor were given a compactive effort equivalent to that of standard AASHTO compactive effort, as determined by varying the hydraulic pressure setting on the compactor until at zero time delay the density at the optimum moisture content for maximum strength was equal to the standard AASHTO density.

The effect of delaying compaction up to six hours is shown in Figure 4. Each data point represents the average of three test specimens. Where a constant compactive effort is used, time delay before compaction results in a significant loss of density, which in turn results in strength reduction. The direct interrelationship between density and strength is also shown in Figure 5.

This behavior has also been noted by MacLean and Lewis (8).

The durability of the material, as measured by the strength retained after saturation, is also shown in Figure 4. It can be seen that regardless of the amount of time delay, approximately 80 percent of the as-compacted strength was retained after saturation. Thus time delay before compaction does not appears to have a major influence on durability losses for cement stabilized soils.

Figure 4. Effect of time delay on strength and density of cement stabilized Soil A at constant compactive effort.



Constant Dry Density Study

Time Delay Influence on Strength. The preceding results imply that time delay causes reduced density, and thus reduced strength, where compactive effort is held constant. It should be noted, however, that most construction specifications require that a minimum density be achieved. The question of whether there would be a loss in strength due to time delays where a constant density is achieved was the subject of the third phase of the research.

In Figure 6 are shown the results of strength tests on the three cement stabilized soils, where the compactive effort at each level of time delay was adjusted to enable attaining approximately 100 percent of standard AASHTO density. For Soils A and C each data point represents the average of three tests. For Soil B each data point represents a single test.

Soil B presented difficulty in achieving constant density. Slight variations in density resulted in large variations in strength. The influence of density on strength was quantitatively defined in a manner similar to Figure 5, and the individual test results were normalized to adjust for density variations before they were plotted in Figure 6.

Figure 5. Strength-density relationship for cement stabilized Soil A.

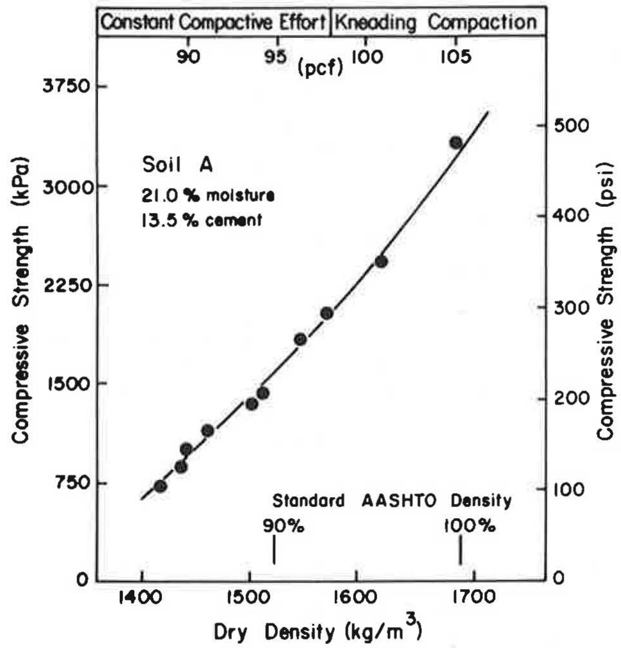
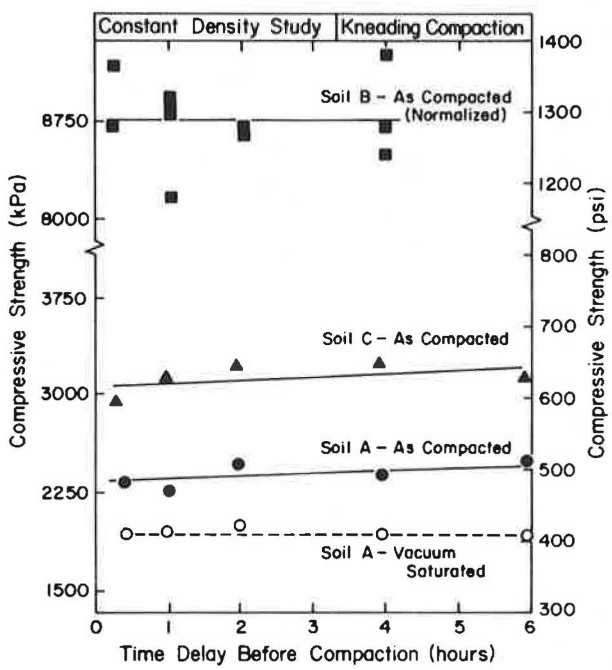


Figure 6. Effect of time delay on strength of cement stabilized soils at constant density.



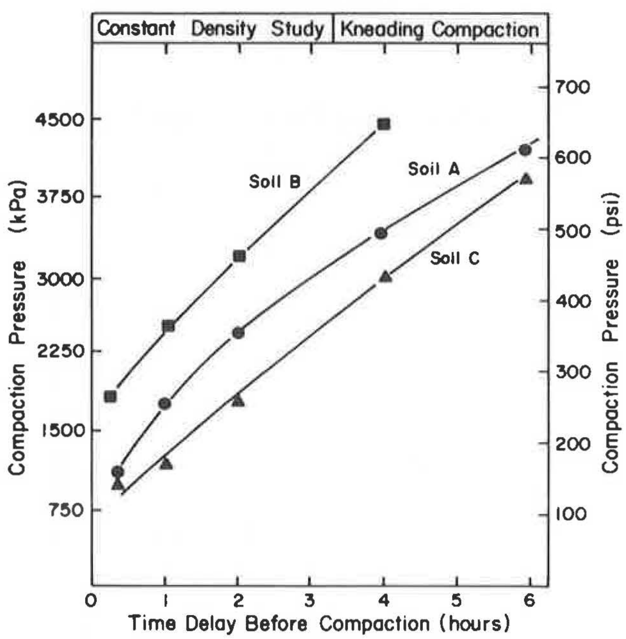
Statistical analyses have determined that none of the lines reported in Figure 6 have slopes which are different from zero at the 0.05 significance level (20). Thus it may be concluded that time delays have no influence on strength when a given density is achieved.

Again it is seen in Figure 6 that saturation of Soil A specimens resulted in approximately 80 percent strength retention, regardless of the amount of time delay. These results further support the previous conclusion that time delays up to 6 hours do not adversely affect the durability of soil-cement.

Time Delay Influence on Compaction Effort. It has been shown that time delays of up to six hours do not result in strength losses where a given density can be achieved. However, time delay does make it increasingly difficult to achieve density, as shown in Figure 7. In the figure it can be seen that the level of compactive effort, as measured by the pressure setting on the kneading compactor, increases significantly with increasing time delay. The coarse-grained Soil B exhibited a higher frictional resistance for all levels of time delay than did either Soils A or C, which were fine-grained.

At zero time delay a kneading pressure of about 2600 kPa (380 psi) was sufficient to provide at least modified AASHTO density in all three cement stabilized soils (Table 3). As shown in Figure 7, however, after one to three hours of time delay, depending upon soil type, a kneading pressure of 2600 kPa would only provide standard AASHTO density.

Figure 7. Effect of time delay on compaction pressure required to achieve standard AASHTO density for cement stabilized soils.



Limiting Time Delays. It has been reported that the modified AASHTO compaction curve represents an upper bound on the density that can generally be produced by most types of field compaction equipment (21, 22). For the purposes of this study a limiting time delay was defined as the time delay at which modified AASHTO compactive effort was required to obtain standard AASHTO density. Based upon the results reported in Table 3 and Figure 7, for the cement stabilized soils A, B, and C the limiting time delays would be 2.3, 1.0, and 3.3 hours, respectively.

It can be concluded that it would be very difficult to attain a specified standard AASHTO density for a cement stabilized soil when a time delay between mixing and compaction in excess of three hours is incurred. The higher frictional resistance of coarse-grained soils, such as those used in base courses, may be expected to result in a limiting time delay more on the order of one hour.

Significance of Cement Time of Set

The cement used in these tests had an initial setting time of 1.7 hours, and a final setting time of 3.4 hours. Arman and Saifan (2) have noted the influence of cement setting time on the behavior of fine-grained soil-cement mixtures. In Figure 4 it can be seen that the greatest strength losses occur prior to initial set, and that after final set only a small amount of strength loss occurs. It should be noted however that the setting test for cement is somewhat arbitrary, and it depends on the mechanical response of the paste to applied load. Setting time is therefore defined by the frictional resistance of the hydrating paste. Similarly, cement stabilized soils exhibit increased frictional resistance to applied compaction, when compaction is delayed for a period of time (Figure 7). However, when sufficient compactive effort is applied to overcome the frictional resistance, such that a specified density is achieved, no loss of strength results. The data in Figure 7 do not suggest that either initial or final setting time had any major influence on the compactive effort required to achieve strength and density in the three soil-cement mixtures considered in this investigation.

Application to Compaction Specifications

The intent of this research was to establish a method for determining a realistic field compaction specification for cement stabilized soils subject to time delays before compaction. The results of this research have shown that a limiting time delay of one to three hours, depending on soil frictional characteristics, can be tolerated if cement stabilized soils are to be compacted to 100 percent of standard AASHTO density with field compaction equipment. Alternatively, time delays greater than the limiting time delay can be expected to result in reduced dry density and therefore lower strength.

For satisfactory field performance of cement stabilized soils, achieving a given minimum level of strength may be expected to be of primary importance. Strength determines the load supporting capability of the pavement, and it has an important influence on the durability and fatigue resistance of the material. This research has shown that the strength of soil-cement is controlled by the density that is achieved during compaction. Since most construction specifications for cement stabilized soils include a requirement for minimum density, the strength and satisfactory performance of the material is determined by the degree to which the specification is reasonable.

The way in which the procedures described in this report may be used to develop reasonable compaction specifications will be illustrated in the following example.

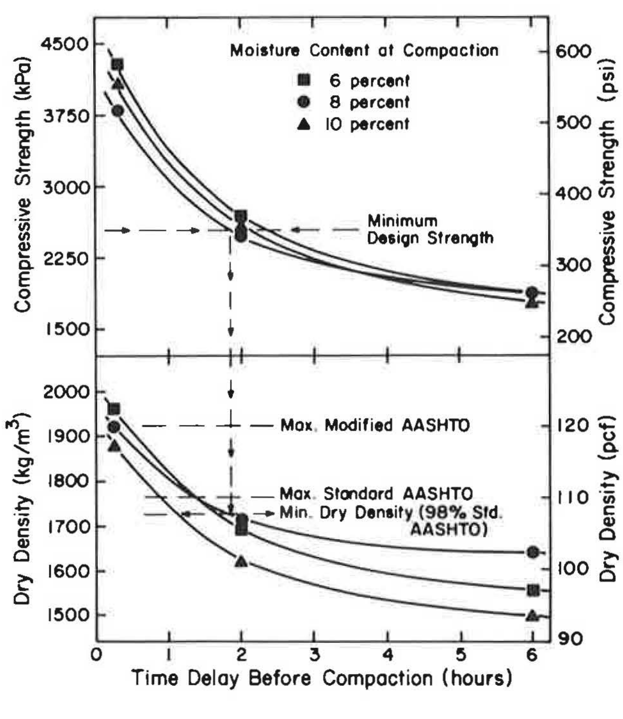
1. It is first necessary to select a minimum acceptable 7-day compressive strength. Local experience can determine this value. A minimum strength of 2400 kPa (350 psi) is recommended where local experience is insufficient.

2. For the selected soil, the minimum required cement content is determined using conventional procedures (14). Careful attention must be paid to avoiding time delays before compaction in this work. The compacted laboratory specimens from the moisture-density investigation are cured for 7-days at 23°C (73°F) and 100 percent relative humidity. The specimens are capped and their compressive strengths determined. Test results are plotted in a manner similar to Figures 1-3. The moisture content and density for maximum strength are determined and will be used in the compaction specification.

3. Using the same compaction procedures as in step 2, additional specimens are prepared after time delays of two hours and six hours, using optimum moisture content, and optimum plus and minus two percentage points. The specimens are cured as before and the strength and density results are plotted as in Figure 8.

4. The results shown in Figure 8 are interpreted as follows. Entering with the minimum acceptable strength (from step 1), the minimum allowable percentage of standard AASHTO density that will permit this strength to be attained can be determined. This minimum density will be used in the compaction specification. The associated maximum allowable time delay can be read in Figure 8 (approximately two hours in this example). If the time appears to be unrealistically short for the expected construction procedures, then either the minimum acceptable compressive strength must be reduced, or the design cement content must be increased.

Figure 8. Example of application of methods to construction specifications for a cement stabilized soil.



For this example a typical compaction specification might read: "The cement stabilized soil shall have a minimum cement content of 5 percent by weight of dry soil. It shall be compacted at a moisture content of 8 plus or minus 2 percent. A minimum compacted dry density of 172.5 kg/m^3 (107.7 lb/ft^3) shall be achieved." Consideration should be given to utilizing quality control measures such as running average density or probability-based control measures. The use of these measures is widely reported in the literature. Consideration might also be given to specifying a limiting time delay, after which compaction should not be attempted. Care should be taken to avoid making this time period too short, so that it does not subordinate the minimum density requirement.

Conclusions

This investigation has studied the effects of delayed compaction on the strength and durability of three cement stabilized soils, representative of base course and subgrade materials. The results of this investigation have led to the following conclusions:

1. If a specified level of density is achieved, no adverse effects on the strength or durability of cement stabilized soils will be attributable to time delays between mixing and compaction of up to six hours.
2. Time delays greater than one to three hours will increase the required compactive effort for cement stabilized soils to a level which may be beyond the capabilities of ordinary highway compaction equipment.
3. For any particular cement stabilized soil, a limiting time delay can be identified, after which time the required density cannot be achieved.
4. Multiple-part treatment does not appear to offer any advantages over ordinary one-part treatment in terms of the compressive strength of cement stabilized soils.
5. Moisture content, compactive effort, time delay, and soil type jointly affect the strength and durability of cement stabilized soils. A rational means of considering these factors in preparing specifications for field construction control has been described in this report.

Acknowledgements

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POZZOLANIC ACTIVITY AND MECHANISM OF REACTION OF RED TROPICAL SOIL-LIME SYSTEMS

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This paper presents a study on the pozzolanic properties and mechanism of reaction of nineteen red tropical soils from north east Brazil. The results of the study show that the mechanism of reaction between soil and lime is mainly a diffusion controlled process which can be expressed by the equation of Jander. Based on the constant rates of reaction obtained from this equation and on the quantity of reactive components of the soils an activity index A_i is proposed and its validity tested against the unconfined compressive strength of the lime-soil system. A valid statistical correlation is presented linking the A_i value with the empirical reactivity parameter of Thompson.

Approximately one third of the land surface of the earth is contained within the huge belt limited by the tropics of Cancer and Capricorn. Within this area "red tropical soils" are the dominant feature of the landscape, and whenever man decides to construct any facility, red tropical soil properties have to be determined and their behaviour predicted.

Geologists, soil scientists, agriculturalists and engineers are interested in different aspects of the behaviour of red tropical soils. This interest is reflected in the great volume of work devoted to classification and preparation of an acceptable nomenclature of red tropical soils. Unfortunately for a civil engineer these classifications are very confusing and of limited use. The terminology and classifications in actual use are so varied that an engineer is tempted to agree with F.C. Bawden who said "Classification is easy: it is something you just do" (1). Ideally, a classification should serve not only as a practical tool to transmit information but as a tool to predict behaviour when experience is not available.

Information regarding the engineering behaviour of red tropical soils is still scanty, this is reflected in the lack of classifications with engineering validity. An example of a classification for engineers is the one proposed by Lyons Associates in their study of African soils (2) and of South American soils (3). These classifications, the first related to d'Hoore's simplified classification and the other related to the FAO-UNESCO classification have had moderate success and can only be used as a very rough guide to the properties of red tropical soils. Perhaps a great part of the useful data gathered on the behaviour of red tropical soils has never been fully used due to the confusion and controversy originated by the very nomenclature proposed by researchers in their desire to precisely define these materials. Terms like laterite, lateritic soil, latosol, ferricrete, ferruginous soil, ferrisol, oxisol, ferralsol, etc. have been used for the same type of material in some cases, while only one name has been applied to various types of materials in other cases. Therefore in this paper use is made of the term "red tropical soil" and this is broadly a soil which fits the definition of laterite given by Siverajasingham et al. (4), while the term laterite is restricted to the original definition given by Buchanan (5).

This paper deals specifically with the pozzolanic properties of red tropical soils, it attempts to explain the mechanism of the reaction between lime and the constituents of the soil and proposes an index of pozzolanic activity which is related to the strength of the soil by an empirical statistical relation. Finally, it suggests the activity index as a possible parameter for the classification of red tropical soils.

Lime-Soil Reaction Mechanisms

The different theories on the mechanism responsible for the alteration of some of the engineering properties of soils when mixed with lime in the presence of water have been well reviewed by Diamond and Kinter (6) and more recently by the Transport and Road Research Board U.S.A. (7). The changes that take place when lime is added to a soil are broadly divided into amelioration and/or strength development.

Among the mechanisms offered for the explanation of the amelioration effect of lime on soils are: flocculation due to cation exchange (pH dependent), physical adsorption, minor pozzolanic reactions between soil minerals and lime leading to diffuse cementation.

Flocculation due to cation exchange has been supported by the Iowa State University researchers. Hilt and Davidson (8) explained that amelioration occurred because lime increases the pH of the system thereby generating negatively charged sites at the edges of the clay resulting in an increase in cation exchange capacity. The small amount of lime required for these changes to take place was termed "lime fixation point". Hilt and Davidson indicated that strength increases in the lime-soil system only occur when lime is added beyond this point. Cation exchange phenomena is real enough in many materials, it is a very useful property utilized in a great number of industrial processes, thus it cannot be discounted here. However, the predominant clay mineral in red tropical soils is of the 1:1 type, these clay minerals have low cation exchange capacities and low specific surface values. It is therefore questionable whether this mechanism alone can explain amelioration in red tropical soils.

The physical adsorption theory offered by Diamond and Kinter (6) postulates that lime molecules are adsorbed by clay surfaces and react with edges of other contiguous clay particles to form minute cementitious products.

The diffuse cementation theory proposed by Stocker (9) does not postulate adsorption of lime on the surfaces of the clay but that lime reacts directly with clay crystal edges, generating minute accumulations of cementitious products at or near the edges.

Strength increase is generally attributed to pozzolanic reactions, i.e. a process involving the production of cementitious compounds between lime and soil minerals. Eades and Grim (10) suggested that this reaction involves the dissolution of silica and/or alumina from the clay to react with lime to form the cementitious products while Stocker (9) attributes the mechanism to diffuse cementation - an in situ phenomenon.

All the above postulations have one common shortcoming. They are empirical in nature since they are not based on theoretical models which could be used for prediction purposes.

In the present study the experimental data is used to assess the validity of a theoretical reaction model.

Theoretical Reaction Models

Two fundamental reaction models are considered:- (a) dissolution of minerals out of clay to react with lime, and (b) diffusion controlled reactions between lime and soil minerals.

Dissolution model

This model is based on the assumption that the rate of dissolution of a given mass is proportional to its surface area (10), thus:

$$\frac{dw}{dt} = -cA \tag{1}$$

where $\frac{dw}{dt}$ = dissolution rate at time t

c = rate constant

A = total surface area

For uniform sized spherical particles equation 1 becomes

$$\frac{dw}{dt} = -c_1 w^{2/3} \tag{2}$$

where w = weight of undissolved material at time t

i.e. $$c_2 t = w_o^{1/3} - w^{1/3} \tag{3}$$

i.e. $$c_3 t = 1 - w_m^{1/3} \tag{4}$$

where $w_m = \frac{w}{w_o}$

and w_o = weight of undissolved material at time zero

Equation 4 is valid for a monodisperse suspension. For materials of particle size normally distributed, equation 4 becomes

$$c_4 t = 1 - w_m^{\frac{1}{4}} \tag{5}$$

For materials of particle size log normally distributed, equation 4 becomes

$$c_5 t = 1 - w_m^{1/5} \tag{6}$$

Hence if the dissolution theory is valid, experimental results should obey equation 5 or 6.

Diffusion controlled model

Kinetic models for solid state reactions based on diffusion controlled processes have been considered by Jander (12), Ginstling and Brounshtein (13) and Carter (14). The model used by Jander differs from the model of Carter in that it does not consider the effect of the varying volume of reaction products on the rate of formation of new reaction products. In the systems investigated in this study, the volume of products formed with time was not measured, therefore the experimental data was only tested with the model of Jander.

The model of Jander is based on the reaction which occurs between two spherical particles and on the assumption that the rate of thickening of the reaction products is inversely proportional to its thickness, hence:

$$dy/dt = k/y \tag{7}$$

where dy/dt = rate of thickening of the reaction products

k = rate constant

y = thickness of reaction products

Integrating equation 7:

$$y^2 = 2kt \tag{8}$$

The volume of unreacted material V, at time t is given by:

$$V = 4/3\,(r - y)^3 \tag{9}$$

or $$V = 4/3\, r^3\,(1 - x) \tag{10}$$

where r = radius of reacting sphere

x = fraction of sphere which has reacted

From equations 9 and 10:

$$y = r\,[1 - (1-x)^{1/3}] \quad (11)$$

Substituting y from equation 8 in 11

$$[1 - (1-x)^{1/3}]^2 = 2kt/r^2 = Kt \quad (12)$$

Equation 12 is the well-known Jander relation.

Hence a plot of time against the first term of equation 12 should give a straight line.

Pozzolanic Activity

Pozzolanic activity of a soil is defined as the ability of some of the components of the soil to react with lime to produce cementitious products.

The classical method of assessing pozzolanic materials (15,16) involves the determination of the amount of lime left in solution after a mixture of the material and Portland cement is kept in water at 313°K for eight days. This method does not take into account the rate at which reaction products are formed during the reaction.

Raask and Bhaskar (11) have proposed a method which takes into consideration the rate at which lime is consumed; they used a diffusion model and proposed a pozzolanic index to assess the pozzolanic activity of pulverized fuel ashes.

Other empirical methods to quantify the pozzolanic properties of materials involve the measurement of some engineering property in relation to time, for example the method of Thompson (17), which is very useful for engineering purposes, consists in measuring the unconfined compressive strength of soils mixed with the optimum percentage of lime (for maximum strength), compacted at maximum density and cured for 28 days. The difference in the value of the unconfined compressive strength of the lime-soil and the strength of the pure soil is termed the lime reactivity of the soil. The reactivity parameter has been used to assess the pozzolanic properties of red tropical soils by Harty and Thompson (18) and it is used in this paper in order to ascertain its relation with the method proposed in this study which consists of the assessment of pozzolanic activity in terms of the reaction rate and the proportion of reactive components present in the soils.

Materials and Experimental Methods

Nineteen red tropical soils from the states of Paraiba and Pernambuco in Brazil were used for this investigation. The soils were treated with different percentages of high calcium hydrated lime, cured for 28 days at 22°C and their unconfined compressive strengths measured. Their properties and strength behaviour are described in detail elsewhere (19).

Table 1 presents data on the textural composition, compaction and strength characteristics of the 19 soils studied. The value of reactivity as used by Thompson (17) is also included.

The reaction between lime and soil was studied by a method similar to that used by Barret et al. (20) which consists basically in measuring the amount of lime left in solution at different time intervals. The proportion of lime in solution was measured in this study by atomic adsorption spectrophotometry.

The detailed procedure followed during the investigation is described below:

1. Preparation of water-lime solutions by mixing 0.15 g or 0.10 g of lime with 100 ml of CO_2 free distilled water.
2. Mixing of 2.5 g of soil with the appropriate lime solution so as to obtain either 6 or 4.5 per cent lime concentration in the soil. These percentages correspond to the optimum percentages of lime for maximum strength (see Table 1).
3. Placing the mixtures of soil and lime-water in plastic containers tightly sealed to prevent carbonation of the lime.
4. Storing the sealed containers in an oven at a constant temperature of 25°C.
5. Shaking of containers periodically and measuring the lime concentrations at predetermined times. To measure lime concentration by atomic adsorption it is necessary to have very clear solutions, therefore before measuring the amount of lime in solution the lime-water and the soil were centrifuged at 2,500 rpm for about 10 minutes.

A preliminary study was conducted with two soils using three size fractions of each one, i.e. 2mm-0.074mm, 0.074mm-0.02mm and <0.002mm. This was done in order to assess the magnitude of contribution to lime consumption by the coarser fractions of the soil. It is a well-known fact that many of the coarser particles in red tropical soils are only aggregations of the clay size fractions and thus they may influence the overall pozzolanic activity.

The results of this preliminary study are presented in Table 2. As expected, the clay size fraction is overwhelmingly the most reactive and therefore it seems apparent that by using the clay size fraction a representative result can be obtained for any of the soils. Consequently the study was only conducted with the clay size fraction of the 19 soils.

Presentation of Results and Discussion

Lime consumption

One of the problems when dealing with red tropical soils, which is still a matter of debate, is related to the nature of the components of the soil which are responsible for its pozzolanic properties. Though it is not the object of this paper to deal with this aspect of the overall problem of pozzolanic activity, it appears of interest to point out that in red tropical soils the components which appear to be of major influence on the rate of lime consumption are mainly the amorphous silica, alumina and possibly iron compounds while the clay mineral kaolinite is only of minor importance. The amorphous components in terms of percentages on the clay size fraction of the soils studied are presented in Table 3. It can be seen that in terms of, for example, SiO_2 the variations are considerable as well as in terms of the other oxides.

The relations between lime concentration and time obtained during this study are presented in Fig. 1; for clarity of presentation the soils have been divided into four groups as shown in the Figure. These relations show that lime is consumed at a "fast" rate in the initial stages of the reaction, that is, up to five to seven days. From there on the rate of consumption of lime slows down drastically. This is an indication that the uptake of lime after the initial rapid period is very slow and limited in quantity. It is interesting to point out that the total consumption of lime between seven and twenty-eight days corresponds only to eight per cent of the lime consumed during the initial seven days. Therefore, it appears that the increases in strength of a soil-lime system beyond the seven day period cannot be explained in terms of generation of pozzolanic reaction products, since this should be manifested by greater amounts of lime consumption. From the data shown in Fig. 1 it is apparent that the major part of the reaction takes place within the initial seven days. Thus it is suggested here that the increase of strength shown by these soils beyond seven days is mainly due to the changes that take place in the structure of the cementitious products formed during the pozzolanic reaction, i.e. hydration and increases in crystallinity of the reaction products.

Table 1. Physical and Engineering Properties of the Soils used for the Investigation.

Soil Name	Soil Symbol	Textural Composition			Compaction and Strength Characteristics							Lime Reactivity
					Natural Soil			Lime Treated Soil				
		Sand 2mm-0.06mm	Silt 0.06-0.002mm	Clay < 0.002mm	max. density kg/m^3	moisture content for max. density %	unconfined compressive strength MN/m^2	max. density kg/m^3	moisture content for max. density %	unconfined compressive strength MN/m^2	Lime for max. U.C.S. %	
JOAO P.ACIMA	JPA	79	14	7	1911	10.35	0.29	1901	10.50	1.21	4.5	0.92
JOAO P. MEIO	JPM	60	12	28	2029	13.20	1.80	1944	16.20	5.39	6.0	3.59
JOAO P.ABAIXO	JPB	81	10	9	1861	7.70	0.10	1880	10.10	0.51	4.5	0.41
CUITE	CT	68	20	12	1983	13.85	0.41	1904	15.35	1.14	4.5	0.73
AREIA I	AI	50	20	30	1839	17.35	1.55	1812	18.55	2.05	4.5	0.50
AREIA II	AII	65	18	17	1982	14.20	0.51	1788	17.25	1.60	4.5	1.09
SOLANEA IA	SIA	66	18	16	1980	11.00	0.55	1918	14.70	0.59	4.5	0.04
SOLANEA IB	SIB	86	7	7	1873	4.80	0.08	1950	6.75	0.21	6.0	0.13
SOLANEA II	SII	56	20	24	1982	10.25	0.80	1936	11.35	2.96	4.5	2.16
NOVA FLOREST	NF	68	19	13	2061	13.30	0.50	1966	15.25	0.99	4.5	0.49
JUNCO I	JI	58	32	10	1939	10.10	0.52	1888	13.80	1.14	4.5	0.62
JUNCO II	JII	76	15	9	2079	12.35	0.20	2046	12.10	0.78	6.0	0.58
TEIXEIRA I	TI	48	30	22	1834	13.95	1.26	1782	15.40	3.22	6.0	1.96
TEIXEIRA II	TII	41	28	31	1753	16.60	1.28	1719	16.85	1.75	6.0	0.47
RECIFE	RC	60	10	30	1918	12.80	0.89	1880	14.40	2.59	6.0	1.70
USINA S.MARIA	USM	46	20	34	1742	17.30	1.85	1710	18.40	2.10	4.5	0.35
S.BANANEIRAS	SB	49	13	38	1916	12.50	1.92	1851	13.45	3.91	4.5	1.99
PENHA	PH	64	15	21	1828	14.30	1.11	1777	16.50	0.84	4.5	0.00
SAPE MARI	SM	58	19	23	1918	15.30	0.94	1865	17.10	2.49	6.0	1.55

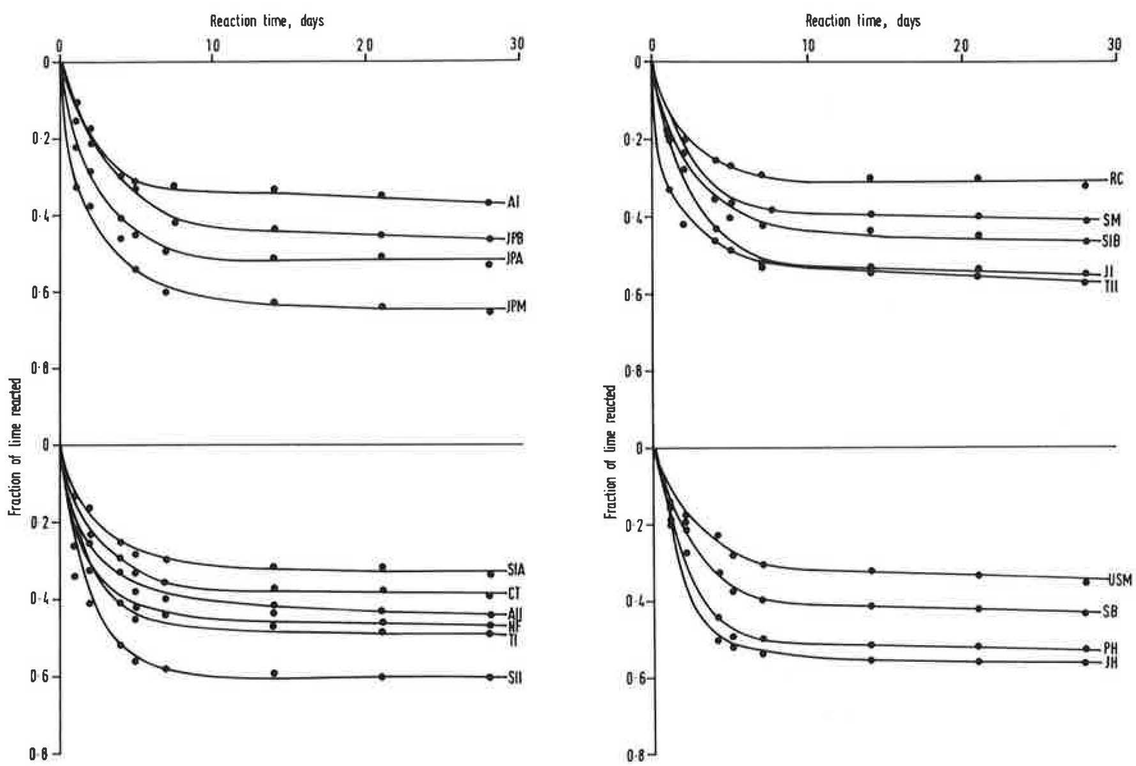
Table 2. Fraction of Lime Consumption at varying times for different fractions of two of the soils investigated.

Soil Name and Soil Symbol	Soil Fraction	Fraction of Lime reacted after				
		1 day	2 days	4 days	5 days	7 days
Joao P. Meio (JPM)	2mm-0.074mm	0.062	0.091	0.098	0.102	0.113
	0.074mm-0.002mm	0.124	0.140	0.162	0.168	0.198
	<0.002mm	0.298	0.367	0.461	0.517	0.589
Areia I (AI)	2mm-0.074mm	0.030	0.065	0.082	0.096	0.117
	0.074mm-0.002mm	0.102	0.115	0.146	0.172	0.196
	<0.002mm	0.092	0.147	0.289	0.314	0.342

Table 3. Amorphous Components of the Clay Size Fraction of the soils investigated.

Soil Name	Soil Symbol	Constituents extracted by 0.5N NaOH		
		$\%SiO_2$	$\%Al_2O_3$	$\%Fe_2O_3$
Joao P. Acima	JPA	2.81	3.10	0.46
Joao P. Meio	JPM	10.91	7.97	0.68
Joao P. Abaixo	JPB	6.71	6.11	2.51
Cuite	CT	7.84	4.12	0.80
Areia I	AI	8.11	6.95	1.07
Areia II	AII	9.69	5.89	0.62
Solanea IA	SIA	2.08	3.28	1.01
Solanea IB	SIB	4.13	3.97	0.79
Solanea II	SII	11.29	7.56	0.21
Nova Floresta	NF	8.31	7.02	0.43
Junco I	JI	5.02	6.11	0.78
Junco II	JII	6.69	4.07	2.05
Teixeira I	TI	8.12	6.98	0.30
Teixeira II	TII	4.68	4.02	0.87
Recife	RC	9.21	6.16	0.10
Usina S. Maria	USM	3.71	4.41	0.19
S. Banareiras	SB	8.19	6.84	0.28
Penha	PH	2.91	3.80	0.29
Sape Mari	SM	7.98	6.48	0.78

Figure 1. Relations between consumption of lime and time for the nineteen soils investigated.



Mechanism of reaction

The numerical values of the lime consumption vs. time relations were used to test the mathematical models discussed previously.

It was found that the experimental results do not follow the relations based on the equations which represent the dissolution theory, despite the fact that the experimental conditions were such as to give monodisperse suspensions for which the equations are intended to be valid. This finding coincides with the results of other investigators (10) which reported that the dissolution model was not applicable to pulverized fuel ashes.

The experimental results were found to approximate very closely to the values obtained with the equation of Jander based on the diffusion model. The deviation of the experimental results coincides with the time at which the rate of lime consumed changes, i.e. five days. The experimental points and the lines corresponding to the Jander equation are shown in Fig. 2. Fig. 3 shows the relation between reacted lime and time for the total period investigated for some of the soils. This figure highlights the fact that the experimental data follows the theoretical relation only for a limited time period, i.e. five days. The deviation beyond five days may be explained by the fact that one of the weaknesses of the Jander equation is its inability to incorporate the effect of the varying volume of reaction products being formed and therefore their influence on the reacting system. It is apparent then, that the increased volume of reaction products reduces the rate of diffusion as shown in Fig. 3.

Figure 2. Relations between the first term of Jander's equation F(x) and the time of reaction for some of the soils.

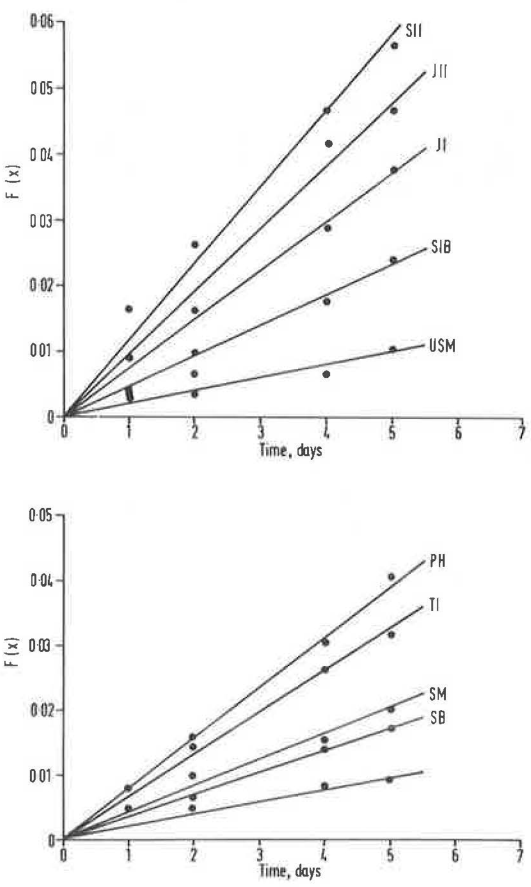
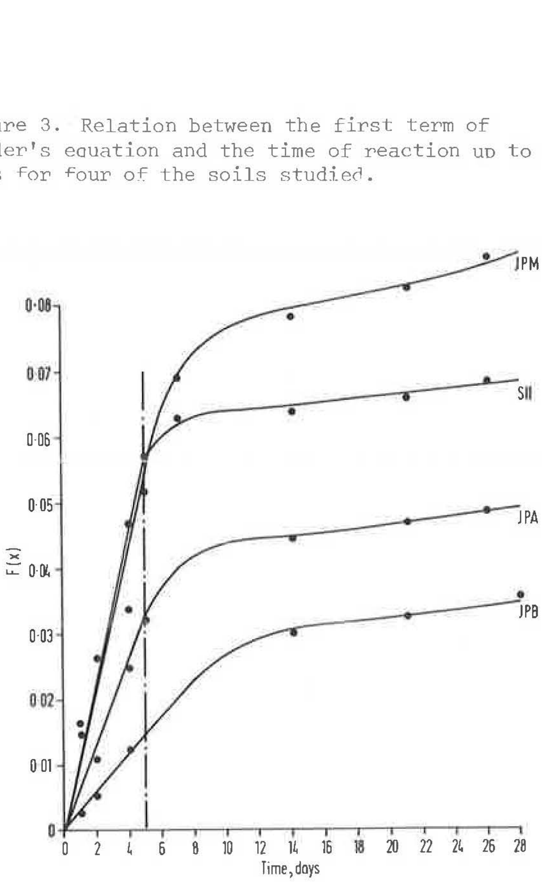


Figure 3. Relation between the first term of Jander's equation and the time of reaction up to 28 days for four of the soils studied.



It must be pointed out that the findings reported here may not represent other types of soils, since the major characteristic of the red tropical soils studied is their composition in terms of amorphous components, which react with the lime probably at a faster rate than crystalline clay minerals due to their high specific surface and their reactivity (21). Based on this fact, i.e. the high reactivity of amorphous components of the soil, the amelioration phenomena can be explained in terms of the rapid formation of cementitious products which cause changes in the engineering properties of the soils.

Figs. 4 and 5 are presented to show the micromorphology of the reaction products, they were obtained with a GEOL Scanning Electron Microscope and show very clearly massive plates of hydrated calcium silicates, which are the main hydration products of red tropical soils and lime (22). Fig. 5 which is a magnification of Fig. 4 shows furthermore evenly distributed amorphous components which are presumably the non crystalline reaction products.

Pozzolanic activity

From the findings of other investigators and the results presented here it is apparent that pozzolanic activity is directly related to the reaction rate and the quantity of the reacting components of the soil.

The slopes of the lines presented in Fig. 2 were used to determine the reaction rate constants for the nineteen soils and the quantity of reacting components was expressed by the amount of material smaller than 0.002 mm. It was decided that the clay size fraction should be the fraction more representative of the reacting components, though it is recognised that the most active components are really the amorphous materials in the clay size fraction.

Figure 4. Scanning Electron Micrograph, Magnification X3000 showing the micromorphology of C-S-H and scattered amorphous reaction products.

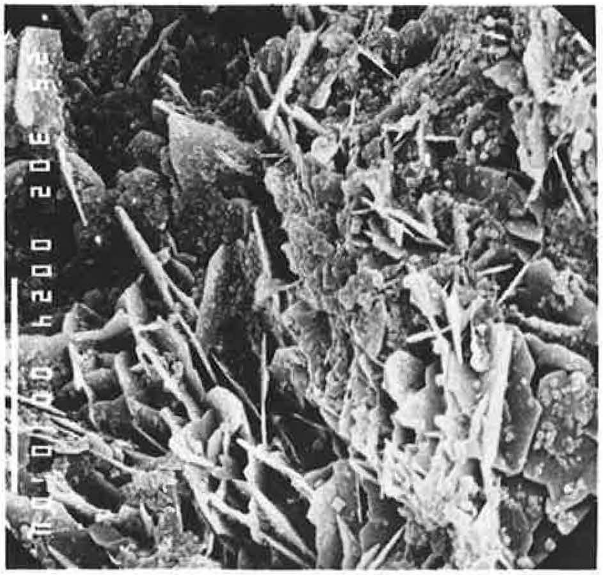
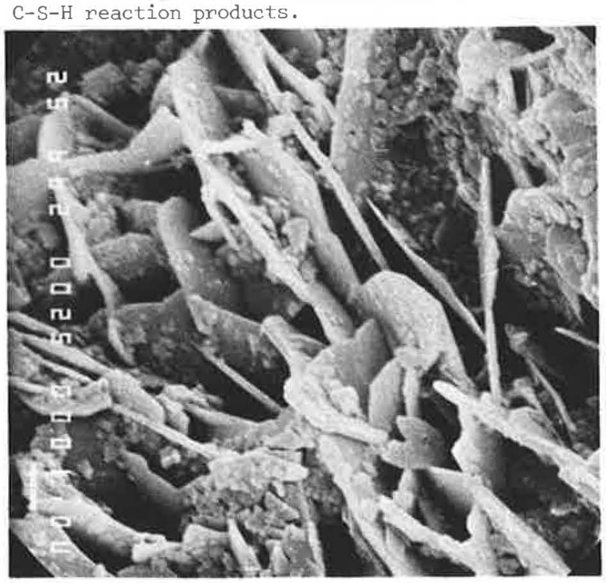


Figure 5. Detailed view of Fig. 4, Magnification X6600 showing massive plates characteristic of C-S-H reaction products.



It is proposed here that the pozzolanic activity of a red tropical soil should be expressed by an activity index which is related to the rate constant and the percentage of reacting compounds by the following equation:

$$A_i = K n^x$$

where A_i = activity index

K = rate constant

n = percentage of clay-size fraction

x = a constant dependent on the type of material

Although the introduction of the constant x may seem arbitrary, it is based on the assumption that different materials may have different types of reacting components within their clay size fractions. For the soils of this study it is proposed that x takes the value of 2.

The values of the rate constants and the activity indexes for the soils are presented in Table 4. It can be seen that the A_i values range from a value of 0.12×10^{-4} to a value of 4.08×10^{-4} giving really a wide range for classification purposes.

Table 4. Rate Constants and Activity Indexes for the Soils Investigated.

Soil Symbol	Rate Constant	Pozzolanic Activity Index $A_i \times 10^{-4}$
JPA	0.0035	0.172
JPM	0.0052	4.08
JPB	0.0015	0.12
CT	0.0016	0.291
AI	0.00124	1.116
AII	0.00234	0.791
SIA	0.00108	0.461
SIB	0.00245	0.120
SII	0.00543	3.13
NF	0.00279	0.472
JI	0.00384	0.384
JII	0.00471	0.567
TI	0.00340	1.646
TII	0.00384	3.689
RC	0.00118	1.062
USM	0.00108	1.248
SB	0.00174	2.512
PH	0.00411	1.812
SM	0.00202	1.07

The usefulness of the activity index should be proved against other accepted parameters, this was done in this paper by correlating the activity index with the unconfined compressive strength of the soil lime systems and also by investigating its relation with the well-known reactivity parameter of Thompson (17).

The relation of A_i vs. unconfined compressive strength is shown in Fig. 6. The equation obtained for this relation is:

$$UCS = 1.639\, A_i + 0.0135\, A_i^2 - 0.0276\, A_i^3 + 0.33$$

where UCS = unconfined compressive strength of the soil-lime system in MN/m^2.

A_i = activity index x 10^{-4} per day.

This equation was found to be statistically highly significant with a multiple R equal to 0.97327 at 95% confidence limit. The standard error of estimate is 0.3586.

The relation of the lime reactivity parameter of Thompson and the activity index proposed in this paper is shown in Fig. 7. This relation, unlike the one shown in Fig. 6, is a linear relation of the following form:

$$LR = 0.72\, A_i + 0.278$$

where LR = lime reactivity parameter in MN/m^2

A_i = activity index x 10^{-4} per day

This equation is statistically highly significant, with R equal to 0.88 at 77% confidence limit. The standard error of estimate is 0.4567.

Thus it can be seen that the activity index obtained by the use of a theoretical model is a valid representation of the pozzolanic activity of the red tropical soils investigated in this study.

Figure 6. Relation between activity index and the unconfined compressive strength of the red tropical soils.

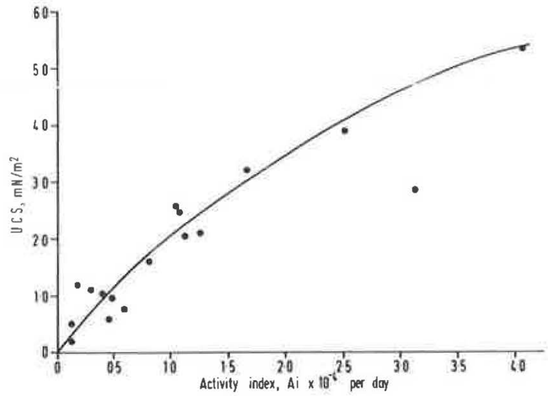
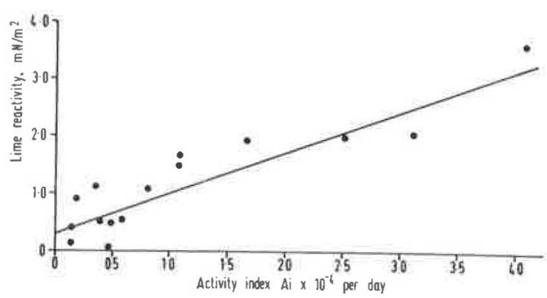


Figure 7. Relation between the activity index and the parameter of Thompson.



Conclusions

1. Formation of reaction products as represented by the consumption of lime takes place during the initial period of five to seven days, therefore the gains of strength beyond this period appear to be due mostly to the hydration and increased degree of crystallinity of the reaction products formed during the initial five to seven days period.

2. Under the experimental conditions of this study the reaction between the soil and the lime is mainly a diffusion controlled process obeying the equation of Jander for the initial period of five days.

3. Amelioration in red tropical soils appears to be a result of the initial rapid reaction between the amorphous soil minerals and lime which form products of initially very weak cementitious properties.

4. The activity index based on the diffusion model and the quantity of reacting compounds of the soil is statically related to the unconfined compressive strength of the soil-lime system.

5. A highly significant statistical linear correlation links the empirical reactivity parameter of Thompson and the proposed activity index. This finding is of significance since it allows the use of either of the two values for assessing the pozzolanic properties of these materials. The advantages of the A_i value are that it is a parameter based on a theoretical model and that it can be obtained faster than the reactivity value which requires not only time but great input of labour.

6. Since the activity index gives a wide range of values it may be used to classify red tropical soils. However, a limiting value of the Activity Index A_i which will separate the pozzolanic from the non-pozzolanic soils is very difficult to propose at this stage of knowledge. Thompson (17) has indicated that a reactivity value of 0.35 MN/m^2 (50 psi) should be this limit, while other researchers (7) have emphasised a minimum value of strength linked to a particular requirement.

Acknowledgements

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INNOVATIONS IN DESIGN AND CONSTRUCTION OF A LOW VOLUME LOW COST ROAD ON WINDBLOWN SANDS

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The design of a low cost pavement structure for low volume roads in some arid parts of Southern Africa is described. Only two available sources of material exist, namely calcrete and windblown sand, both of which vary greatly in quality. Several methods of possible improvement in strength are described and results are based both on laboratory as well as field measurements. Two pavement systems are considered in design namely a thin surfacing on a stabilized base and an asphaltic concrete made of calcrete/sand on in situ compacted sand. Both systems are susceptible to variance in material quality and this is considered in a method of pavement design. The statistical method of design in which variance in material properties is taken into account, considers a simplified two layer system which allows failure of the pavement area to be predicted. It is concluded that the improvement of the quality of available materials through selection and stabilization is necessary but the extent thereof depends on the variance and the mean strength of the product.

The design and construction of a highway as an indispensable part of the infrastructure of a developing country takes on a new dimension when a deficiency in quality material exists. The in situ material in parts of South West Africa mainly consists of an aeolian or windblown sand, often called a Kalahari-type sand, that compacts well under optimum moisture conditions but by its very nature is single-sized and hence very unstable in its uncompacted state. Thus the bearing capacity drops considerably under uncompacted saturated conditions.

The majority of vehicles which use these roads can be classified as heavy vehicles, i.e. bigger than 6 ton trucks, which take a heavy toll in riding quality on dirt roads. Although the designed for number of equivalent 80 kN (18 000 lb) axle loads is only of the order of 80 000 to 200 000 for a 20 year design life, it is economical to surface these roads in order to cut on maintenance costs.

Gravel material that can be used in pavement construction is calcrete with variable quality as far as material strength is concerned. The usable strength of calcrete is further impaired in the course of time by the presence of soluble salts. These salts lead to the destruction of any bituminous surfacing if not checked by preventive measures such as stabilization or proper construction procedures.

The use of sand/calcrete asphaltic concrete as surfacing is generally favoured since crushed stone has to be imported over long distances with the result that treatment or conventional asphaltic concrete as surfacing becomes too expensive. If high quality surfacing is used, it is implicit that it contributes to the structural strength of the pavement.

The final result is that two types of pavements can be considered namely a 75 mm (3 in) asphaltic concrete layer, or a 12 mm (½ in) asphaltic surface layer and 150 mm (6 in) stabilized calcrete base, on the well compacted windblown sand subgrade.

Characteristics of Materials

Considerable areas of the northwestern region of Southern Africa can be classified as arid or semi-arid and in situ materials mainly consist of an aeolian or windblown sand. The most common source of road building material is calcrete which primarily occurs in lower lying, poorly drained areas. Other than calcrete which consists of nodules, hardpans and strongly calcified sands, no other source of aggregate is economically available. Thus the engineer has to make the most of the meagre supply of good quality calcrete and sand to substitute for the commonly required high standard materials.

Windblown Sands

The single-sized sand virtually all passes the 2 mm (0,078 in) sieve and is non-plastic with a A-2-4 or A-3 classification. No more than 12% passes the 0,075 mm sieve which tends to cause low stability in the sand. Thus the CBR ranges from 7 to 45 with a mean value of 22 and standard deviation of 8 at a 95% Modified AASHTO density.

Apart from relatively low strength considerable variation of in situ density occurs. Field densities can be as low as 76% Modified AASHTO which implies a possibility of severe settlement under favourable conditions such as saturation and heavy loading (dynamic or dead loads). The CBR strength of the material at this density is 2 when fully saturated but has a CBR of 15 at field moisture conditions. Thus the obvious means of ensuring strength is by proper compaction and drainage.

Calcrete

Calcrete, a pedogenic material, is formed by the cementation or replacement of existing soils through the deposition of calcium carbonate from soil water (1). The material can be indentified as strongly calcified sands, hardpans or nodules, or a mixture of these.

Usually chunks of calcareous material up to a size of 75 mm (3 in) occurs in a matrix of fine material of which as much as 35% passes the 0,075 mm (No 200) sieve. The fine material can be classified as calcified sand and the plasticity index is generally lower than 13. The strength of the calcrete ranges from a CBR of 7 to 110 at 95% Modified AASHTO density. Borrow pit material, however, can be quarried with discretion to obtain a mean CBR strength of 58 and standard deviation 18 or under consistent control a mean CBR of 70 with standard deviation 12.

The biggest concern when using calcrete material in road layer work is the existence of soluble salts. Several investigators (2, 3, 4) have reported on this phenomenon and it was found that the most common deleterious salts are NaCl and Na_2SO_4. The actual damage is caused by the crystallization of salt between the bituminous surface seal and base course due to the evapotransporation of soil water. If the calcrete material is not chemically stabilized, attacks from sulphates may occur with a resultant break-up of the layer so that the limitation of both soluble salts as well as sulphates in calcrete are of prime importance.

Variations in calcrete quality makes it suspect as a natural base material, especially since soluble salts contribute to a degeneration of the engineering properties. Thus, mechanical or chemical improvement is essential in most cases, not only to avoid salt damage but also to increase mean strength.

Improvement of Materials

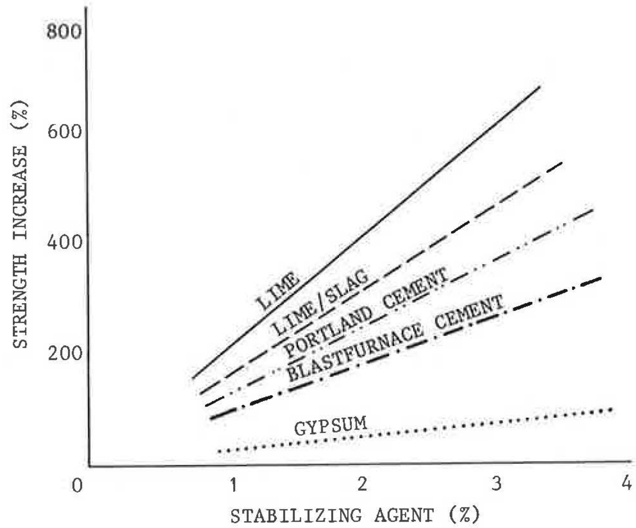
Several routes are available to the engineer for improving the engineering characteristics of the available materials. Amongst these are chemical alterations, mechanical stabilization and the addition of a suitable bituminous binder material. Although laboratory testing is necessary, the measuring of actual field performance by experimental sections is a more reliable way to evaluate the relative success of each method of stabilization on an equal basis. The laboratory methods of assessment is through testing using unconfined compression (UCS), Marshall, CBR and indirect tensile strength (ITS). Although the laboratory testing cannot fully take into account the effects of climate and traffic especially on chemical reactions, weathering and stability of the layer work, it nevertheless gives an indication of relative strength.

Chemical Improvement

Improvement of the bearing capacity of calcrete is possible by ion exchange and/or cementation through the addition of a chemical substance such as a calcium type road lime, milled blastfurnace slag, Portland cement or a combination of these. A beneficial byproduct of stabilization with lime in some calcretes is the prevention of eventual salt damage.

The improvement in strength can be measured in terms of CBR, UCS or ITS. Figure 1 gives an indication of the increase in strength through different stabilizing agents and stabilizer contents for a calcrete material from one borrow pit.

Figure 1. A change in strength with type and quantity of stabilizing agent.



Due to the great variation in calcrete quality, considerable scatter can be expected in the strengths of stabilized materials and it is therefore quite possible that the other types of stabilizing agents may perform better with other calcretes than the lime or lime/slagment as indicated. However, road lime is generally favoured because of better reaction with deleterious salts and slower gain in tensile strength in due course. The latter is important since block cracking of the stabilized layer must be curtailed in poor drained areas.

The change in soluble salt content with the addition of lime can best be illustrated by actual field results as indicated in table 1. Table 1 shows results of laboratory testing on samples taken from two sections of pavement in service. One section was built with natural calcrete material and the second section with exactly the same calcrete material but stabilized with 3% road lime. Samples were taken at 50 mm (2 in) intervals to a depth of 200 mm (8 in). The results show that a concentration of soluble salt can be found at the surface. It is also clear from the values in table 1 that lime treatment assists in reducing the soluble salt as well as the sulphate content.

Table 1. Soluble salt content* on a typical sample of natural and lime stabilized calcrete.

Depth of Sample	Unstabilized (% Salt)	3% Lime stabilized (% Salt)
0 - 50 mm	1,67	0,37
60 - 100 mm	0,96	0,13
100 - 150 mm	0,59	0,07
150 - 200 mm	0,25	0,10
Mean Sulphate content of layer	0,257	0,035

*Tested according to test method CSIR CA 21 (2).

No salt damage is experienced with the sand, therefore the chemical stabilizing of sand is aimed at improving strength. However, unless the quality of available calcrete is excessively poor, stabilized sand is not used as base material. Several reasons for this can be put forward :

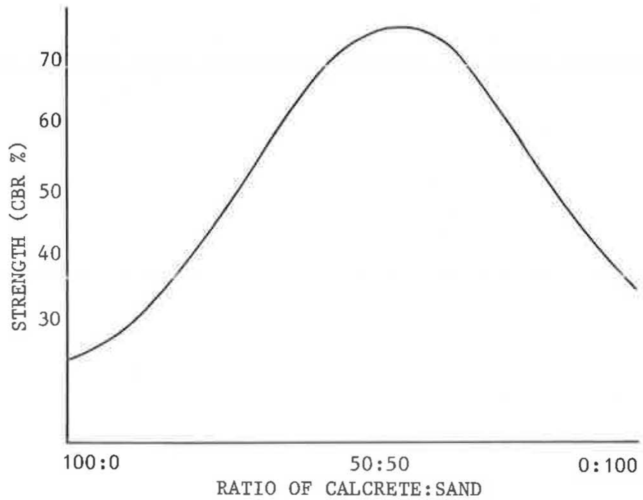
1. The pH of sand is low, of the order of 3,4 to 6,2. This will call for an increase in the quantity of alkaline stabilizing agents.
2. Only cementitious agents can be used for stabilizing since the sand is a non-cohesive, non-active material.
3. Shrinkage crack widths can be expected to increase as tensile strength increases. This reduces the structural performance of the layer.
4. Chemical stabilization of sand with Portland cement is more expensive since more agent is required than when calcrete is stabilized with lime for the same final cost benefit.

Thus, lime stabilized calcrete is favoured as a base under thin surfacing. The mean CBR of a good quality calcrete properly mixed with 1% lime, is 118 with a standard deviation of 22 compared to a mean CBR of 70 and standard deviation of 12 for the unstabilized calcrete.

Mechanical Improvement

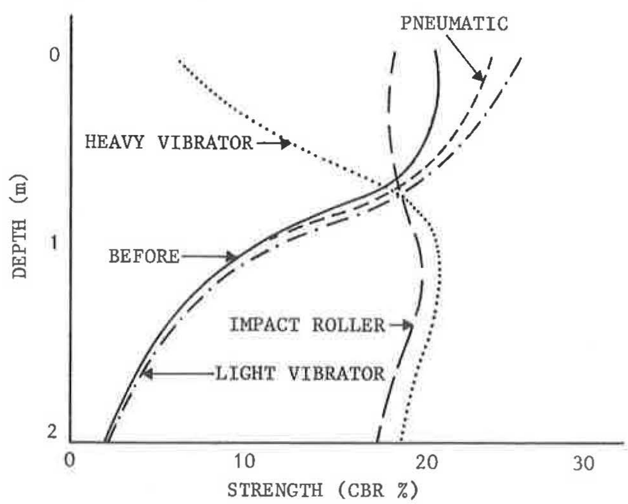
Two methods by which the strength of available materials can be improved mechanically are by mixing calcrete and sand, and by means of heavy compaction. The first method involves the improvement of the grading of calcrete by adding sand whereby density and mechanical interlock, and thus strength, is increased. The relative success of adding the sand to calcrete is shown in figure 2. In this particular case the maximum strength was reached at a 50 : 50 ratio of calcrete to sand. The increase in strength in terms of CBR is generally of the order of 25 per cent which implies an expected mean CBR of around 70 at 95% Modified AASHTO density for the mixture. This method of improvement, however, does not decrease variation in strength since it was found that at a fixed ratio of calcrete : sand, the increase in strength may vary between 0 and 50 per cent for materials from the same borrow pit. Chemical stabilization may thus be necessary to further increase strength to a level high enough for use as a base material.

Figure 2. Mechanical stabilization by mixing calcrete and sand.



In the evaluation of these strengths it is assumed that a high degree of compaction is attained. Keeping in mind that in situ densities of undisturbed material may in some cases be as low as 76% Modified AASHTO, improvement of strength through proper compaction is essential. This can be achieved by using heavy compactors directly on in situ material without necessarily scarifying or adding moisture to the material. Figure 3 indicates the increase in strength in terms of CBR by using a pneumatic, light vibratory, impact or square roller, and heavy vibrator with static weights of around 12 ton, 8 ton, 10 ton and 12 ton respectively. (The impact roller is a 10 ton roller with pentagonal drum which affects compaction through impact (5)). The relative success of the different rollers is very much dependent on the types of materials, moisture content and depth below surface at which measurements are taken. Generally it can be said of a windblown sand that the combination of a heavy vibratory followed by a light vibratory roller achieves the best results.

Figure 3. The effect of various roller on strength with depth.



The results of this combination are illustrated in table 2 which lists the change in CBR as well as density with depth on a windblown sand. The results in table 2 clearly indicate the benefit derived from heavy compaction in the sand. Not only has the mean strength increased but the variance of strength decreased over the compacted area because of more uniform densities.

Table 2. Average change in density and CBR of a windblown sand when compacted by a combination of heavy and light vibratory rollers.

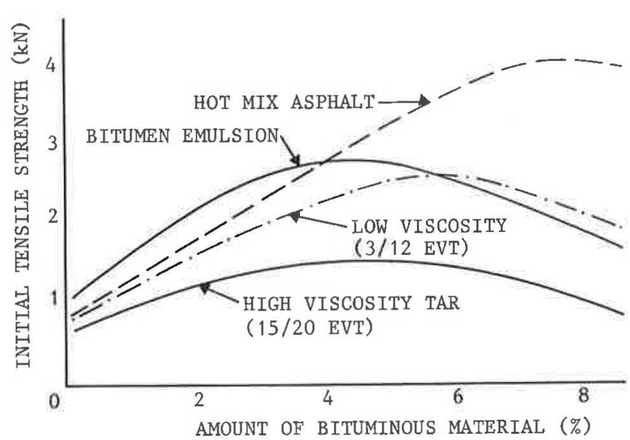
Depth (mm)	Density (kg/m^3) Before	After	CBR* Before	After
0 - 200	1 710	1 760	15	20
200 - 500	1 680	1 760	12	20
500 - 800	1 650	1 755	8	19
800 - 1 200	1 615	1 740	6	18
1 200 - 1 600	1 610	1 710	5	5

*CBR deduced from laboratory curves of CBR versus density.

Improvement by Asphaltic Materials

Several types of bituminous binders can be utilized as base stabilizers. Materials that have been used include different grades of tar, bitumen and bitumen emulsion. Figure 4 indicates the change in strength of sandy calcrete stabilized with different types of binders, as a function of binder content. The ability of the materials to be mixed properly appear to have a marked effect on the final strength. Thus the low viscosity tar (3/12 EVT) as well as the aniomic bitumen emulsion showed greater relative laboratory strength when compared with high viscosity tar (15/20 EVT). Also shown in figure 4 is the relative strength of a hot mix asphalt which contains 50 : 50 good quality calcrete and Kalahari sand mixed with 60/70 pen bitumen.

Figure 4. Variation in strength of a bituminous stabilized sandy calcrete.



Experimental sections of the different types of binder materials were constructed. These sections consisted of a 150 mm (6 in) stabilized layer without any seal and with a traffic load of about 10 twelve ton tandem axle trucks per day for 6 months. A panel rating of these sections is shown in table 3.

Table 3. Relative success* of bituminous stabilised sandy calcrete.

Time (months)	Property	Emulsion	3/12 EVT Tar	15/20 EVT Tar
3	Ravelling	4	4	3
	Deformation	3	3	3
	Cracks	5	5	5
12	Ravelling	2	2	3
	Deformation	3	3	3
	Cracks	3	2	3

*Scale : 5 excellent
4 good
3 average
2 poor
1 very poor

The low viscosity tar, showed inferior performance under actual field conditions since ravelling of the layer occurred within twelve months under traffic. The high viscosity tar was the best performer after 12 months as less ravelling and very little rutting occurred under traffic. This is somewhat contrary to laboratory strength results as indicated in figure 4. The reason for this difference may be found in the ease of mixing low viscosity tar with sandy calcrete but with a gradual loss of strength due to ageing.

Pavement Design

The foregoing background to the different types and varying quality of the available materials now leads to the question of how to implement the information into the most economical design/construction strategy. The strategy can be discussed under two headings namely the application of methods to improve the strength characteristics and the accommodation of variance in the strength of the materials.

Application of Methods of Improvement

The availability of material does not leave many options open to the engineer. However as an important first step the subgrade can be compacted. Not only does this improve strength but it also decreases variability which will find application in the statistical design method.

The improvement in base strength can be achieved by the various means of stabilization already discussed. The extent of improvement however depends on the strength required which is dependent on the type of surfacing or wearing course to be used.

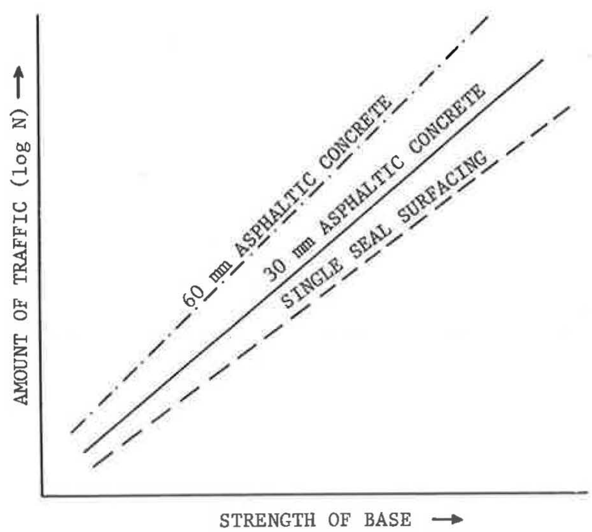
As far as the wearing course is concerned, an asphaltic concrete consisting of a high quality calcrete/sand mixture and bitumen gives the best performance. Some typical Marshall criteria are listed in Table 4. Although the voids are high, low air permeability is measured which implies a dense mix not prone to excessive ageing. Since the stability is reasonably high, the layer can also be utilized as a structural layer.

Table 4. Characteristics of a Typical Calcrete/sand Asphaltic Concrete

Binder Content (%)	Voids in Mix (%)	Stability (kN)	Flow (mm)	Air Permeability ($X\ 10^{-8}\ cm^2$)
7	16	3,5	2,0	0,9
8	14	3,7	1,5	0,7
9	12	3,3	1,3	0,5

The contribution made by a thickness of asphaltic concrete can conceptually be illustrated as in figure 5 where the thickness of asphalt required to withstand a certain amount of loading without overstressing the unstabilised sublayers is shown. Also shown in the figure is the alternative design to an asphaltic concrete layer namely the use of a single seal surfacing on top of an even stronger base. Stabilization may become inevitable unless a relatively salt free, high strength calcrete can be selected from the borrow pit areas and a relatively thick asphaltic concrete can be used as surfacing.

Figure 5. The conceptual relationship between base strength and thickness of surfacing.



where

z = the standard normal variable from which the area failed can be determined using statistical tables

n = the expected total number of load applications e.g. equivalent 80 kN (18 000 lbs) axle loads

N = the total number of load applications that can be tolerated on the pavement system, i.e. the mean number of load application designed for

$S^2_{\log n}$, $S^2_{\log N}$ = variance of log n and log N respectively

The value of n can be determined with the expected life, growth as well as present day traffic in mind. The value of N is determined from known pavement design procedures such as multi-layer and fatigue analysis and by using the mean strength of materials to be incorporated in the pavement structure. The value for $S^2_{\log n}$ is purely a function of the accuracy with which n can be estimated while $S^2_{\log N}$ has to be calculated.
The most convenient way to calculate the value of $S^2_{\log N}$ is by making use of Taylor's Theorem (7):

$$f(x,y) \simeq f(\bar{x},\bar{y}) + (x - \bar{x})\ f'\ (\bar{x}) -$$
$$(y - \bar{y})\ f'\ (\bar{y}) + \text{second order derivatives} \qquad (2)$$

where x,y = value of variables with means $\bar{x}$, $\bar{y}$

f' = first derivative of a function

Equation 2 can be expanded to the form for variance and can be written as follows (6):

$$\text{Variance } f(x,y) \simeq f^2(\bar{x}) \text{ variance } (x) + f^2(\bar{y}) \text{ variance } (y) + \text{second order derivatives} + \text{factors with covariance} \qquad (3)$$

The function (x,y) which simulates a function of N in this case, has to be derived from a mathematical equation for N which relates strain with fatigue characteristics :

Materials Variation and Design

The influence of variation in material strength on the ultimate performance of a pavement can be taken into account and the principle can be illustrated by the expression (6):

$$z = \frac{\log n - \log N}{\sqrt{S^2_{\log n} + S^2_{\log N}}} \qquad (1)$$

$$N = f\left(\frac{1}{\epsilon}\right)^t \qquad (4)$$

where

t = factor relating strain ϵ to number of loads to failure

ϵ = f (Modulus of elasticity E, stress S)

The principle can be illustrated by assuming an end loaded continuous beam on an elastic foundation. The simplified form for the stress in the top of the bottom layer can be written as (8):

$$\text{stress } S = f\left[\frac{P}{h^{0,6}}\sqrt[5]{\frac{E_2}{E_1}}\right] \qquad (5)$$

where

P = magnitude of load

h = thickness of top layer

E_1, E_2 = stiffness values for top and bottom layer

Assuming a constant relationship between stress and strain and by substituting equation 5 in 4 :

$$\log N = C - t \log\left[\frac{P}{Eh^{0,6}}\sqrt[5]{\frac{E_2}{E_1}}\right] \qquad (6)$$

where

C = constant

Differentiating equation 6 and deriving an equation in the format as shown in 3, a value for variance S^2 can be written for failure in the bottom layer.

$$S^2_{\log N} = (t \log e)^2\left[\left(\frac{S_p}{p}\right)^2 \quad \left(\frac{0,6S_h}{h}\right)^2 + \left(\frac{0,80\ S_E}{E_2}\right)^2 + \left(\frac{0,20\ S_E}{E_1}\right)^2\right] \qquad (7)$$

In all cases the values for material stiffness E are assumed to be related to strength and thus can be substituted by UCS, ITS or CBR, whatever is convenient, thus assuming a fixed relationship between these factors and E. Since the ratio of standard deviation to mean strength is employed, the actual relationship is of little importance in this calculation.

The fact that a two layered system is assumed may be a simplification but the main consideration is that a relatively thin wearing course, about 12 mm (0,5 in), with virtually no structural contribution will be used on a stabilized base which means that the stabilized layer is considered as the top layer for all practical purposes. The only question is whether the subgrade is uniform enough in depth to be considered as one layer. In the event of no stabilized base course being used, the sand asphalt, about 75 mm (3 in) thick, can be considered as the top of two layers on a uniform bottom layer, the subgrade material.

As an example the previously mentioned design of a 150 mm stabilized material with thin surfacing on an in situ compacted subgrade can be used. Using the mean and standard deviations for the subgrade and stabilized layer as reported on previously and a standard deviation of 25 mm for the thickness of the base, a value of 0,37 is derived at for the variance of log N by using equation 7. Assuming no variance in determining log n, the value $S^2_{\log n} + S^2_{\log N}$ in this case is calculated to be 0,37. If 80 000 axle loads i.e. log n is equal to 4,90, are expected in the life of the pavement, the value of z becomes 0,99 since the value of N is approximately 320 000 from multi layer and subgrade strain analysis based on mean material characteristics. This implies 16% of the area will have failed in terms of rutting and unevenness after the design life of 80 000 axle loads have expired.

Conclusions

The use of relatively low quality material in low volume roads is becoming increasingly necessary, especially since the cost has to be kept to a minimum. The quality of these materials can be increased substantially by different techniques of which mechanical and chemical procedures show the biggest potential for base materials. Since relatively thin layers of wearing course are used on stabilized bases the incidence of the reflection of stabilized cracking increases. Thus small amounts of stabilizing agents are normally added whereby strength is increased to the required level. In the event of a calcrete/sand asphalt being used as wearing course, an increase in thickness of this layer may decrease the required strength of the subbase and only mechanical stabilization may be required.

The single most important aspect in the design and construction of a low volume low cost road is the variation in material quality. It can be mathematically shown that a decrease in the variance of log N, the designed for number of axles loads, decreases the possibility of failure provided mean strength remains constant. This implies that good quality control as well as sound construction practice is most important especially where calcrete base material is chemically stabilized. On the other hand the mean strength may be increased and greater variance be accepted with the same result. This philosophy of increasing mean strength can be followed especially where more difficult processes of construction such as bituminous stabilization is considered and also where reflection cracking from chemical stabilization is of no concern. One important aspect that needs consideration is the possibility of increasing the mean thickness of the top layer. Not only does this action increase the expected life log N, but also decreases the variance of log N with a resultant decrease in failure area.

Finally, a few important points need to be stressed :

1. Mechanical stabilization especially in the form of deep compaction by heavy equipment is beneficial to improve the strength characteristics of windblown sands.
2. Provided the materials are selected carefully, a mixture of sand and calcrete produces a material that can be used as base as well as aggregate for asphaltic concrete.
3. Unless the calcrete has a low soluble salt content, chemical stabilization is almost certainly a requirement in order to obtain a material of base quality under a thin surfacing.
4. Bituminous materials can be used with success in stabilizing the available borrow pit material provided the mixing-in process is of high standard or the mean strength is sufficiently high.

5. Asphaltic concrete manufactured from calcrete/ sand mixtures can be used both as a wearing course and as a base.
6. The variation in quality of material is as important as the mean strength in designing a successful pavement structure. At the present time the knowledge is available permitting the application of the variance and mean of pavement characteristics, such as strength and thickness, in design and construction control.

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FOREST SERVICE EXPERIENCE WITH IN-PLACE REDUCTION OF OVERSIZED ROCKS IN UNSURFACED ROADS

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The Rocky Mountain Region of the USDA, Forest Service has about 11,100 km (6,900 mi.) of unsurfaced roads in a system totaling 37,400 km (23,225 mi.). Surface maintenance of unsurfaced roads in rocky soils is difficult, yet gravel surfaces or pavements often are not economically justified. A Browning RB-4 traveling hammermill crusher has been evaluated to reduce oversize rocks in place. In several trials in 1976 and 1977, the cost averaged $1,708 per km or $2,750 per mile. Different applications were tried and the most efficient determined. Results were generally satisfactory, but a sizable crew and fleet of supporting equipment are essential. This is not a routine maintenance tool, but rather a heavy maintenance tool approaching reconstruction. Some equipment modifications are suggested.

The Rocky Mountain Region of the USDA, Forest Service includes the States of Colorado, South Dakota, Nebraska, Kansas, and Wyoming east of the Continental Divide. In this area, there are 15 National Forests and 3 National Grasslands.

The Regional road system includes about 37,400 km (23,225 mi.) of roads of various standards. About 716 km (445 mi.) are paved, and these are primarily in recreation areas. The remainder includes about 4,280 km (2,660 mi.) of aggregate surfaced roads, 11,100 km (6,900 mi.) of unsurfaced roads, and about 21,300 km (13,230 mi.) of primitive roads. Unsurfaced roads may handle from as few as two or three vehicles per day to more than a hundred vehicles on peak days in some recreation areas. Most of the unsurfaced roads are old, but some are still built where the soils will support the expected traffic without undue environmental damage. Forest Service roads primarily serve timber harvest, recreation uses, and land management activities. The system does not include Federal, State, and local roads with which it is connected.

Construction specifications have changed over the years, but typically it has been required that all rocks larger than 10 cm (4 in.) in size be removed from the top 15 cm (6 in.) of the subgrade. Below that level, rocks up to 61 cm (24 in.) in diameter may be placed in embankments. However, surface degradation, frost action, abrasion, erosion, or poor quality maintenance eventually bring large rocks to the road surface. The road becomes rough, unpleasant to drive, hard on vehicles, and sometimes unsafe. Users properly criticize the owning agency.

The standard technique for handling the problem is to remove the rocks a few at a time as they are pulled out of the road in normal maintenance. In time, this removes a substantial volume of material and results in the profile grade being lowered, sometimes by more than 30 cm (1 ft.). The boulders accumulate along the roadside. In relatively flat terrain, they sometimes make an unsightly row of rocks beside the road.

Where a road is located on terrain such as a glacial moraine where a large number of boulders are present, the problem is very difficult. There are too many rocks to remove without replacing the lost volume, the cost is very high, and the resulting road is usually poor. On an old road, it is difficult to place an aggregate surface over an uneven, rocky subgrade, and even more difficult to keep the aggregate in place. The roads usually do not justify a bituminous pavement system due to low traffic volumes.

In-place treatment of such rocky surfaces was a logical idea and was not new. It was widely practiced in the past by counties, but has largely been abandoned as county systems were upgraded to pavements. The Forest Service decided to try the technique and in research reported in 1974 (1), the San Dimas Equipment Development Center examined two systems. One, a traveling jaw crusher, would have required the material to be dug up, lifted into the crusher, and spread behind the crusher. The mobile jaw crusher available at that time would handle rocks up to 30 cm (12 in.) in diameter. Moist or cohesive materials would have caused problems. According to the San Dimas research prediction, the cost was about 10 percent greater than that of the mobile hammermill system.

Machine

The traveling hammermill type machine was selected by San Dimas for further evaluation, and their 1974 report gave a favorable recommendation

Figures 1 and 2. Browning RB-4 parked without towing unit. Hammer chamber is under sloping cover on front view. Note many hand-operated adjustment cranks, turnbuckles, levers, etc., which have been eliminated in newer equipment. Rubber skirts control flying fragments.

Front view.

Rear view.

on the basis of controlled tests and computer models. Tests, however, are not the same as experience in day-to-day production work. The Rocky Mountain Region decided to buy a machine and evaluate it in routine use. A used Browning Model RB-4 "Rock Buster" was purchased from an area contractor and overhauled before being placed in service. It was somewhat smaller than the Pettibone Model P-500 used in the San Dimas test, but otherwise similar (Figures 1 and 2).

The machine obtained is quite old and does not represent current equipment technology. It was built in about 1952. It is 2.44 m (8 ft.) wide, 5.00 m (16 ft. 5 in.) long, 2.75 m (9 ft.) high, and weighs 5,443 kg (12,000 lbs.). The engine is an International Model UD-14 diesel rated at 180 hp. The engine drives the hammers through a multiple V-belt. The machine rides on two wheels which are adjustable for height, and is towed by a tractor. The only controls are the clutch and throttle, which are operated by ropes to the tractor. All other adjustments are manual. The arrangement is awkward but workable. There are some problems with parts availability resulting from the age of the machine.

Manufacturer's literature indicated that a D-4 class tractor could pull the crusher. However, both San Dimas and Regional crews found that such a small tractor was marginal in power and lacked underclearance to straddle the windrow. A D-6 class tractor with power shift transmission was preferable. Underclearance was satisfactory, and the power shift transmission provided smooth operation at very slow forward speeds. A side benefit was noted in that the larger tractor crushes some rocks under its tracks, assisting the operation.

Crushing is done by 18 free-swinging hammers which are mounted on four pivot pins arranged parallel to a central driveshaft (Figures 3, 4, and 5). The hammer speed is 1,000 rpm, and with new hammers the total assembly has a diameter of 96 cm (38 in.). The rated capacity is 115 to 153 m^3 (150 to 200 cu. yds.) per hour. The width of the impact area is 1.22 m (4 ft.), and the loose depth of the processed material behind the machine is 20 cm (8 in.). This is a maximum capacity of 0.07 m^3 (2.67 cu. ft.) per 30 cm (lineal foot) of travel, or about 0.76 m^3 (1 cu. yd.) per 3 m (10 ft.) of travel.

Objectives

In the uses described here, performed in 1976 and 1977, the following objectives were considered:

1. Find the most efficient application technique.
2. Determine the optimum crew and support equipment arrangements.
3. Determine costs of routine production operations.
4. Determine lives and costs of several different hammer alloys and compare, if possible, with rock type being processed.
5. Suggest equipment modifications if any are indicated.
6. Determine crew training needs and see if the machine can be effective with minimal training.

Trials

The first trial, while getting the crew acquainted with the equipment, was simply to eliminate loose rocks pulled out of a road by previous normal blading operations. Rocks were pulled out of the ditches, etc., and brought onto the road surface, which was not disturbed. A single pass of the machine crushed and scattered them. A layer of gravel about one pebble thick was produced over the road width, and within a week or two most of this had been whipped off by traffic (Figure 6). Rock that might otherwise have been loaded and trucked to a dump site was

Figure 3. Inside hammer chamber of Browning RB-4. In operation, hammers swing free on shafts. Note different wear patterns depending on position. These hammers have a few hours of life remaining. Replacement hammers will be placed where blank spaces occur in this set, enabling two sets of hammers to be used on one set of pins.

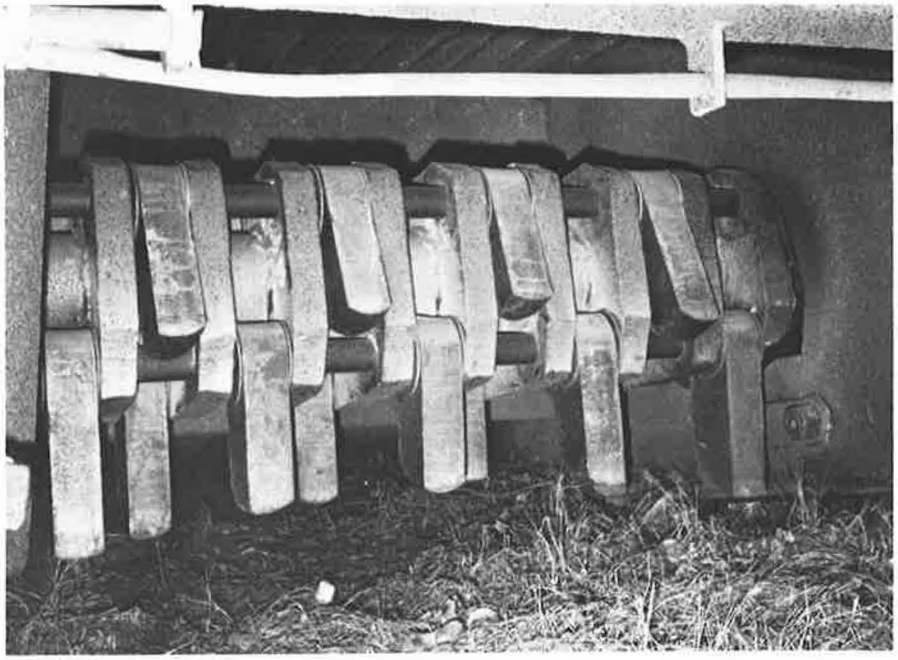


Figure 4. New hammer for the Browning RB-4. Dimensions 40 cm (16 in.) long, 11.1 cm (4-3/8 in.) wide, 6.3 cm (2-1/2 in.) thick; Weight 24.5 kg (54 lbs.).

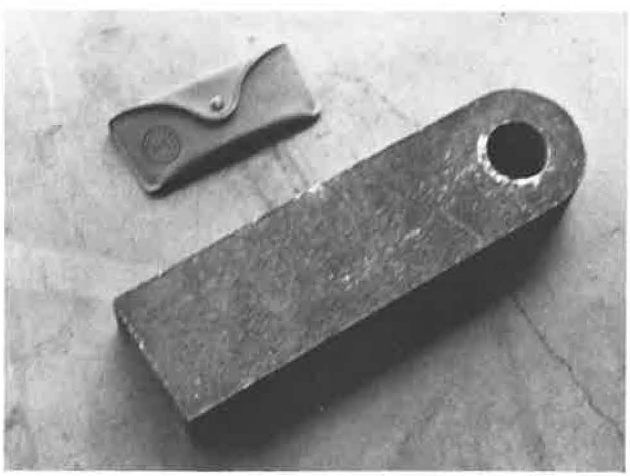


Figure 5. Worn hammers from Browning RB-4. Hammer on left is about 25 cm (10 in.) long.

eliminated. However, there was no lasting benefit to the road. It was a convenient way to get rid of these rocks, but was judged to be an inefficient use of the equipment.

The next experiment was to scarify the road surface to a depth of 15 cm (6 in.) to 30 cm (1 ft.), windrow and crush the material, and relay it without compaction. Equipment included a D-7 tractor with three ripper teeth, one grader, and a water truck. Total progress was about 0.8 km (1/2 mi.) per day, and the cost was about $1,875 per km ($3,000 per mi.). The material was weathered granitic rock and residual soil.

The road was in very good condition when completed, but potholes began to develop within 3 months. This potholing may have been due to the nature of the soils, but other roads where compaction was used have not developed much potholing. Thorough compaction after processing was highly beneficial.

The equipment array was not efficient with respect to the crusher. The ripper, blade, and other equipment worked steadily for the full day to prepare and respread the surface, but the crusher was in operation only a little more than 2 hours. Also, it was observed that the three-tooth ripper was not as effective as a five-tooth, short-shank ripper

Figure 6. Windrow before and after processing by the RB-4 and the same spot about 3 weeks later. This windrow is smaller than the maximum which can be handled, but is otherwise typical. This section was not ripped.

a.

b.

c.

would have been, because the ripper teeth were too widely spaced.

Optimum Equipment Array

The most efficient operation and equipment array was developed by trial and error and includes:

1 Crusher with towing tractor.
1 Bulldozer, D-7 class, with multitooth, short-shank ripper.
2 Motor graders, Caterpillar No. 12 class.
1 Water truck, 19 m^3 (5,000 gal.) capacity.
1 Rock rake.
1 Dump truck.
1 Wheel type tractor-backhoe with loader and hydraulic rock splitter attachments.
1 Mechanic's truck, well equipped.
1 Welder.
1 Compactor, usually pneumatic tired.
1 Rock drill, either air- or gasoline-driven.
2 Pickups or personnel transports.
1 Brush chipper (optional).

The crew included six operators, eight laborers, one mechanic-welder, and two flagmen, if needed.

Optimum Technique

The bulldozer-ripper and one grader prepare the road by ripping the surface. Several passes with the ripper are necessary to fully loosen the surface, and a single-lane road usually must be totally closed during operations. The blade makes a few trips to work the rock to the surface and makes a windrow.

Windrow construction is an important step. The windrow must be of the proper volume for the machine. Rocks larger than 35 cm to 41 cm (14 to 16 in.) must be removed from the windrow. There must be some fine soil for efficient crushing and to confine flying rocks, but too much fine soil slows the process. The operation depends heavily on the blademan's judgment and skill.

One to three passes by the crusher are usually necessary to crush the rock in the windrow. The blade then reworks the windrow and brings out any rocks missed in the first passes. A new windrow is built and processed, and the operation is continued until the entire volume is processed. For the usual one-lane road, this requires about three full windrows. In a dry climate, almost continuous watering is required to control dust. During crushing, the bulldozer-ripper and one grader are starting the next section.

After crushing, the second grader, rock rake, compactor, and water truck work to spread, shape, and compact the surface. If this operation exposes any uncrushed rock, a final pass with the crusher may be worthwhile before final compaction.

The tractor-backhoe is an auxiliary machine used for digging out boulders which the ripper cannot handle, as well as clearing culverts and other incidental work. The hydraulic rock splitter reduces large boulders or rock outcrops. Blasting is inefficient for this kind of work. The air compressor and rock drill are required with some types of hydraulic splitters.

The dump truck, brush chipper, and other small equipment is used for incidental work.

The mechanic-welder and his equipment are indispensable with this amount of equipment operating at considerable distance from town.

The overall operation can be speeded by the use of two crushers. Windrow processing is faster, and lost time due to crusher breakdowns and hammer changes is greatly reduced.

It is obvious that this process is a heavy maintenance operation approaching reconstruction. It was originally hoped that the crusher could be used in routine surface maintenance, but such is not the case.

Materials

The Browning machine has been used in many different soil and rock types, including fresh and weathered granite, glacial moraine, volcanic extrusives of several types, limestone, quartzite, etc. It has performed about equally well with all.

Initially, an attempt was started to relate rock type to hammer life and cost. A number of samples were taken before and after one, two, and sometimes three passes of the machine. Gradation tests were run, and Los Angeles abrasion tests and some visual rock classifications by particle counts were made, but this effort was eventually dropped because of lack of time and funds to pursue it.

Some observations based on very limited data are as follows:

1. The crusher effectively reduces rocks up to about 41 cm (16 in.) in size to the 5 cm (2 in.) sieve size and smaller. Contract specifications could be based on this performance.
2. There was conflicting data on the material passing the No. 4 sieve. In some tests there was no significant change, but other tests indicated changes even in the material passing the No. 200 sieve and in the Plasticity Index. Accurate comparison sampling was almost impossible, and this may be the reason for conflicting data. The question was not pursued to a solution. No important changes in the fine soil were expected.
3. There appears to be some correlation between crusher performance, hammer wear, and the Los Angeles abrasion characteristics of the principal rock type present, but no correlation with the sulfate soundness loss. Tests included rocks having L. A. losses from 21 to 70 percent. Visual identification of the rock type and weathering condition might be as informative as the L. A. abrasion test. Rock toughness and abrasiveness are important.
4. The manufacturer states that rocks up to 61 cm (24 in.) can be crushed. Experience shows that 41 cm (16 in.) is the practical limit without risking damage to the machine. The pivot pins which carry the hammers are very difficult to remove if they are bent.

Equipment Life and Modifications

Several different alloy steels were tried for the hammers with the following life spans reported:

1. Mild steel, grade 1020, average life 19 hours. These cost about $16 each.
2. Mild steel, grade 1020, faced with welding rods, Eutectrode No. 4 and Chromecarb N600G, average life less than 24 hours in weathered granite and metamorphic rocks. The cost and life of these vary with the amount of hardfacing used.
3. Cast manganese steel, life 24 to 65 hours. This was the best, though the hammers cost $31 each in 1977. The reduction in time lost to hammer changes more than compensated for the initial cost. A full set of hammers and pins cost about $1,000 in 1978.

Several different steels for the hammer support pins were also tried. Cold rolled steel was unsatisfactory. The best to date has been grade C1144 steel, which usually lasted through two sets of hammers by staggering the hammer location (Figure 3).

A number of modifications were made in the machine. Many corrected peculiarities of the rig, and fell in the general category of strengthening or toughening it to cope with the strong vibrations and very severe operating conditions. The most important was to weld heavy reinforcement to the underside of the crushing chamber to absorb the impact of rocks thrown up by the hammer. The material used was buildup bar for crawler track shoes, which worked very well.

A second major category of modifications was related to speeding the change of hammers and pins. When the Browning machine was first received, it took in excess of 4 hours for two men to replace the hammers. With some modifications, it is possible to do it in less than 2 hours unless something is bent or broken.

Most of the mechanical problems appear to have been solved in the newer Pettibone machine.

Operating Problems

The most serious problems are not usually directly concerned with operation. They involve coordination and control of the work.

1. Traffic control. During this operation it is very hard to keep a two-lane road open to traffic, even intermittently, and on a one-lane road, total closure during the workday is highly desirable. On a system of one-lane roads, detours are often difficult to find, and work must be scheduled during minimum traffic seasons.

2. Coordination. Only one or two of the Forests in this Region have either the men or equipment to handle the full operation. It is often necessary to combine crews from two or more Forests with counties or other agencies. This is very difficult to coordinate, especially since crews and funds must be diverted from other work which may be equally important. One project in 1978 involved crews and equipment from the Forest Service, a county, and a contractor. Management has been extremely difficult, and this kind of arrangement should be avoided.

3. Crew and equipment. Since the machine is moved from one Forest to another, a new crew must be broken in every time. This always results in some lost time, inefficiency, and excessive wear on the machine until the new crew gets accustomed to it. A full-time crew chief traveling with the rig helps, but a permanent crew with its own supporting equipment would be the best arrangement. Separate funding would also help avoid many of the problems and permit more efficient scheduling.

Costs

The precise operating cost of the Browning RB-4 was difficult to isolate because a number of variables such as crew size, hammer type, soil and rock type, etc., were being tested. It was intended that the entire trial be handled as nearly as possible in a "real world" environment. Thus, all personnel were regular construction and maintenance foremen, operators, and laborers. Cost accounting was no more elaborate than in routine operations.

A total of about 59 one-lane km (36.8 one-lane mi.) of road were processed in 1976 and 1977. These included many rock conditions and soil types. Several crews were used, and the work was geographically scattered throughout the Region.

The costs ranged from a high of $2,086 per km ($3,338 per mi.) to a low of $1,580 per km ($2,462 per mi.). The overall average for all work was $1,708 per km ($2,750 per mi.). In general, the more costly projects were those where a small crew and minimum outfit of supporting equipment were used.

By comparison, the cost of 10.2 cm (4 in.) of crushed aggregate, in place, would average between $1,875 and $3,125 per km ($3,000 to $5,000 per mi.). The additional cost of preparing an old subgrade to receive the gravel would average more than $625 per km ($1,000 per mi.). There may be rare situations where gravel would compete with the Browning machine.

Other Trial Applications

One other minor test was conducted on a section of old bituminous pavement that was so badly deteriorated that it was no longer maintainable. It was decided that the road would be more economical with a gravel surface. The pavement was scarified, windrowed, and processed with the Browning machine in the manner previously described. The product was a very satisfactory gravel surface. The alternative would have been to remove the old pavement and replace it with new crushed aggregate.

In this case the old asphalt was very brittle. While some relatively fresh patches crushed satisfactorily, it is not known whether the process would work with bituminous material which is not generally brittle.

The machine might also be useful in new construction to finish the top of a rocky subgrade in lieu of removing all rock greater than that allowed by specification. This idea has not been tested.

Summary and Conclusion

The original objectives were generally fulfilled, and the findings can be summarized as follows:

1. This is a useful technique, though not inexpensive. It is not a routine maintenance tool, but rather a tool for heavy maintenance approaching complete reconstruction. It requires a considerable support force and trained manpower for efficient operation. The overall average of costs in 1976 and 1977 was $1,708 per one-lane km or $2,750 per one-lane mile. A more efficient machine might be much cheaper to operate. Crushed gravel surfacing may occasionally be competitive.

2. The most effective hammers are high alloy types which have higher first costs but last longer, reducing time lost in changes. There appears to be some correlation between soil or rock type and equipment behavior, but this work was not conclusive.

3. Improvements can be made to any available machine, generally toward strengthening it and simplifying maintenance and hammer replacement.

4. Trained crews are essential to smooth operation. It takes about 2 weeks for a crew to "shake out" and begin to work well together.

5. The major problems are with coordination between agencies, financing, etc., rather than in operation, which is fairly straightforward once crews are well trained.

6. Other practical uses for the equipment probably will appear.
7. The trial was successful. The technique will continue to be used by the Region.

In 1978, the Pettibone P-500 machine originally used in the San Dimas research was obtained and used in place of the Browning RB-4, which was sidelined due to parts shortages. Early indications suggested a major reduction in costs due to greater speed and easier hammer changes, but full information was not available at this writing.

Reference

1. R. M. Gallup and C. S. Airall. Treatment of Oversized Rock on Non-Surfaced Roads. USDA, Forest Service, San Dimas Equipment Development Center, 1974, 54 pp.

AN INTEGRATED NATIONWIDE RURAL ROAD SYSTEM FOR THE GAMBIA

Paul E. Conrad and John G. Schoon, Wilbur Smith and Associates

The rural road system in The Gambia, West Africa, comprises over 2300 kilometers of paved, gravel, and earth roads. These connect rural communities with each other, to riverside staging points, to the larger towns and cities, and to produce storage and transshipment depots. The role of the road system is considered in regard to these functions and as related to needs for future rural development consistent with national goals and objectives. Data based upon recent studies in The Gambia are presented, particularly those which address future agricultural development potentials and road integration with river linkages. The categories of primary, secondary and principal feeder roads are examined from the viewpoint of current function, traffic, and existing deficiencies. Future highway needs based upon optimum use of the River Gambia and the road network for transporting a variety of import and export commodities are described and a tentative road investment program is proposed. Guidelines are then outlined to assist in the geometric and structural design of future highways in The Gambia and a review is made of material types and availability for future use.

Location

Located on the west coast of Africa, The Republic of The Gambia covers an area of 10,400 square kilometers, and extends eastward from the mouth of the Gambia River a distance of approximately 330 kilometers inland. At its widest point, the country is approximately 40 kilometers wide and is surrounded, except for the coastal strip, by Senegal.

The Existing Road Network

The two basic forms of transportation in The Gambia are river and road. Due to the elongated east-west orientation of the country, surrounding low-lying topography and presence of the navigable River Gambia extending nearly the entire length of the country, the river was historically the preferred means of transporting persons and goods. The later development of roads has increased economic development and agricultural production. Need for improved communications and general inland travel was therefore a secondary development in the transportation scene.

In general, travel by road has proved to be more desirable for persons and small cargo loads and for certain trips which do not involve movement between opposite sides of the River Gambia. Bulk cargo movements along the river and passenger and small cargo movements involving cross-river movements remote from ferry crossings, or longer trips where time is less important, are almost exclusively accommodated by use of river craft.

Road Classification

Existing roads throughout The Gambia are shown in Figure 1, indicating the Public Works Department's functional classification system of primary, secondary, and feeder roads. It can be seen from this that many roads lead directly to and from riverside points--used extensively as river-road goods transfer nodes.

Primary roads are defined as those connecting the main centers of activity, and form the major, continuous, all-weather lines of communication. These roads carry most of the rural traffic in The Gambia. Some sections have been bitumenized and the remainder are gravel roads. The alignments and widths (generally 6.7 meters) are such as to allow a free flow of two-way traffic, except at some bridges and at ferries.

Secondary roads are all-weather roads connecting a particular region or locality to the primary network. They are essentially gravel roads of somewhat lower standard than the primary roads and do not form a continuous network.

The largest group of secondary roads serves the coastal area and the southern border of The Gambia. Others are isolated connectors to the primary roads.

Feeder road is the term used to describe the multiplicity of routes used primarily for access between villages, and to transport crops to buying stations, riverside depots and processing centers. They are basically unformed roads, with little or no provision for drainage. They range from well-defined (and sometimes gravelled) tracks for vehicular use to footpaths or cart tracks. Some of these roads, located on higher, stony or free draining land, are usable throughout the year, but most feeder roads provide reliable access during the rainy season only for four-wheel drive vehicles.

Of the national total of approximately 2,360

Figure 1. Existing rural road network, The Gambia.

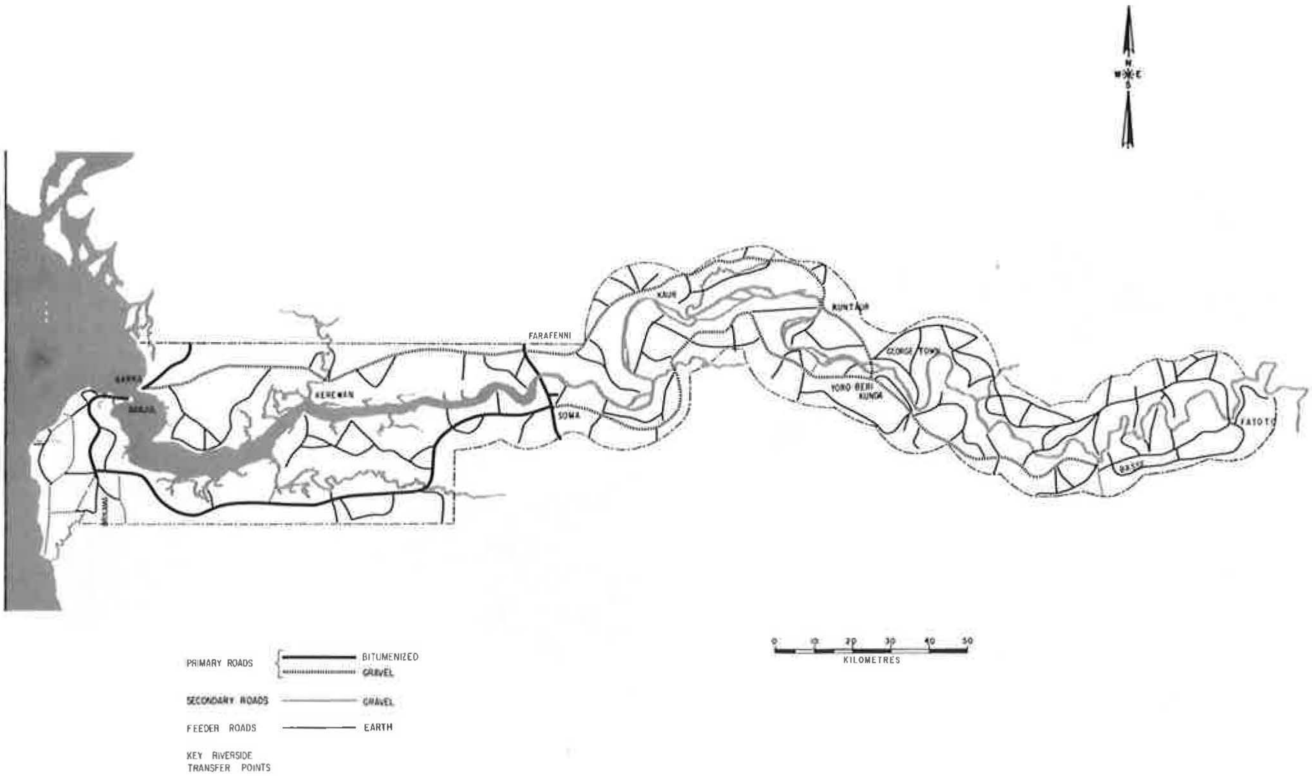


Figure 2. PWD road classifications.

Primary road, gravel surface.

Feeder road, earth surface.

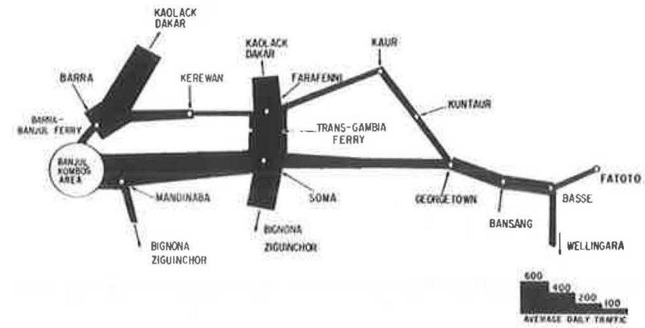
kilometers, the greatest length, 1,555 kilometers, or 66 percent, are major feeder roads, while primary and secondary roads comprise 28 and 6 percent of the total, respectively. Examples of each classification category are shown in Figure 2.

Traffic Characteristics

Sample traffic counts were made at 14 locations and origin-destination interviews were conducted at 8 locations on the road network. The counting period lasted from 0700 hours to 1900 hours in two-hour divisions. This procedure and the data obtained, together with monthly indications of traffic trends in The Gambia, formed the basis for estimating annual traffic levels and related characteristics.

Daily traffic volumes on rural roads, as shown in Figure 3, were estimated to vary between nearly 700 vehicles on the Barra to Kaolack Road (leading to Senegal) to approximately 40 vehicles on roads in eastern areas of the country on the north bank. Over 600 vehicles per day occurred between the major centers of Banjul and Soma. On the Trans-Gambia Highway, between northern and southern areas of Senegal, 230 vehicles per day occurred at the ferry crossing and approximately 350 on the road segments leading

Figure 3. Estimated daily traffic volumes on principle rural roads.



to and from it, indicating considerable local activity. Volumes diminished significantly toward the eastern extremity of the country, reflecting primarily the reduced settlement density.

Traffic volumes in urban areas, as might be expected, were much higher than those on rural roads. Nearly 9,000 vehicles per day were observed on the main road into Banjul, the Capital.

The percentage of non-Gambian vehicles in the traffic stream varied from approximately 75 percent at the Trans-Gambian ferry to approximately 2 percent on other major roads. As expected, apart from the Trans-Gambia highway, the greatest volumes occurred on roads leading to and from Senegal, varying between 8 and 25 percent.

Vehicle classification observations indicated that passenger cars were the largest component of the traffic stream, usually varying from between 60 and 80 percent on the roads with the higher volumes to as low as 25 percent on the more remote roads. Most passenger cars were used as taxis. Between 10 and 20 percent of vehicles were trucks. However, this was exceeded on the Trans-Gambia highway, where 31 percent of all vehicles were recorded as trucks. The percentage of buses is considerably lower, varying between zero on some north bank roads which are unpaved, to 32 percent on the Trans-Gambia highway ferry.

On feeder roads in The Gambia, traffic rarely exceeds an average of 50 vehicles per day, based upon existing records and observations made during reconnaissance and evaluation trips throughout the road network. In many cases the volume appears to be in the order of 20 vehicles per day or less, and a large proportion of this traffic consists of animal-drawn carts, government vehicles, motor and pedal cycles. In some instances where villages, groundnut buying stations or other activity centers are located near main roads, higher volumes occur. However, these may be regarded as access volumes on a very limited portion of the total network.

Physical Deficiencies of the Road System

Field inspections of road conditions were made to establish a "condition log" which could be used as a basis for establishing needed improvements and costs.

The condition of the road pavement has been ranked from a numerical index to one of an index of five, as follows:

Index 1 - New Pavement
Index 2 - Pavement not new but with adequate riding quantities and transverse shape
Index 3 - Pavement irregular in shape but still basically sound, correctable with grading and/or a premix regulation course
Index 4 - Excessively deformed pavement shape requiring base and surface repair
Index 5 - Pavement failed; route detour required

Typical examples of the above conditions are shown in Figure 4.

Figure 4. Road condition index.

Condition 2 - acceptable.

Condition 3 - surface deterioration.

The nature of general deficiencies noted at specific locations during the field observations are as follows:

<u>Shell Bitumen Surfaced Roads</u>

1. Excessive camber and longitudinal deformation in outside wheel tracks.
2. Some heavily travelled road surfaces are in imminent danger of failing.
3. Edge failure of the bituminous surface.
4. Uncut grass to the edge of the bituminous surface (December, 1977).
5. Culvert markers, headwalls, bridge parapets and bridge handrails need repairs and repainting.

<u>Sand Bitumen Surfaced Roads</u> (mostly Banjul and adjacent areas)

1. Excessive deformation.
2. Pavement failures on heavily travelled sections.

3. Edges eroded and regraveling needed.

Basalt Chipping Bituminous Surface Road (Trans-Gambia Highway)

1. Failure of the base and subbase on approach sections to Gambia River ferry. (This road carries heavy vehicles.)
2. Ferry approach sections need reconstructing.
3. Edges eroded; regraveling needed.

Gravel Surfaced Roads

1. Surfaces worn and corrugated.
2. Cross-section irregular.
3. Needs regraveling to restore the pavement strength and camber.
4. Edges eroded.
5. Culverts, bridge edges eroded and require repair.

It should be noted that many of the above observations were made shortly after the wet season, 1977, and maintenance was being undertaken to remedy deficiencies.

Future Design and Construction Guidelines

During the 20-year period from 1977 to 1997, an increased demand for road and river transport will occur to accommodate increased crop movement, imports and exports, and general demand for personal mobility. A review of the planning and systems aspects of the proposed system is provided below to provide a background to the design and construction guidelines recommended for future development.

Transportation Systems Review

Following estimation of the major future demands and formulation of potential river and road improvement alternatives (three road networks), cargo and passenger demands were assigned to the networks. This process consisted of a series of manual iterations of traffic assignments, initial cost evaluations, network adjustments and consideration of the potential benefits expected from unit expenditures on the principal projects.

The river and road traffic estimated to occur in the future was reviewed for consistency with general economic indicators and with potential development in The Gambia as described in the Five-Year Plan and other documents.

In general, traffic volumes are expected to be approximately 3.5 to 4 times greater in 1997 than in 1977. This includes an anticipated increase in the size of trucks from 6 to 11 ton capacity for most groundnut transport and consolidation of buying stations and shipping points. Associated use of ferries is expected to increase in direct proportion to traffic volumes.

Functional Classification. The recommended roadway system is classified into three functional components--primary, secondary, and tertiary roadways. See Figure 5.

Of the 1,212 kilometers of highway in the designated system, about 658 are classified as primary, 345 as secondary, and 209 as tertiary. With the completion of the recommended plan, almost 55 percent of the principal roads system will have a bitumenized paved surface; a little more than 45 percent will be gravel surfaced.

Road Network Development Guidelines

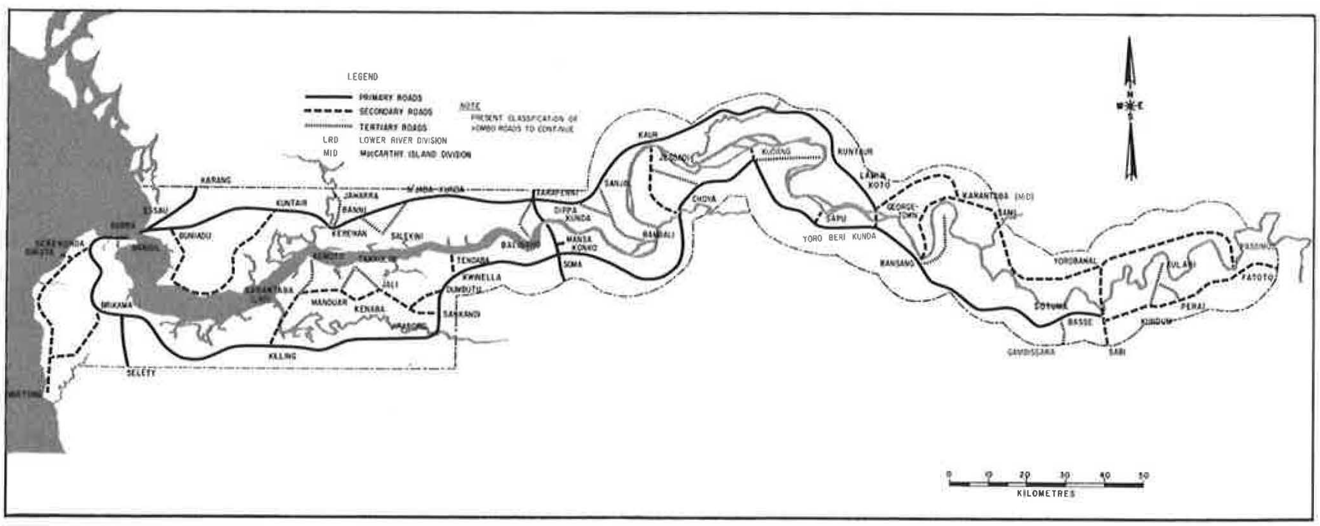
It was apparent that new or modified practices would be beneficial in several areas related to design, and construction. This section, therefore, indicates guidelines to assist in these areas.

Basic Considerations. One of the characteristics of road traffic in The Gambia is the diversity of both vehicle types, ranging from ox carts to articulated lorries, and operating speeds. This, combined with pedestrian activity within and between the many villages along main roads, is likely to continue into the forseeable future. Road design standards must recognize these complexities to minimize hazardous situations encountered by the road user and at the same time attain the maximum facility effectiveness.

Recognizing the diversity of travel needs, vehicle types and operating speeds, the following basic criteria for the major roads were set:

1. Pedestrian paths or tracks should be provided along all paved roads, adequately separated from motor vehicle traffic.

Figure 5. Recommended roadway system, functional classification.



2. The road pavement should be wide enough to allow two large commercial vehicles to pass safely, without reducing speed.

3. Road geometry should be able to accommodate operating speeds of up to 80 to 100 kilometers per hour, depending upon the type of road.

4. The road shoulders should be wide enough to accommodate very slow moving vehicles well clear of the road pavement. They should also provide a safe refuge for disabled or parked vehicles.

5. The horizontal and vertical alignments must allow ample opportunities for overtaking with safety.

6. The right-of-way should be wide enough to minimize the chance of a vehicle, when forced to leave the travelled way, colliding with a man-made structure or other object.

Suitable provision should be made for bus, taxi, and lorry stops. Adequate lay-byes and, particularly in built up areas, service or frontage access roads should be provided, together with appropriate signs and markings for pedestrian crossings.

The guidelines on road design and construction and the general approach to their implementation have been developed with the above considerations in mind.

Classification of Roads. In recommending a road classification system, the following objectives were set:

1. There should be only one classification system established which can be used by all interested parties.

2. The classification must have a specific purpose, be capable of clear definition, and be adaptable to future needs.

3. If the classification results in visually distinctive road types, then the road system must appear to be logical to the road user.

4. The road classification system should be consistent with the method of financing. Nationally important roads could be financed from the national sources, locally important roads from local revenue.

5. Definition of the various classes of roads should be sufficiently rigid, on the one hand, to enable any road to be logically classified, and sufficiently flexible, on the other hand, to allow for improvement of a road without having to change its class simply because of the improvement.

6. The system of roads that the user perceives and uses should be logically constructed. For a major road, the standard of construction should be uniform throughout its length. Progressively lower standards will apply to roads carrying less traffic.

Guidelines. The future classification of roads in The Gambia described here are based on functional performance categories. They can be applied as a tool for national planning.

It was recommended that the primary, secondary, and tertiary roads should be officially gazetted and road design standards adopted for each class. Local roads should not be gazetted. The main features of these categories are as follows:

1. Primary Roads. Those roads which, at a national scale, form a continuous all-weather network of roads, linking all major centers of population and providing the major international links.

2. Secondary Roads. Those roads which, by their connection to the primary road network or the river transport system, serve to maintain all-weather access to and within the various regions of the country.

3. Tertiary Roads. Those roads which are necessary for the maintenance of all-weather access to local areas, from either the primary or secondary roads or the river transport system.

4. Local Roads. Those roads which are made solely for local use. They may or may not be all-weather roads.

Responsibility. Central Government should accept responsibility for:

1. Definition of the actual network of primary, secondary, and tertiary roads.

2. Adoption of road design standards for these roads.

3. Construction and maintenance of primary roads.

4. Construction and maintenance of secondary roads on the understanding that as local government develops, maintenance of the secondary roads could be undertaken by local government, with some financial assistance from central sources.

5. Such financial and technical assistance to local government as may be necessary to enable Area Councils to develop and maintain the gazetted tertiary roads.

There are many aspects as to classification, funding, and general administration of a national road system. It is believed that the guidelines mentioned above will provide a sound basis for more extensive planning.

Road Design Standards. Road design standards so far submitted for projects in The Gambia have either been for a particular road or have been formulated by road class. However, these design standards, although based on sound engineering principles, may not be sufficiently flexible to efficiently cater for existing and proposed road traffic movements within The Gambia.

Consequently, based upon specific requirements in The Gambia, five design classes were recommended, predicated on average daily traffic volumes on opening and modified to recognize the type of terrain (flat or rolling) through which a road would pass. (On a newly constructed road, it may be necessary to base the design class on predicted rather than actual volumes.) A summary of the main features of these guidelines is shown in Table 1, and typical cross-sections are shown in Figure 6.

These design recommendations are based upon a World Bank review (A Review of Highway Design Practiced in Developing Countries, Con, F. W., May 1975) of highway design as practied by developing countries, wherein design standards are related to anticipated traffic flows, rather than road classification. Adoption of this principle will enable The Gambia to make most use of available funds, at the same time ensuring that each road segment is adequately designed to carry the present and future traffic.

Essentially, the design classes as shown in Table 1 and Figure 6, vary from Class A with over 2,000 vehicles per day to Class E, with less than 50 vehicles per day. For each design class, the following design elements are considered:

1. Design speed.
2. Width of surfacing.
3. Shoulder and formation width.
4. Width of bridge decks.

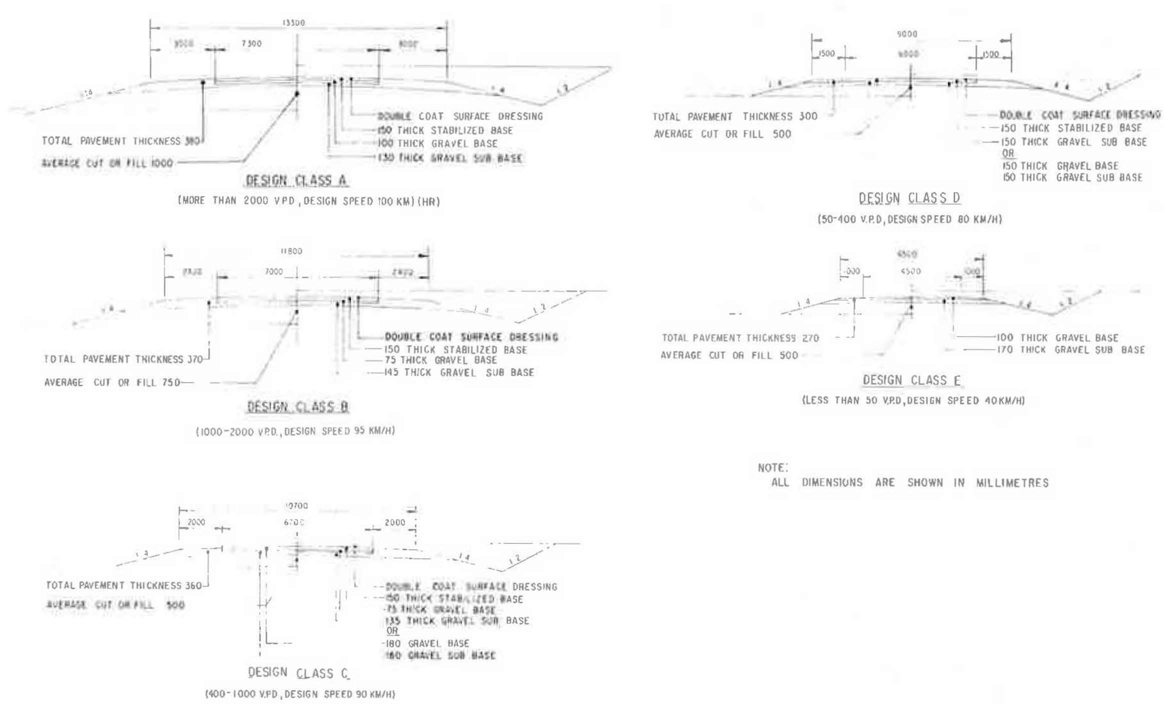
Table 1. Summary of geometric design standards.

Design Element	Unit	Primary Roads Flat	Primary Roads Rolling	A Flat	A Rolling	B Flat	B Rolling	C Flat	C Rolling	D Flat	D Rolling	E Flat	E Rolling
				Design Class by Terrain									
ADT on opening (mixed traffic)	Veh.			Over 2,000		1,000 to 2,000		400 to 1,000		50 to 400		Under 50	
Design Speed	km/h	100	90	100	90	95	80	90	80	80	65	60	40
Surfacing width	m	As for Design Class		7.3	7.3	7.0	7.0	6.7	6.7	6.0	6.0	4.5	4.5
One shoulder width	m	As for Design Class		3.0	3.0	2.4	2.4	2.0	2.0	1.5	1.5	1.0	1.0
Width between bridge parapets													
20m. long or longer	m	As for Design Class		13.3	13.3	11.8	11.8	10.7	10.7	9.0	9.0	4.0	4.0
less than 20m. long	m	As for Design Class		9.3	9.3	9.0	9.0	8.2	8.2	7.7	7.7	4.0	4.0
Stopping sight distance	m	160	135	160	135	150	115	135	115	115	80	70	40
Passing sight distance	m	670	600	670	600	635	530	600	530	530	420	380	240
Maximum gradient	%	4.0	5.0	4.0	5.0	4.0	5.0	5.0	6.0	6.0	7.0	6.0	8.0
				(note: Maximum gradients may be exceeded by 1% on short lengths of road)									
Horizontal curve minimum radius	m	345	280	345	280	310	210	280	210	210	135	115	50
Vertical Curve K value													
Crest curves		128	91	128	91	113	66	91	66	66	32	25	8
Sag curves		38	31	38	31	35	25	31	25	25	16	13	6

		Primary Roads	Secondary Roads	Tertiary Roads	Local Roads
Right of way width	m	60	60	40	30
Minimum distance of services from centre line of road	m	25	25	15	10

		All Roads	
Crossfall on half width			
Bituminous surface	%	2.5	
Gravel surface	%	4.0	
Embankment fill slopes			
Up to 2m. high	%	25	
Over 2m. high	%	50 - 67	Depending upon stability requirement
Cutting slopes	%	Up to 100	Depending upon stability requirement

Figure 6. Standard cross sections.



5. Sight distances.
6. Gradients.
7. Vertical curvature.
8. Horizontal curvature.

Right-of-way widths are proposed by road classification, and a uniform set of cross-slopes is recommended for all road types.

The degree to which a road can be built to satisfy the above standards will depend primarily on the funds available and the actual dimensions and limits adopted will depend upon the status of the road and the number of vehicles using it.

Axle Load Regulations. It was recommended that a ten ton axle load limit be applied to all vehicles operating on Gambian roads. This limit would be applicable to all roads--primary, secondary, and local.

However, it is recognized that at least one road in The Gambia, the Trans-Gambia Road, will be carrying significant volumes of international traffic with axle loads up to a limit of 13 tons.

The existing Gambian axle load limits for Special Roads (The Laws of The Gambia, Motor Traffic Act, 1966) (The Trans-Gambia Road and the Banjul-Mansa Konko Road) are 12.2 tons for a two-axle vehicle and 16.8 tons for an articulated vehicle.

It was recommended that these limits be rationalized to 13 tons, regardless of vehicle types, for Special Roads. In the very long term, other international routes may be declared Special Roads, although there appears to be little need to do so in the near future.

Pavement design recommendations (United Kingdom, Department of Transport, Road Note 31, A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Subtropical Countries, 1977) take axle loading into account.

Materials Availability

Within the foreseeable future, there is a definite possibility of a road construction materials shortage occurring in The Gambia. In the cases of laterite, material of the required quality and grading has been identified in various locations throughout the country. Sand and shells, however, appear to exist only in the western parts of the country, making transport to other locations an expensive necessity. As regards construction timber, the ruhm palm is becoming increasingly scarce, therefore jeopardizing the economical construction of jetties, bridges, and related works.

Several actions concerning the construction materials situation are recommended. The provision of a limited amount of shell extraction and washing equipment could be procured and assembled locally. A construction materials availability study, including the technical feasibility of ruhm palm plantation establishment, could be considered an essential feature of a national construction program of benefit to governmental and private organizations alike.

Acknowledgements

The authors would like to acknowledge the Kuwait Fund for Arab Economic Development, who sponsored the study upon which this paper is based, and The Government of The Republic of The Gambia, for their kind permission to quote sections of the study. Recognition is also due to other team members, David Emery and Peter Green, highway engineers, and John M. Thompson, economist, in developing much of the data presented. The statements herein, however, are the sole responsibility of the authors.

ROAD NETWORK ANALYSIS FOR TRANSPORTATION INVESTMENT IN EGYPT

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A highway transportation planning framework developed at M.I.T., the Road Investment analysis Model (RIAM), has been applied to the analysis and planning of highway investment decisions in Egypt. Ten alternative maintenance policies were analyzed over the network study zone, reflecting three types of investment concerns important to the Egyptian Transport Planning Authority. Among the questions addressed were: (1) the relative frequency of the maintenance activities; (2) the relative magnitude of the investments as reflected in the overlay thickness; and (3) the relative levels of investment among the three road classes-primary - secondary, and tertiary - to achieve the most effective overall investment strategy in the network. Additionally, the heavier the level of investment, the greater was the economic return from that investment, given that the investment was distributed fairly evenly among the various road classes. The optimum alternative identified was found to consist of frequent, light overlays on the primary system (where additional structural strength is not required) and initial heavy overlays followed by frequent, light overlays on the secondary and tertiary systems (where additional structural strength is needed to meet future traffic demands). These results held true whether the performance was judged on economic efficiency or user satisfaction (consumer surplus).

The road network in Egypt comprises over 26,000 kilometers of roads with approximately 12,000 km paved and 14,000 km unpaved, and highway intercity transportation accounts for approximately three-fourths of present annual ton-kilometers and passenger kilometers (a situation which is expected to be maintained over the next decade). Although recent partial appraisal reports of the road network by independent consultants have concluded that on average the highway system is in relatively good condition, with the poorer conditions being concentrated in the less heavily trafficked paved roads, the capabilities of the Egyptian Ministry of Transport to maintain the present system or expand the paved network during the coming decades have not been sufficiently addressed. The relative importance of the various links comprising the primary, secondary, and tertiary road networks to the overall performance of the road transport network are not well understood, nor is there agreement on the appropriate design and maintenance standards for the various classifications of links and upon criteria for upgradings.

Given the limited resources available for road transportation investments, either in construction of new facilities or maintenance and upgradings of existing facilities, there is a growing need within the Egyptian Ministry of Transport for a hierarchy of analytic techniques capable of evaluating the capital budgeting and programming of alternative scenarios for major investments in expanding or upgrading the network; of analyzing the various interdependencies between the alternative projects and maintenance policies of a road system, including both the appropriate timing and scaling of these projects; and of evaluating the economic consequences of alternative design, construction, and maintenance standards. The present Egyptian techniques for evaluation of road transport investments and corresponding data inventories are rather inadequate for the application of analytic models for road investment evaluation.

The purpose of this research is to demonstrate the application of an existing analytic framework, the Road Investment Analysis Model, to road transport investment decisions in Egypt, and to develop the necessary procedures for its modification and and adaptation to the local Egyptian conditions. The scope of the research effort during the first phase was to evaluate road transport in a zone comprising roughly one-fourth of the paved road network of the Nile Delta at both the link and the network level. During the link-evaluation phase, five reconstruction and subsequent maintenance policy alternatives were analyzed on each of twenty-five links independently (1). The five alternatives included do-nothing; a premix overlay followed by either scheduled or demand-responsive maintenance; and a double (asphalt over premix) overlay followed by either scheduled or demand-responsive maintenance. For each alternative and on each link, economic analyses of net present value, first year benefits, and internal rate of return were performed against the do-nothing alternative. Where data were lacking or unreliable, surveys were undertaken or engineering judgments were used.

The results of the twenty-five link-evaluation applications of the Road Investment Analysis Model indicated that there were four classes of link performance in the study zone under the proposed structural overlays and subsequent maintenance standards. These classes were:

1. Links which are in excellent present condition and for which the proposed investments may be postponed;
2. Links which are in poor present condition and for which the proposed investments produce acceptable results. This means that the heavier structural overlay performs satisfactorily for the entire analysis period thus requiring no major subsequent maintenance investments;
3. Links which are in poor present condition and which performed marginally under the proposed investments, in that the heavier structural overlay failed in from 5-14 years requiring major subsequent maintenance investments, and for which heavier initial investments should be considered; and
4. Links which are of weak structural strength and poor present condition and which would have performed poorly under the proposed investments, in that both of the proposed structural overlays failed in less than 5 years and where all of the net benefits accrued from major subsequent maintenance investments, and for which heavier initial investments or reconstruction must be considered.

A tentative general finding which held true on all links analyzed was that, of the four post-construction maintenance policies studied, the one with the heaviest overlay dominated the economic analysis, independent of the effectiveness of the original double (asphalt over premix) overlay, which was the more extensive of the two construction options analyzed. Thus, on all links studied, the heaviest construction and maintenance investments produced the greatest economic net present values. This and the above performance classification are being reviewed in light of assumptions made during data collection. A visual inspection was made of representative links in the study zone to verify these predictions.

The link analyses disclosed several key data items which had to be obtained before the ensuing network analysis could be undertaken. These data items were collected by the Transport Planning Authority (road user costs), the Road and Waterway Authority (traffic counts and axle load distributions), and Cairo University (road deterioration relations) personnel. These data collection efforts are documented in a companion report to this study (2). In addition, the need for rapid and accurate road condition evaluation equipment was emphasized.

This paper procedes from the work done at the link-evaluation level, with most of the data items, including construction, maintenance, and vehicle operating costs, reported in our previous work (1). The new data items include the maintenance strategies to be analyzed, and the origin-destination matrix for the base year conditions. These will be discussed further in subsequent sections of this paper.

The area chosen for study is flat, mainly cultivated land lying to the north of Cairo, and containing the Monofia and Qalubia governorates road network. This area was selected for the study zone for the following reasons:

1. Availability of records concerning construction and maintenance of the road network;
2. Physical homogeneity of link characteristics;
3. Diversity of traffic volumes, providing a cross-section of the spectrum of traffic conditions likely to be encountered on the Egyptian road network;
4. Possibilities for a route-choice study; and the
5. Economic importance of this road network to the Highway Authority in Egypt.

The zone of study in Egypt is shown in Figure 1, displaying the zones into which the country was divided for purposes of the origin-destination survey, which will be discussed later. The roads analyzed within the study zone were classified into three categories, depending on their utilization, as shown in Figure 2. These classes were as follows: Class I is the Cairo-Alexandria Agricultural Expressway, which carries the majority of the national inter-city transport; Class II which represents the national highways connecting the major secondary cities to the Cairo-Alexandria corridor; and Class III which represents the paved road network connecting the minor cities with the rest of the national road system. Unpaved roads were not analyzed in this investigation.

These classes were chosen so that investigations into the relative and absolute magnitude of investments among the various road classes could be undertaken. Specifically, we wished to address the question of, given a network investment program, where do the major benefits accumulate?

An origin-destination survey was conducted in the study zone on the 8th of August, 1977. The interview was designed to obtain information on vehicle type and route choice. Besides requesting where the vehicle was coming from and going to, which were at the governorate and markaz level, the questions were aimed at determining route choice behavior: What are the major cities on your route? Will you stop at any city en route: If so, for how long? Do you normally take this route? If so, why? If not, why this time? and what is your estimate of your total travel time? The intent of the interview was to determine as completely as possible any previous knowledge of O-D route-alternatives, perception of travel time, and pre-trip route decision.

The study area was divided into 16 zones, corresponding to the markaz boundaries. The external area surrounding the study area was divided into 11 zones, using natural boundaries, such as a canal, where possible and such that each external zone contained a major highway which entered the study area as shown in Figure 1. The external zones' traffic (generation and attraction) was then assigned to the neighboring zone centroid in the study area to convert all the survey volumes into demands within the study area.

After the data had been collected, manual processing was carried out. Since five vehicle types were interviewed (private cars, inter-city taxis, buses, trucks, and trucks with trailers), there are five O-D matrix sheets for every interview station. As the sample size interviewed for each vehicle type was considered as representative of the daily traffic for the corresponding vehicle type at the interview station, the O-D matrix was expanded by the Average Daily Traffic (ADT) for a certain vehicle type at the interview station divided by the sample size for that vehicle type. Each cell in the O-D matrix was multiplied by this factor, producing five enlarged O-D matrices for each interview station.

Traffic generated or attracted to a specific zone within the study area was considered to be accurately given in the resulting O-D matrices from interview stations on road links crossing the zone

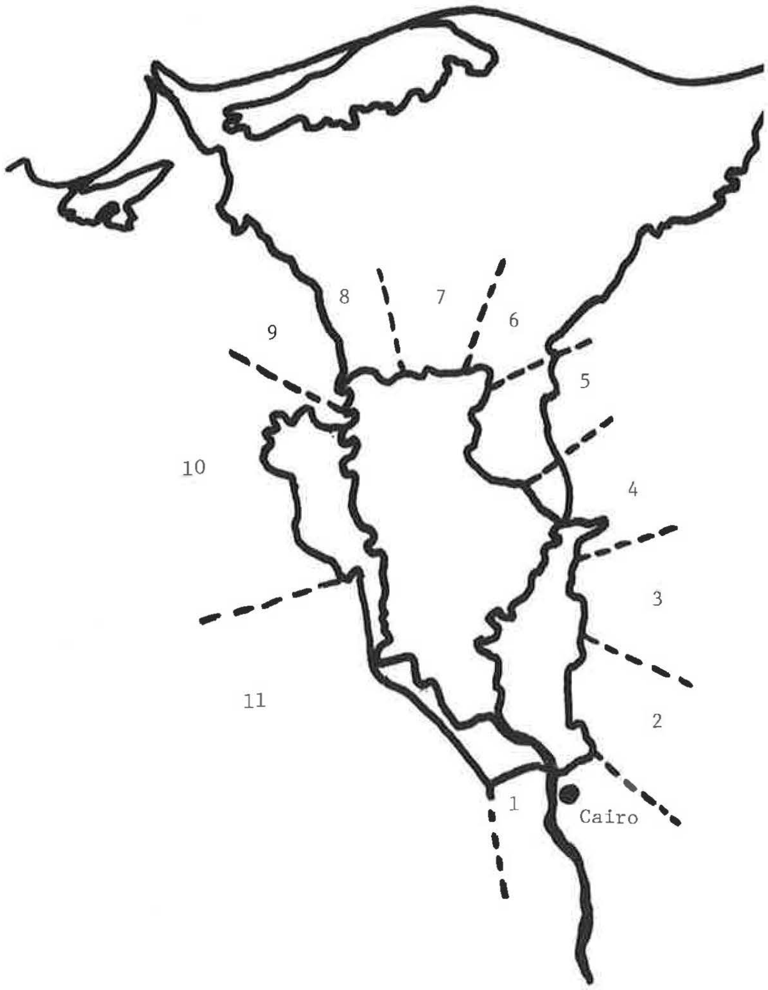


Figure 1. Nortern Egypt and the study zone.

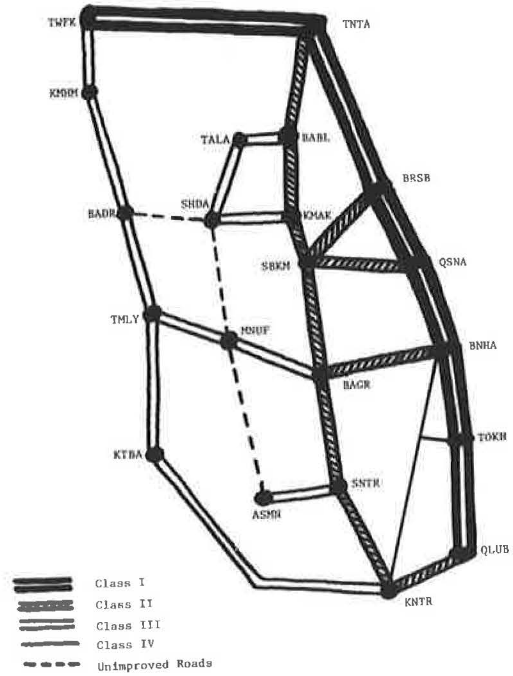


Figure 2. The Monofia and Qualubia Governorates road network.

Note: Class IV and unimproved roads were not analyzed due to lack of O-D data on these links.

Note: Node identifiers refer to village abbreviations.

Table 1. Final O-D matrix - - two-way average 1977 daily traffic (all vehicle types).

	QLUB	TOKH	BNHA	QSNA	BRSB	TNTA	TWFK	KMHM	BADR	TALA	SHDA	SBKM	MNUF	BAGR	ASHM	KNTR
QLUB	--															
TOKH	609	--														
BNHA	12566	58	--													
QSNA	272	43	720	--												
BRSB	491	--	--	--	--											
TNTA	3165	--	472	201	238	--										
TWFK	11012	58	506	--	63	4590	--									
KMHM	723	--	--	--	--	291	--	--								
BADR	--	--	--	--	--	--	--	--	--							
TALA	145	--	40	--	--	450	--	--	--	--						
SHDA	109	--	--	44	--	71	--	--	--	--	--					
SBKM	969	--	626	508	416	900	291	--	--	76	663	--				
MNUF	628	--	69	68	--	--	691	180	656	--	--	128	--			
BAGR	806	66	--	--	--	--	--	--	--	--	--	67	181	--		
ASHM	1074	--	35	--	--	--	70	--	--	--	--	93	333	175	--	
KNTR	1043	54	37	--	--	--	--	--	--	--	--	88	--	--	117	--

in question. Traffic originating and destinating at any specific zone was then determined by the addition of the zone O-D cells given in the enlarged O-D matrices of the corresponding interview stations.

Once the O-D matrices were in hand, they had to be calibrated to the measured link traffic volumes to determine the traffic assignment diversion intensity parameter incorporated within the Road Investment Analysis Model. This parameter is vehicle type specific (but not a function of time or origin and destination), and was estimated using the disaggregate route-choice data obtained from the O-D survey. The values of time used for Egypt were taken as zero for private cars, taxis, and trucks; LE 1.85/hr. for buses; and LE 10/hr. for trucks with trailers; which were obtained from Transport Planning Authority estimates.

Sensitivity runs were performed on the diversion intensity parameter until the discrepancies between measured and predicted link volumes could no longer be attributed to route-choice behavior. This resulted in a diversion parameter of 1.0, meaning that one unit of actual cost difference between routes is perceived as one unit of cost difference by the road user. Local traffic was then added between some neighboring O-D pairs (less than 2 percent of all traffic demand), to bring all predicted link volumes within ten percent of the measured volumes. The resulting master O-D matrix is given in Table 1. Traffic was assumed to grow uniformly at 7 percent for all vehicle types throughout the analysis period, 1977-2000.

The following conclusions were drawn from this calibration effort:

1. The final assignment was reached with a diversion intensity parameter of 1.0.
2. The network flow pattern was sensitive to the value of the diversion parameter, and any significant change from 1.0 would result in a decrease in the overall accuracy of the predicted link volumes.
3. The final assignment reached with a diversion parameter of 1.0 necessitated the addition of some local traffic (less than 2 percent of the total demand) to the system.
4. The average error between the actual traffic measurements and the predicted traffic flows on the network from the final assignment was 5.2%.

Economic Analysis and Results

Ten alternative maintenance policies were analyzed over the network study zone, reflecting three types of investment concerns of importance to the Egyptian Transport Planning Authority. First was the question of the relative frequency of the maintenance activities, in this case asphalt concrete overlays, which will affect the mean road surface conditions for each road class. Second, was the relative magnitude of the investments as reflected in the overlay thickness, which would affect the rate of future deterioration of the riding surface in each road class. Third, was the question of the relative levels of investment among the three road classes - primary, secondary, and tertiary - to achieve the most effective overall investment strategy in the network. The ten maintenance investment policies are summarized in Table 2. The road classes within the study zone were shown in Figure 2.

All policies were identical as regards the maintenance activities of patching, surface dressing, and miscellaneous maintenance. The patching standard was to repair 80 percent of all unpatched cracks each year, but not to exceed 50 square meters per kilometer per year. The surface dressing standard was to perform a surface dressing on the entire road surface whenever the total area of cracking and patching exceeded 10 percent of the total road surface area, but not more frequently than every two years or less frequently than every five years. No surface dressings were performed after the year 1995, so that major maintenance investments would not be performed in the last analysis years where they might not have sufficient time to realize their full benefits. Although these items were standardized in all policies analyzed, the actual quantities of patching and surface dressing varied slightly from strategy to strategy due to the demand-responsive nature of the maintenance. Routine maintenance of drainage structures, vegetation, shoulders, safety installations and miscellaneous items was constant over all policies analyzed.

The primary focus of this study was on the relative frequency and thickness of overalys among the various road classes. The alternatives were selected to illustrate as thoroughly as possible the effects of the various options available in a reasonably small number of strategies. Five of the alternatives involved holding the thickness of all overlays constant at 40 mm of asphalt concrete while varying the absolute and relative frequency of overlays among the various road classes, as measured by the threshold roughness at which an overlay would be performed. The remainder of the alternatives involved varying the thickness of the applied overlays with the overlay thickness increasing by 20 mm for every 1000 mm/km of additional roughness tolerated before performing the overlay, while using similar absolute and relative frequencies of overlays among the three road classes as were analyzed in the first group of strategies. No overlays were performed after 1994, so that these major maintenance investments would not be done in the last analysis years where they might not have sufficient time to realize their full benefits.

Figure 3 shows the effects of overlay frequency while holding the thickness of all overlays constant, presenting the discounted net present value (in this paper, net present value includes maintenance and user costs) of all alternatives (using Alternative 1 as the base case) versus total discounted maintenance cost, discounted at 12 percent. All quantities are economic values in millions of Egyptian pounds. The arrows indicate the direction of increasing overlay frequency.

Alternative 1 represents a light, but relatively uniform, level of overlay investment in the network, whereby the links are overlaid at roughness levels of 5000, 6000, and 7000 mm/km in road Classes I, II and III, respectively. This policy also produces the worst economic network performance of all of the alternatives analyzed. Alternative 2 represents a heavier, but still relatively uniform, level of investment with overlays at 4000, 5000 and 6000 mm/km, respectively. This produces a great improvement over Alternative 1, yielding a net present value of roughly LE 140 million for an incremental investment cost of less than LE 1 million. Alternative 3 represents a yet heavier, but still relatively uniform, investment level with overlays at roughness levels of 3000, 4000 and 5000 mm/km, respectively. This produces an incremental net present value of LE 106 million (over Alternative 2) at an incremental investment level of only LE 1 million. Thus, from the three cases of relatively uniform investments in the three road classes, we can conclude that the returns on investment, although decreasing as the investment level increases, are very large.

Alternative 5 in Figure 3 represents a very non-uniform investment level among the three road classes, with links in road Classes II and III never being overlaid and links in road Class I receiving overlays at a threshold roughness of 3000 mm/km. Although this investment strategy is superior to Alternative 1, it is clearly inferior to the moderate to heavy, uniform investment strategies as represented by Alternatives 2 and 3. In addition, it produced a highly skewed traffic distribution pattern which is viewed negatively by the road users, as will be discussed later in this report. Alternative 4 represents another non-uniform investment proposal, wherein links in Class I are not overlaid and those in Classes II and III are overlaid at roughness levels of 3000 and 4000 mm/km, respectively. This strategy is inferior economically to the non-uniform investment in Class I roads (as given by Alternative 5) and is also inferior to the moderate to heavy, uniform investment proposals as given by Alternatives 2 and 3. Indeed, the heavy, uniform strategy of Alternative 3, which is in a sense a "superposition" of the non-uniform investments, produces over twice the net present value as Alternatives 4 and 5 combined. However, Alternative 4 is preferable to Alternative 5 from the road user point of view, as measured by consumer surplus, as will be described later.

From the above, we can conclude that for the case of constant thickness of overlays throughout the system, the more intensive and uniform investment alternatives dominate the economic performance of the system. This is even more true when the consumer surplus criterion is used as a measure of network performance, as will be shown later.

Figure 4 shows the effects of varying overlay thickness while keeping the absolute and relative frequency of overlays among the three road classes constant. Alternative 6 reflects the same overlay frequencies as Alternative 1, namely at roughness levels of 5000, 6000 and 7000 mm/km, but incorporates overlays of thickness 80, 100 and 120 mm, respectively, instead of the 40 mm used in Alternatives 1 through 5. This investment strategy produces a net present value of LE 190 million at an incremental maintenance investment of slightly more than LE 2 million, producing an economic network performance inferior only to the heavy, uniform investments of Alternative 3 (of the alternatives discussed so far). Alternative 7 reflects the same overlay frequencies as Alternative 2, namely at roughness levels of 4000, 5000 and 6000 mm/km, but incorporates overlays of thickness 60, 80 and 100 mm, respectively, instead of the 40 mm overlays. Again, the economic return of the thicker overlays is impressive, LE 100 million on an incremental investment of LE 1.5 million, although, through a combination of the higher overlay frequency and thinner overlays, the transition is less massive than that indicated by the comparison of Alternatives 1 and 6. Alternative 8 reflects the uniform, heavy overlay frequencies of Alternative 3, namely at roughness levels of 3000, 4000 and 5000 mm/km, but uses overlays of thickness 40, 60 and 80 mm, respectively, rather than the uniform 40 mm overlays. Again the return is impressive, LE 34 million on an incremental investment of LE 0.7 million, although again there is less return on investment than generated through the 1-6 or 2-7 transitions. Alternative 9 reflects the same non-uniform investment distribution as Alternative 4, namely, no overlays in Class I links and frequent overlays in Class II and III links, although in this case the overlays are of thickness of 60 and 80 mm, respectively. The huge economic return of nearly LE 100 million on an incremental investment of less than one-half million indicates that, if only Classes II and III are to be overlaid, these overlays should be of substantial thickness.

From the above considerations, we can conclude that, given a relative frequency distribution for overlays in the various road classes, the heavier the overlays the greater the economic return, and the greater the user satisfaction with the system as measured by the consumer surplus criterion, which will be discussed later.

Figure 5 shows the economically "efficient" maintenance policy set, which is bounded by Alternatives 5, 3, 8, and 10. The common characteristic of this set of alternatives is that each specifies overlaying Class I roads at the most frequent level analyzed: i.e., 3000 mm/km with a 40 mm overlay. These overlay policies are summarized in Table 3. (This is not surprising, since a review of Table 1 indicates that 73 percent of the total traffic demand is between cities lying in the Class I corridor. What is surprising, however, is that so little of the potential benefits come solely from investments in this class.) From Table 3 and the figure, we can see that the majority of the benefits come either from the high overlay frequencies on the Class II and III links contained in Policy 3 or from the increased thickness of these overlays presented by Policy 10, which performs the overlays much less frequently as measured by threshold roughness levels. This suggests that a hybrid policy consisting of the thick initial overlays of Policy 10 followed by the lighter, but more frequent overlays of Policy 3 would produce the greatest economic performance and user satisfaction with the network.

Figure 6 shows the discounted net present values of the various alternatives as a function of discount rate. As was expected, in a multi-link situation such as a network analysis where many separate investments may be undertaken in any given year, the ranking of the alternatives is not sensitive to the discount rate. This is shown by the "parallelness" of the net present values as indicated in the figure.

Consumer Surplus Criterion

The discounted maintenance costs, vehicle operating costs, total costs, and net present values for all alternatives are shown in Table 4. Also shown is the net consumer surplus of each alternative. The consumer surplus is a measure of the "user satisfaction" with the system, and in the current case is taken as the sum of the actual economic costs of the system and the difference between the actual and perceived financial costs of the system. Thus, the difference between the actual and perceived cost of the system is a "proxy" measure for the user satisfaction with the system. The greater the difference, the less the users of the system think they are actually paying with regards to the actual cost of using the system. In another sense, the greater the difference between the actual and perceived costs, the more route-choice options are available to the user, since in the case of only one route the actual and perceived costs are identical. In the current case study, the difference between the actual and perceived costs is taken as the measure of network performance, and hence as a benefit to be added to the economic performance of the system (3).

Using this new measure of system performance, which is included in Table 4, we can again rank the network performance of the various maintenance

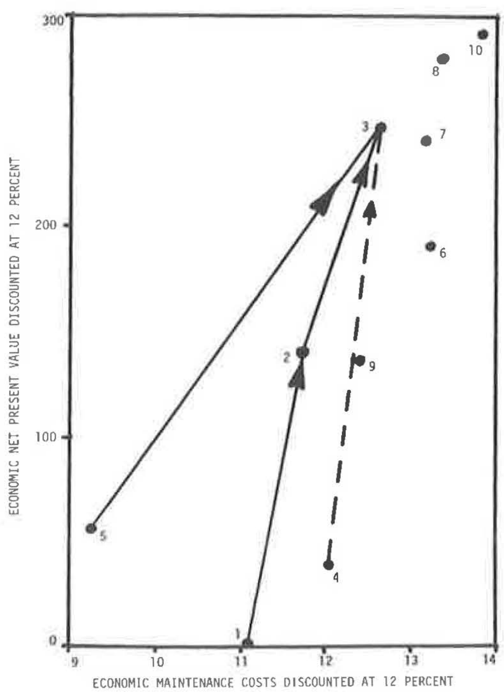
Table 2. Maintenance overlay policies.

Policy	Overlay Frequency		
	Class I	Class II	Class III
1	5000[a] (40)[b]	6000 (40)	7000 (40)
2	4000 (40)	5000 (40)	6000 (40)
3	3000 (40)	4000 (40)	5000 (40)
4	Never	3000 (40)	4000 (40)
5	3000 (40)	Never	Never
6	5000 (80)	6000 (100)	7000 (120)
7	4000 (60)	5000 (80)	6000 (100)
8	3000 (40)	4000 (60)	5000 (80)
9	Never	4000 (60)	5000 (80)
10	3000 (40)	6000 (100)	7000 (120)

[a]Overlay roughness threshold, mm/km.

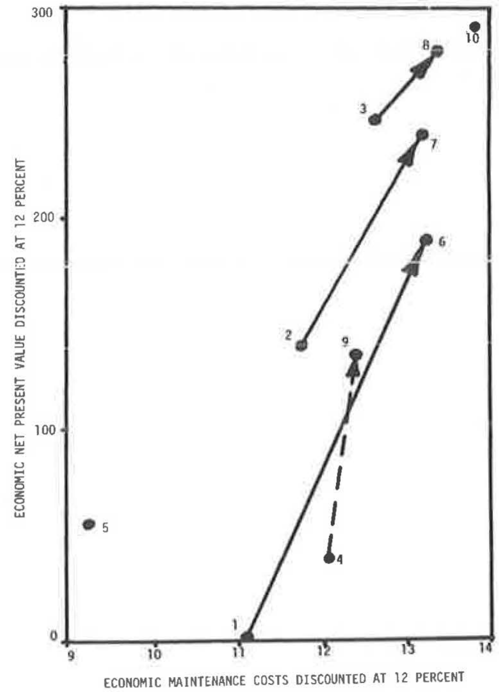
[b]Overlay thickness, mm.

Figure 3. Effects of overlay frequency on economic performance (million Egyptian pounds).



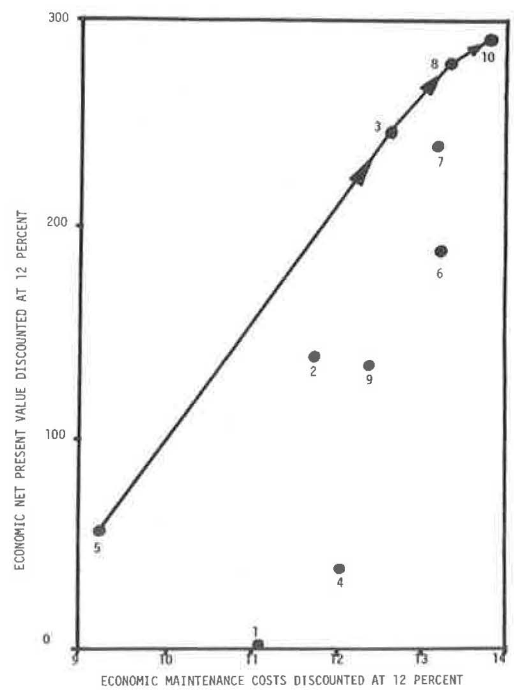
Note: Net present value is with respect to alternative 1, and includes road maintenance and road user costs.

Figure 4. Effects of overlay thickness on economic performance (million Egyptian pounds).



Note: Net present value is with respect to alternative 1, and includes road maintenance and road user costs.

Figure 5. Economically "efficient" maintenance alternatives (million Egyptian pounds).



Note: Net present value is with respect to alternative 1, and includes road maintenance and road user costs.

Table 3. Economically "efficient" maintenance policies.

Policy	Net Present Value[a]	Overlay Frequency Class I	Class II	Class III
5	56	3000[b] (40)[c]	Never	Never
3	246	3000 (40)	4000 (40)	5000 (40)
8	280	3000 (40)	4000 (60)	5000 (80)
10	293	3000 (40)	6000 (100)	7000 (120)

Note: 1 Egyptian pound = US $1.40.

[a] Million Egyptian pounds discounted at 12%; includes road maintenance and road user costs.

[b] Overlay roughness threshold, mm/km.

[c] Overlay thickness, mm.

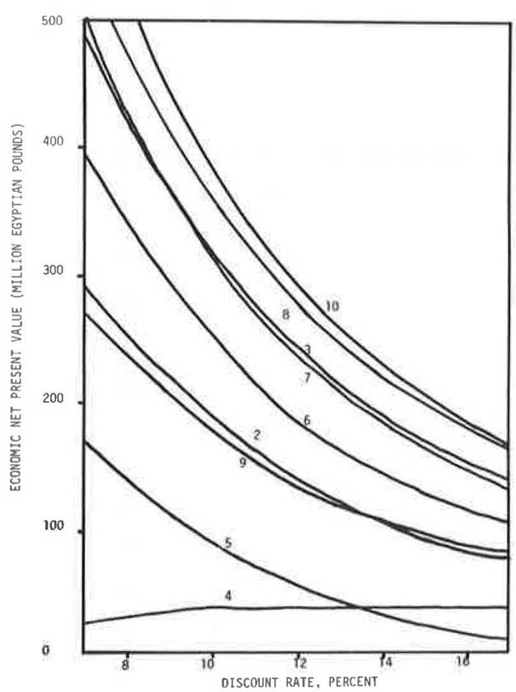
Table 4. Economic performance summary (million Egyptian pounds discounted at 12%).

Policy	Maintenance Costs	Vehicle Operating Costs	Total Costs	Net Present Value[a]	Net Consumer Surplus[b]
1	11.13	1867	1878	0.0	0.0
2	11.73	1727	1739	139.1	137.0
3	12.65	1619	1632	246.3	254.4
4	12.07	1829	1841	37.6	35.8
5	9.24	1813	1822	56.0	11.6
6	13.27	1675	1688	189.8	190.9
7	13.18	1625	1638	240.3	243.8
8	13.41	1585	1598	279.8	292.8
9	12.43	1729	1741	136.9	130.2
10	13.86	1571	1585	293.0	295.9

[a] Net present value is with respect to Policy 1.

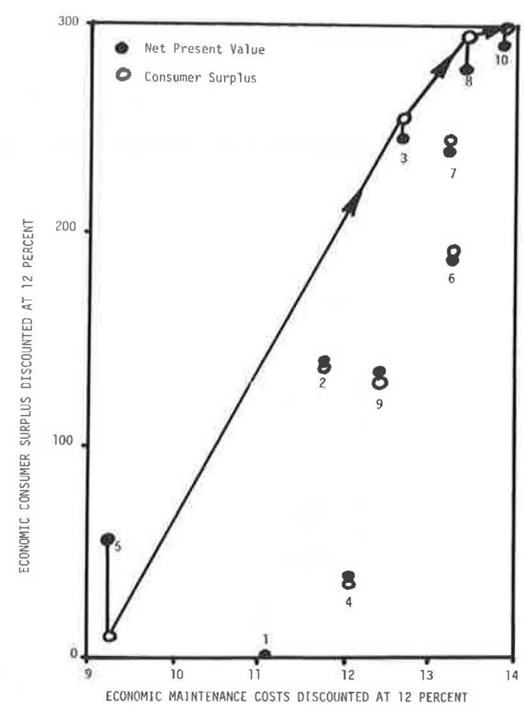
[b] Net consumer surplus is with respect to Policy 1.

Figure 6. Economic net present value as a function of discount rate.



Note: Net present value is with respect to alternative 1, and includes road maintenance and road user costs.

Figure 7. Economicall "efficient" maintenance alternatives from consumer surplus criterion.



Note: Net present value and net consumer surplus are with respect to alternative 1.

alternatives. Figure 7 shows both the discounted net present value and the discounted net consumer surplus of all alternatives (measured against Alternative 1) plotted against the discounted total maintenance investment, discounted at 12 percent. From the figure, we can see that the relative ranking of the various alternatives is largely unchanged, and that the same set of maintenance alternatives which is economically "efficient" is also the efficient set from the consumer surplus criterion. All of the non-uniform investment alternatives (4, 5 and 9) perform less well under the consumer surplus criterion than under the strict economic efficiency criterion, most notably Alternative 5, which is to invest only in the primary links. This indicates that, from a user satisfaction point of view, stressing one road class over another is highly disadvantageous, and that a uniform level of investment is to be preferred. Although policies with maximum investment in the primary system, which handles at a minimum 73 percent of the total demand, are still dominant, those policies which also incorporate heavy investments in the secondary and tertiary networks (Alternatives 3, 8 and 10) produce much higher overall performance, as measured both by the economic efficiency and user satisfaction (consumer surplus) of the system.

Returns on Maintenance Investments

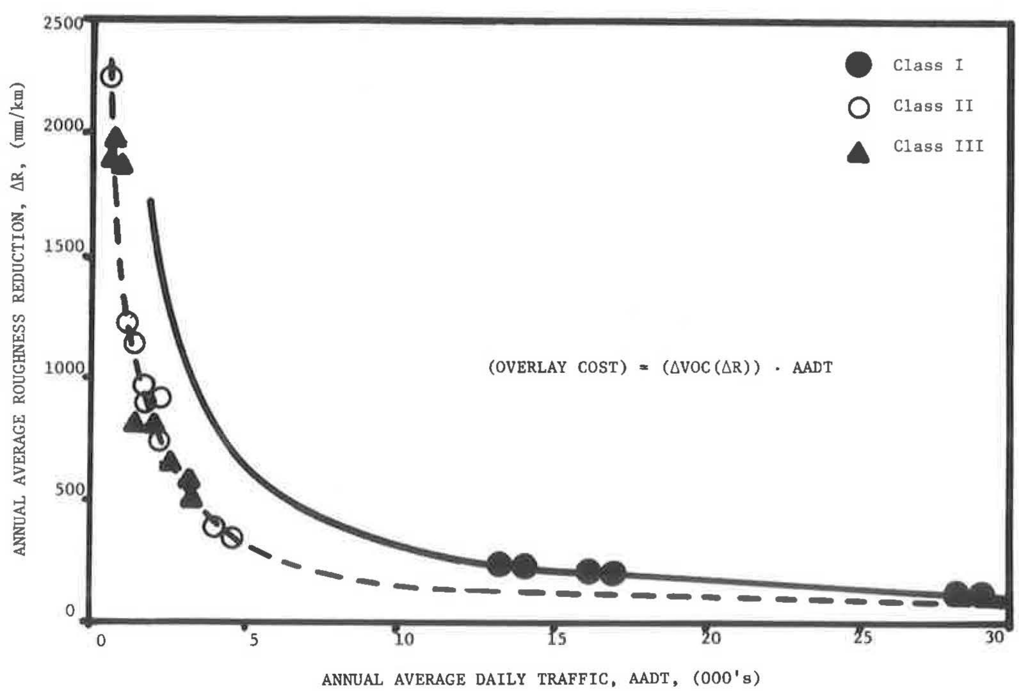
To explain the huge returns in the road network to the proposed maintenance investments, it is useful to study Figure 8. The figure shows the average roughness reduction in mm/km which must be maintained over a one year period such that the resulting vehicle operating cost savings equal the cost of a 40 mm overlay. One can see why the "economically efficient" set of maintenance alternatives includes the heaviest investments in the primary links, since even an average annual roughness reduction of 500 mm/km will pay for itself in less than one year (the overlay threshold of 3000 mm/km means an average reduction of approximately 1500 mm/km, since the model assumes that an asphalt concrete overlay restores the riding surface to a roughness level of 1500 mm/km). Figure 8 was derived on an annual basis so that the results would not be affected by the chosen discount rate. From the graph, we can see that the secondary and tertiary links require either a greater roughness reduction or a longer time period within which to accumulate benefits in order to achieve the same level of return as is available in the primary system, requiring either frequent, light overlays to maintain the riding quality or infrequent, heavy overlays to reduce the rate of road deterioration. The economic returns of both of these policies have been discussed previously.

Conclusions

From the above discussion, the following summary can be made. A highway transportation planning framework developed at M.I.T., the Road Investment Analysis Model, has been successfully applied to the analysis and planning of highway network investment decisions in Egypt. The disaggregate origin-destination survey data was used to calibrate the probabilistic traffic assignment diversion parameter to Egyptian conditions.

Ten alternative maintenance policies were analyzed over the network study zone, reflecting three types of investment concerns important to the Egyptian Transport Planning Authority. First was the question of the relative frequency of the maintenance activities, in this case asphalt concrete overlays, which affect the mean road surface condition of each road class. Second was the relative magnitude of the investments as reflected in overlay thickness, which affects the rate of future deter-

Figure 8. Average roughness reductions as a function of average annual daily traffic such that the annual road user cost savings equal the cost of a 40 mm overlay.



ioration of the riding surface in each road class. And third, was the question of the relative levels of investment among the three road classes - primary, secondary, and tertiary - to achieve the most effective overall investment strategy in the network.

The economic analysis indicated that the more uniform the investment levels among the road classes, the higher the economic performance of the network. Additionally, the heavier the level of investment, the greater was the economic return from that investment, given that the investment was distributed fairly evenly among the various road classes. The best alternative analyzed was found to consist of frequent, light overlays on the primary links, which as a corridor represent at least 73 percent of the total traffic demand, and infrequent, heavy overlays on the secondary and tertiary links. However, since this alternative was only marginally better (considering all alternatives) than a policy of frequent, light overlays throughout the system, we conclude that the optimum maintenance overlay policy would consist of frequent, light overlays on the primary system (where additional structural strength is not required) and initial heavy overlays followed by frequent, light overlays on the secondary and tertiary systems (where additional structural strength is needed to meet future traffic demands). These conclusions held true whether the system was judged on strict economic efficiency or on user satisfaction, as measured by the consumer surplus criterion.

In considering the above conclusions, one must bear in mind the limitations of the model, which does not include the effects of congestion, roadway occupancy during maintenance and rehabilitation operations, motorized/non-motorized vehicle interaction, or maintenance resource or capacity constraints. However, the predicted economic returns seemed so large that these considerations would probably not alter the general findings of the analysis.

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EVALUATION OF HIGHWAY ROUGHNESS IN BOLIVIA

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Highway roughness measurements were made in Bolivia in the Districts of La Paz, Cochabamba, and Santa Cruz (1) using a Mays Ride Meter. The work was sponsored by the International Bank for Reconstruction and Development (IBRD) as part of a pavement maintenance project with the Servicio Nacional de Caminos, Bolivia (7). Several measurement results are presented. These include Mays Meter calibration results using the TRRL pipe course, both prior to and after completion of a large survey. Roughness survey results are compared to results obtained in Kenya. Roughness versus speed information taken during the calibration and roughness measurements from paved, gravel, and earth roads are presented. The effectiveness of motor-grader operations in reducing roughness is discussed. A preliminary roughness scale developed for use in Bolivia is presented and compared with a Kenya roughness scale developed by the IBRD (8). Conclusions drawn from the paper include the establishment of tentative roughness limits for roads in Bolivia and similar countries. The measurements obtained were successfully used with IBRD maintenance cost models, thus assisting the maintenance programming in Bolivia. Since the completion of the work presented in this paper, further measurements have been made for planning and study purposes.

Background

The International Bank for Reconstruction and Development (IBRD) uses models developed from a Kenya study to relate user cost to road roughness (8). Unfortunately there are significant problems involved in determining road roughness in other countries and scaling the resulting values to the Kenya equations. The purpose of this study was to establish compatibility between various methods of evaluating pavement roughness on a world-wide basis. Texas Research and Development Foundation (TRDF) conducted this study in 1978 for IBRD and the Servicio Nacional de Caminos (SNC) of Bolivia.

The objectives of the project were:

1. To provide a basis for development of a general roughness index (7) which can potentially be used for world wide roughness comparison,
2. To obtain roughness data on selected section of Bolivian highways for input into IBRD maintenance cost models, which use a Kenya-based roughness index,
3. To measure the effectiveness of motor-grader maintenance operations in smoothing gravel and earth roads,
4. To establish a roughness measuring capability for SNC use in Bolivia, and
5. To train SNC personnel in the installation, calibration, and use of roughness measurement equipment.

A Mays Ride Meter with two different roughness devices was used in the study; (1) an electronic digital system and (2) an electromechanical paper output system. The paper output system is manufactured by Rainhart Company (2), while the electronic digital system has been developed on an IBRD project in Brazil (3, 4, 5, 6). The Mays Ride Meter paper output contains three traces; (1)pavement roughness, (2) an odometer distance marking, and (3) a special event trace controlled by the operator. The paper chart flow is a function of the roughness measured and this feature allows roughness to be summed continuously. The electronic summary equipment uses the Mays Meter roughness transmitter manufactured by Rainhart. However, it uses a digital distance measuring instrument (6) which sums roughness over 50 m (165 ft.) or 200 m (660 ft.) intervals (3, 4, 5). The Mays Meter transmitter is mounted in the rear of the vehicle over the differential. The measurement vehicle for this study was a 1976 Chevrolet Suburban, which has a relatively stiff suspension.

The primary scope of the project included sampling a total of 1300 km (780 mi.) of roadway, approximately one-third of which was in each of the three Districts: La Paz, Cochabamba and Santa Cruz, as shown in Figure 1. Table 1 shows a breakdown of the sites according to location and road type. Roughness measurements were also made on fourteen unpaved roads prior to and after the passage of a standard motor-grader. Calibration sections were established in Bolivia to facilitate maintenance and calibration of the roughness measurement devices. A Transport and Road Research Laboratory (TRRL) pipe calibration section (8) was established on a portland cement concrete road at the El Alto Airport in La Paz. Calibration sections were also established on in-service roadways in all three districts.

Equipment Calibration

Initially the Mays Ride Meter was calibrated on the TRRL calibration strip which was established in La Paz. The joint spacing of this pavement is 6 m (20 ft.), and the pavement is 10 cm (4 in.) thick with .635 cm (.25 in.) diameter mesh steel reinforcement. There is a granular base of varying thickness and a granular subgrade soil with little plasticity. Thus, the roughness of the basic section should remain stable for many years.

The development of the pipe course is reported by Abaynayaka et al, (8). Measurements are taken for six different states or levels of roughness simulating virtually the full range of roughness, from a paved road in good condition to an unpaved road in extremely poor condition. The procedure consists of establishing a permanent test section on a rigid concrete pavement with a number of pipe segments (from 0 to 150) bolted across the vehicle wheelpaths at varying intervals to simulate varying stages of roughness. The vehicle and instrument make three passes over the test section in each state of roughness to establish the calibration. The pipes are 2.2 m (7.25 ft.) long with an external diameter of 33.9 mm (.38 in.). The six different states of roughness are given as follows: (Note - 1 m = 3.3 ft.)

Stage I No pipes.

Stage II 25 pipes, placed at points 0, 12, 24, etc., to 288 m.

Stage III 25 additional pipes, placed at 8, 20, 32, etc., to 296 m.

Stage IV 25 additional pipes, placed at 4, 16, 28, etc., to 292 m.

Stage V 39 additional pipes, placed at 2, 6, 10 26, 30, 34, 50, 54, 58, etc., to 290, 294, 298 m.

Stage VI 36 additional pipes - to fill all spaces at 2 m intervals.

Table 2 summarizes the results from the calibration course with the electronic equipment, which was used in the sampling measurements due to its data presentation format. Initially (July 8), the calibration was done at 30 km/hr (20 mph), as recommended by TRRL. Subsequently (July 13), the device was calibrated at speeds of 20, 30, and 50 km/hr (12, 20, and 32 mph) to facilitate comparison of Bolivian measurements with measurements currently being made on a Brazilian project (3) and to establish speed correlations. The maximum percent difference between July 8 and July 13 30 km/hr (20 mph) calibrations was 11 percent. This is close, considering that on a given pavement it is normal to have variations of 10 to 15 percent in average roughness readings from one day to another. The measurement repeatability is also good: the average coefficient of variation of these measurements is 3 percent. On August 8, upon completion of sampling measurements, the electronic equipment was recalibrated on the TRRL calibration course. Those mean values are also shown in Table 2, with their percent change as compared to the calibrations of July 13. Again, the calibrations compare well, except for two 20 km/hr (12 mph) comparisons which are greater than 15 percent. This difference, however, is not abnormal because at slower speeds the roughness is more variable, depending upon wheel path, tire pressure, and weight of the measurement vehicle. However, comparison of the other calibrations indicates that the equipment remained in calibration for the measurement period.

Figure 2 shows the speed correction relationships developed for the different calibration speeds using the electronic device data. In general roughness decreased with speed. Also, at 50 km/hr (30 mph) the roughness-stage relationship is linear. These relationships were used to calibrate roughness measurements taken at one speed to equivalent roughness values for other speeds, which may have been impossible to take due to alignment, dust, etc. Additional roughness data taken at different speeds on in-service roadways are contained in Table 3, which also shows the magnitude of the coefficient of variation which may occur during the sampling of a given roadway. The data are plotted in Figure 3, indicating that, in general, roughness decreases with speed.

Because of the differences in data presentation by the paper and electronic devices, a method was established to transform the output of both systems into identical comparable units,. i.e., millimeters of pavement roughness per kilometer of pavement length.

Sampling of Inservice Highways

Thirteen paved roads, sixteen gravel roads and ten earth roads (1300 km, 780 mi.) were sampled. These were evenly distributed among the three Districts and were chosen by the SNC as representative examples of different highway types. Table 4 shows the results from the sampling of paved roadway sections. All paved roads measured were designated "primary highway" with the exception of two secondary roadways and one rural roadway. The mean roughness, standard deviation, and coefficient of variation of the measurements on these various pavements are included in Table 4, along with pavement type, condition of the pavement in terms of a present serviceability rating (PSR), and pavement age in years. The Autopista, currently under construction between La Paz and the El Alto Airport, has the minimum mean roughness measured, 231 mm/km (15 in./mi.). The maximum mean roughness, 3,000 mm/km (200 in./mi.), was measured on Route 4 in the Santa Cruz District, which is 30 years old and has severe surface distress in the form of Class 3 AASHO cracking, edge failures, and numerous potholes. Figure 4 shows the relationship between present serviceability ratings (PSR) and roughness for the paved sections. The large coefficients of variation result from the fact that the roads are highly variable and the mean value for each road was obtained by sampling along the highway over distances varying from 30 km (20 mi.) (Santa Cruz to Warnes) to 123 km (74 mi.) (La Paz to Sica Sica). The reason for these measurements was to determine the roughness of the typical pavements over their length.

Table 5 summarizes the mean roughness values for the gravel roadway sections sampled. In some cases, the measurement speed was less than the desired 50 km/hr (30 mph) due to poor visibility from dust, alignment, grade and/or traffic. Therefore, the roughness was measured at a lower speed, either 20 or 30 km/hr (12 or 20 mph) and the roughness value was converted to an equivalent roughness value for a measurement speed of 50 km/hr (30 mph). The mean roughness for gravel roads varied from a low of 4,393 mm/km (293 in./mi.), which is for a good gravel road, probably graded less than one week prior to the measurement, to a high value of 14,986 mm/km (1,000 in./mi.) on a gravel road containing extensive washboarding and poor aggregate size distribution. The mean roughness for primary roads was 9,000 mm/km (600 in./mi.). The mean roughness for secondary roads was 8,100 mm/km (540 in./mi.) and the mean roughness for rural roads was 12,060 mm/km (804 in./mi.).

Table 6 summarizes the sampling measurements made on the earth roads, generally at a speed of 30 km/hr (20 mph) due to problems with dust, alignment, grade and/or roughness. The mean roughness for earth roadways varied from 7,600 mm/km (507 in./mi.) to 16,660 mm/km (1,111 in./mi.).

Effectiveness of Motor-Grader Operations On Roughness

Measurements of the effectiveness of motor grading were carried out on gravel and earth sections. Table 7 shows the results of the grading experiments on gravel sections in the La Paz and Cochabamba Districts. As the before and after grading roughness measurements show, the roughness was improved on all sections except one. Three sections in the La Paz District, due to their proximity to the SNC office, were also measured one or two days later and twenty days later. Figure 5 shows that, in all three cases, within twenty days without maintenance these roads had returned to the condition observed before grading. In the case of Highway 1 near Laja, twenty days after the initial grading, with no additional maintenance, the road was worse than initially. This road is generally maintained more often than once in three weeks because it is a primary route.

Table 8 summarizes the grading experiments on earth sections. The roughness values in Table 8 are for a speed of 30 km/hr (20 mph) because most of the measurements on these earth roads were made at 30 km/hr (20 mph) or less. The earth roads had a much larger percentage reduction due to grading than did the gravel roads. In the case of Highway 1355, for example, there is a 66 percent reduction, with a before grading roughness of 19,000 mm/km (1267 in./mi.) and an after grading roughness of 6,460 mm/km (431 in./mi.). However, it must be understood that these roads will not maintain the after grading roughness for a long period of time.

The grading studies have shown that grading significantly lowers in the roughness of gravel and earth roadways. It is hoped that use of the roughness measurement equipment will assist SNC maintenance forces in determining schedules for grading equipment and other maintenance operations.

Development of Bolivia Roughness Scale

Roughness measurements from the paved, gravel, and earth roads were summarized to obtain the overall range of roughness measured in Bolivia. The 30 km/hr (20 mph) mean roughness values were used to allow comparison with the Kenya scale(8). The mean roughness values varied from a low of 231 mm/km (15 in./mi.) on the new Autopista to a high of 22,000 mm/km (1467 in./mi.) on a gravel road with extremely poor aggregate gradation.

The TRRL roughness measurements in Kenya were made with a towed fifth wheel bump integrator (Similar to a B.P.R. Roughometer). Table 9 shows roughness values for both Bolivia and Kenya for various types of roadways. The overall range of roughness measured in the Kenya study was 1,429 to 20,600 mm/km (95 to 1,374 in./mi.). This roughness range corresponds closely to the roughness range measured in Bolivia at a comparable speed but with a different instrument.

The mean pavement roughness in Bolivia in 1,875 mm/km (125 in./mi.) with a standard deviation of 998 mm/km (67 in./mi.). The mean gravel road roughness in Bolivia is 13,850 mm/km (924 in./mi.), with a standard deviation of 4,631 mm/km (309 in./mi.). The mean roughness of earth roadways is 11,465 mm/km (765 in./mi.), with a standard deviation of 3,892 mm/km (260 in./mi.). When these data are compared to average Kenya roughness data the mean pavement value for Bolivia, 1875 mm/km (125 in./mi.), compares closely with the mean for all pavements measured in Kenya, 1800 mm/km (120 in./mi.) for asphaltic concrete, 2400 mm/km (160 in./mi.) for new surface dressed, and 2700 mm/km (180 in./mi.) for old surface dressed. The gravel and earth mean roughness values for Bolivia, 13,850 and 11,465 mm/km (924 and 765 in./mi.), respectively, compare well with the range of roughness measured on gravel roads in Kenya, 5000 mm/km (334 in./mi.) for gravel roads in good condition and 10,000 mm/km (667 in./mi.) for gravel roads in poor condition.

Therefore, we concluded that the ranges of roughness, in units of millimeters per kilometer, from the Kenya and Bolivian measurements were comparable. While the two devices have different operational characteristics, differences in variability and repeatability, and different output, it may be fairly concluded that the data collected in Kenya and Bolivia could be used with other data to produce a generalized roughness index (GRI) (7).

Subsequent Measurements

In March, 1978, the SNC carried out a series of additional highway roughness measurements on roads representative of each highway type.

A Mays Ride Meter roughness electronic recording device (1) was used. The instrument, the vehicle, and the calibration section were the same as those used by TRDF in the 1977 study.

Roughness Measurements Objectives

The study had the following main objectives:

1. To determine the differences between gravel and earth road roughness in the dry season (TRDF measurements) and those in the wet season (SNC measurements).
2. To determine the effect of motor-grader passes in smoothing gravel and earth roads in the wet season and to compare it with the dry season effect.
3. To increase the existing amount of highway roughness data in Bolivia, in order to fix maintenance standards and minimum desirable road surface conditions for each highway category.

Equipment Calibration

Calibration was performed according to the TRRL calibration method, as described earlier in this paper. The Mays Ride Meter with an electronic recording device was calibrated at two different times March 3, 1978 before starting the field measurements and March 22, 1978 when the field measurements were completed. Calibration was carried out at speeds of 20, 30, and 50 km/hr (12, 20 and 30 mph). Table 10 summarizes the calibration results.

An analysis of Table 10 shows that the percentages of variation between the two calibration records are low and similar to those which were obtained from the TRDF calibration records. Therefore, we conclude that the TRDF and the SNC roughness measurement records are mutually comparable.

Regression analyses were done on these data in order to establish mathematical relationships between highway roughness measurements at 20 and 50 km/hr (12 and 30 mph) running speeds and 30 km/hr (20 mph) records. These curves and mathematical relationships are shown in Figure 6. Correlation coefficients are very high and standard errors are low, indicating that these relationships can be used to compare roughness records at different speeds. This capability is very important for the Bolivian conditions, since there are some roads which do not allow any speed higher than 20 km/hr.

Highway Roughness Sampling Measurements

The most representative highways of each type, including paved, gravel, and earth roads, were selected in the three different areas of the country: the highlands (rolling and dry terrain), the valleys (rugged terrain, medium moisture content), and the flats (low lands, sub-tropical and humid weather).

Most sections are the same as those selected for the earlier TRDF project (see Figure 1).

Paved Highways. Sampling on paved highways was carried out on 500 m (1650 ft.) long sections distributed throughout the whole length of each road considered. Table 11 shows the results of this sampling.

Gravel and Earth Roads. Results of roughness sampling measurements on gravel roads are shown in Table 12. Also shown are the data derived from the TRDF measurements on gravel roads, in order to compare wet season and dry season figures. These data indicate that rainy season road roughness is usually lower than dry season roughness on the same roads. There are only two roads for which data do not agree with this conclusion and both are roads where maintenance is very poor.

Effectiveness of Motor-Grader on Unpaved Roads Roughness Reduction. Some locations were selected for measuring surface roughness before grading, and immediately after grading. On some of these test sections, surface roughness was measured again after different time periods in order to observe its variation. Table 13 shows results of these roughness measurements, as well as those from TRDF in the dry season. These figures suggest that motor-grader passes in the rainy season achieve a smaller percentage reduction in roughness than in the dry season. However, the absolute roughness value in the rainy season is lower than in the dry season.

Summary

As a result of this project the Servicio Nacional de Caminos now has a pavement roughness measurement capability and trained personnel to undertake future studies. Roughness has been measured and summarized on representative paved, gravel, and earth roads of all classifications in three districts on a total of 1,300 km (780 mi.).

If the mean roughness plus one standard deviation is used as a point at which maintenance should be undertaken, the following limiting values should be criteria for the three roadway classifications:

1. Pavements - 2,873 mm/km (191 in./mi.)
2. Gravel - 18,841 mm/km (1,257 in./mi.)
3. Earth - 15,357 mm/km (1,024 in./mi.)

Based on these possible criteria, two paved sections fell outside the maintenance limitation while two gravel sections and two earth sections needed more blading at the time of measurement. These three limiting factors may, in fact, be too high. However, the amount of maintenance equipment available makes it difficult to grade the gravel and earth sections as often as desired. Some earth sections could be graded on a daily basis because of the heavy truck traffic present. Much of the roughness on gravel sections is due to the poor gradation of materials present.

Comparison of the Bolivian and Kenyan data indicates the feasibility of developing a generalized roughness index for world-wide use. This is a particularly desirable use for measurements such as those described. There is definitely a need to express roughness measurements in comparable terms so that data can be shared. The data obtained from

The data obtained from these measurements were successfully used by the IBRD in cost models (8) to evaluate the most effective maintenance expenditure on Bolivian roads. In addition, the measurements of motor-grader effectiveness on earth and gravel roads in both the dry and rainy seasons provide useful information concerning maintenance quality and scheduling.

The ability to schedule maintenance should be improved through the use of roughness measurement equipment. A measurement program can be undertaken and can help to determine the correct time intervals for grading on the various roadway classifications and to further define limiting criteria. It is shown by the motor-grader study that blading definitely lowers road roughness and a systematic blading program based on objective roughness measurements should help maintenance scheduling. Finally, the close comparison of 1977 and 1978 calibrations and measurements indicates the reliability of the Mays Meter equipment.

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Table 1. Location and Extent of Measurements

	La Paz	Coch.	S. Cruz	Total
Paved Roads				
Sections Sampled	4	3	4	11
Kilometers Sampled[a]	185	235	243	663
Calibration Sites	4[b]	2	1	7
Gravel Roads				
Sections Sampled	8	4	2	14
Kilometers Sampled	204	133	21	358
Grading Sections	4	3	0	7
Earth Roads				
Sections Sampled	2	3	5	10
Kilometers Sampled	30	21	198	249
Grading Sections	1	2	4	7
Summation				
Sections Sampled	14	10	11	35
Kilometers Sampled	419	389	462	1270
Grading Sections	5	5	4	14

[a] 1 km = 0.62 mile

[b] Includes airport road/TRRL calibration section

Table 2. TRRL Calibration Course Roughness (1977)

Stage[a]	Measurement Speed km/h	July 8 Electronic Mays Meter Device $\bar{x}$ mm/km	July 8 σ mm/km	July 13 Electronic Mays Meter Device $\bar{x}$ mm/km	July 13 σ mm/km	Percent Change,[b] %	August 8 Electronic Mays Meter Device $\bar{x}$ mm/km	August 8 σ mm/km	Percent Change.[c] %
	50			7493	84		7745	670	3
6	30	10950	369	12271	369	11	12439	302	1
	20			16898	201		20217	17	1
	50			6806	134		6848	134	1
5	30	10025	101	9338	17	-7	10653	553	12
	20			13646	503		15297	587	11
	50			5348	50		5544	50	4
4	30	6722		6639	184	-1	7628	17	13
	20			8684	67		10813	385	19
	50			4359	101		4442	134	1
3	30	5398	151	5432	17	1	5717	201	5
	20			6823	218		7812	34	12
	50			3135	67		3319	101	6
2	30	3721	67	3688	17	-1	3872	151	5
	20			4241	67		4526	17	6
	50			1877	101		1945	151	3
1	30	1660	134	1504	151	-9	1777	67	10
	20			1308	117		989	67	-24

[a] Stage 6 - 150 pipes, Stage 5 - 114 pipes, Stage 4 - 75 pipes, Stage 3 - 50 pipes, Stage 2 - 25 pipes, and Stage 1 - 0 pipes.

[b] July 13 compared to July 8.

[c] August 8 compared to July 13

Note: 1km/hr = .62 mph, 1mm = .04 in., 1 km = 0.62 mi.

Table 3. Roughness vs. Speed Data On Inservice Roadways

Highway Type	Speed, km/hr	Mean Roughness, mm/km	Coefficient of Variation, %
Paved	65	1092	44
	50	1346	45
	30	2286	25
Paved	50	2718	70
	30	3810	69
Gravel	80	3327	25
	65	6350	28
	50	8776	38
	30	15875	22
	20	12497	21
Gravel	50	12751	20
	30	12548	24
	20	12065	34
Gravel	50	8179	23
	30	14656	26
	20	21615	17
Gravel	50	4394	33
	30	7747	32
Earth	50	7290	31
	30	7671	39
Earth	30	17983	18
	20	23927	13
Earth	50	9525	34
	20	15469	25

NOTE: 1 km/hr =.62 mph, 1 mm = .04 in., 1 km = 0.62 mi.

Table 4. Roughness Values for Paved Sections

Highway[a] Route Number	Mean[b] Roughness, mm/km	S. D. mm/km	C. V. %	Pavement[c] Type	PSR[d]	Age, Years
1 (L)	927	305	33	3 ST	3.5	5
2 (L)	1029	406	39	3 ST	3.2	1
- (L)	231	76	33	JCP	4.0	0
107 (L)	1212	592	49	HMAC	3.0	5
- (L)	2108	146	7	JCP	3.5	8
4 (C)	1664	686	41	HMAC	2.4	30+
7 (C)	813	508	63	3 ST	3.0	7
401 (C)	1803	330	18	3 ST	2.3	3
4 (S)	2718	1905	70	3 ST	1.5	30+
4 (S)	2997	2083	70	3 ST	1.0	30+
7 (S)	1245	711	57	3 ST	2.8	10+
9 (S)	381	229	57	HMAC	3.3	2
5050 (S)	1067	864	81	3 ST	2.5	10+

a. (L)-La Paz, (C)-Cochabamba, and (S)-Santa Cruz

b. Measured at 50 km/hr (30 mph)

c. ST - surface treatment, HMAC - hot mix asphaltic concrete JCP - jointed concrete

d. Present serviceability rating PSR by R. F. Carmichael
NOTE: 1 km = .62 mi., 1 mm = .04 in.

Table 5. Roughness Values for Gravel Sections

Highway[a] Route Number	Mean[b] Roughness, mm/km	C.V.	Actual[c] Measurement Speed, km/hr	Highway[d] Type
1 (L)	8776	38%	50	P
3 (L)	7500	25%	20	P
3 (L)	6970	28%	30	P
3 (L)	12751	20%	50	P
106 (L)	7140	15%	20	S
107 (L)	9855	21%	50	S
108 (L)	12649	21%	50	S
1350 (L)	14986	22%	50	R
4 (C)	8179	23%	50	P
7 (C)	8001	34%	50	P
106 (C)	7990	31%	30	S
401 (C)	6630	17%	20	S
4255 (C)	5610	32%	20	R
4 (S)	10820	16%	50	P
501 (S)	4394	33%	50	S
5050 (S)	15596	11%	50	R

a. (L)-La Paz, (C)-Cochabamba, (S)-Santa Cruz

b. Roughness values for 50 km/hr (30 mph) speed.

c. If measurement speed is less than 50 km/hr (30 mph) speed adjustment curves were used to obtain roughness value.

d. P - primary road, S - secondary road, and R - rural road
NOTE: 1 km = .62 mi, 1 mm = .04 in.

Table 6. Roughness Values for Earth Sections

Highway[a] Route Number	Mean[b] Roughness, mm/km	C.V.	Actual[c] Measurement Speed, km/hr	Soil
1355 (L)	11390	31%	50	Clay
1551 (L)	8330	34%	20	Clay
4255 (C)	12410	19%	20	Rocky
4791 (C)	17983	18%	30	Organic with river gravel
4767 (C)	15011	14%	30	Organic with river gravel
9 (S)	16660	34%	50	Clay
5260 (S)	7747	29%	30	Sandy
5134 (S)	7600	34%	20	Sandy
5030 (S)	8500	36%	20	Sandy
5115 (S)	9017	64%	30	Sandy

a. (L)-La Paz, (C)-Cochabamba, and (S)-Santa Cruz

b. All roughness values given for 30 km/hr.

c. If measurement speed is different than 30 km/hr speed adjustment curves were used.

NOTE: 1 km = .62 mi., 1 mm = .04 in.

Table 7. Effect of Grading on Gravel Sections

Highway[a] Route Number	Before Grading[b] Roughness, mm/km	Before Grading[b] C.V. %	After Grading[b] Roughness, mm/km	After Grading[b] C.V. %	Actual[c] Speed, km/hr	Comments
1 (L)	17272	17	8306	11	50	Same day
			9627	12	50	24 hrs later
			8255	17	50	48 hrs later
			18288	10	50	20 days later
1 (L)	4318	27	2540	13	50	Same day
			3962	18	50	24 hrs later
			10262	13	50	20 days later
1350 (L)	13843	14	8839	19	50	Same day
			12929	18	50	20 days later
106 (L)	6630	11	5270	7	30	Alignment bad
106 (C)	7820	17	6460	20	20	Too rough for 50 kmh
4 (C)	8179	23	8369	21	50	No Difference
4255 (C)	5780	33	4250	34	30	Alignment bad

a. Districts are (L) - La Paz and (C) - Cochabamba.

b. All roughness values given for 50 km/hr speed.

c. If measurement speed is different than 50 km/hr speed adjustment curves were used.

NOTE: 1 km = .62 mi., 1 mm = .04 in.

Table 9. Comparison of Bolivia and Kenya Roughness Measurements

Road Type	Bolivia Roughness Measurements Mean, (mm/km)	Bolivia Roughness Measurements Range, (mm/km)	Kenya Roughness Measurements Mean, (mm/km)	Kenya Roughness Measurements Range, (mm/km)
Paved	1875	231 - 3600		
AC on Crushed Stone			1814	1429 - 2697
Surface Dressing on Cement Stabilized - New			2415	1801 - 2918
Surface Dressing on Cement Stabilized - Old			2694	1877 - 3557
Gravel	13850	7000 - 22000	5984	22000 - 20600
Good Condition			5000	
Poor Condition			10000	
Earth	11465	7600 - 18000		

NOTE: Kenya results are after Refs. 6, 8.

Table 3. Effect of Grading On Earth Sections

Highway[a] Route Number	Before Grading[b] Roughness, mm/km	Before Grading[b] C.V. %	After Grading[b] Roughness, mm/km	After Grading[b] C.V. %	Actual[c] Speed km/hr
1355 (L)	19000	29	6460	36	50
401 (L)	12580	54	10540	37	20
4791 (C)	18666	29	13668	21	30
9 (S)	14960	18	11628	19	30
5030 (S)	8874	39	6324	38	30
5134 (S)	14892	21	7344	32	30
5260 (S)	7480	32	4250	26	50

a. (L)-La Paz, (C)-Cochabamba, (S)-Santa Cruz.

b. All roughness values given for speed of 30 km/hr

c. If measurement speed is different than 30 km/hr speed adjustment curves were used.

NOTE: 1 km = .62 mi., 1 mm = .04 in.

Table 10. TRRL Roughness Calibration (1978)

TRRL Stage	Speed, km/hr	(a) 7/13/77 Mean Roughness, mm/km	(b) 3/03/78 Mean Roughness, mm/km	Percent Change of (a) and (b)	(c) 3/22/78 Mean Roughness, mm/km	Percent Change of (b) and (c)
	50	7493	7042	- 6	7142	+ 1
6	30	12271	11732	- 4	11960	+ 2
	20	16898	17034	+ 1	19414	+14
	50	6806	6385	- 6	6274	- 2
5	30	9338	9754	+ 4	9984	+ 2
	20	13646	13240	- 3	14838	+12
	50	5348	5174	- 3	5270	+ 2
4	30	6639	6794	+ 2	6846	+ 1
	20	8684	9122	+ 5	10296	+13
	50	4359	4133	- 5	4108	- 1
3	30	5432	5256	- 3	5026	- 4
	20	6823	6908	+ 1	7298	+ 6
	50	3135	3284	+ 5	3319	+ 1
2	30	3688	3406	- 8	3432	+ 1
	20	4241	4312	+ 2	4212	- 2
	50	1877	2002	+ 7	1924	- 4
1	30	1504	1808	+20	1734	- 4
	20	1308	1538	+18	954	-38

NOTE: 1 km/hr = .62 mi/hr., 1 mm/km = .0667 in/mi.

Table 11. Paved Highways Roughness (1978)

Highway[a] Route Number	Mean Roughness, 30 km/hr	Mean Roughness, 50 km/hr	Average C.V. %	Pavement[b] Type	Age Years
1 (L)	303	615	59.58	3 ST	5
2 (L)	770	1154	29.15	3 ST	1
107 (L)	608	984	71.64	HMAC	5
- (L)	1808	2052	22.09	JCP	8
4 (C)	185	442	35.29	3 ST	30+
4 (C)	618	995	51.91	HMAC	2
4 (C)	1357	1691	45.28	HMAC	30+
401 (C)	914	1295	26.26	3 ST	3
4 (S)	2784	2745	63.53	3 ST	30+
9 (S)	110	311	76.53	HMAC	2
9 (S)	112	314	53.01	HMAC	2

a. (L)-La Paz, (C)-Cochabamba, and (S)-Santa Cruz

b. ST - Surface Treatment, HMAC - Hot Mix Asphaltic Concrete, JCP - Jointed Concrete
NOTE: 1 km/hr = .62 mi/hr

Table 12. Gravel and Earth Roads Roughness In The Wet and Dry Seasons

Highway[a] Route Number	Surface Type	Dry Season Roughness[b] Mean Roughness, mm/km	Dry Season C.V. %	Wet Season Roughness[b] Mean Roughness, mm/km	Wet Season C.V. %	Roughness Change, %
1 (L)	Gravel	8776	38	5837	29	-33
3 (L)	Gravel	7500	25	3795	40	-49
3 (L)	Gravel	12751	20	4621	27	-64
106 (L)	Gravel	7140	15	3385	25	-53
107 (L)	Gravel	9855	21	7184	29	-27
1350 (L)	Gravel	14986	22	5866	25	-61
4 (C)	Gravel	8179	23	7164	26	-12
106 (C)	Gravel	7990	31	5811	21	-27
4255 (C)	Gravel	5610	32	5662	13	+ 1
4 (C)	Gravel	10820	16	6201	16	-43
501 (S)	Gravel	4394	33	3774	41	-14
1355 (L)	Earth	11390	31	5931	41	-48
4255 (C)	Earth	12410	19	10619	31	-14
9 (S)	Earth	16660	34	7676	53	-54
5260 (S)	Earth	7747	29	6089	69	-21
5134 (S)	Earth	7600	34	8280	36	+ 9

a. (L)-La Paz, (C)-Cochabamba, and (S)-Santa Cruz.

b. Roughness values given are for 50 km/hr on gravel roads and 30 km/hr on earth roads.
NOTE: 1 mm = .04 in., 1 km = .62 mi.

Table 13. Motograder Passes Effect on Reduction of Gravel and Earth Roads Roughness in the Wet and Dry Seasons

Highway[a] Route Number	Surface Type	Dry Season Measurements: Before[b] Grading Roughness, mm/km	Dry: Effect of Motor Grader Passes, %	Dry: Time Lapse Grading/ Measurement	Wet Season Measurements: Before[b] Grading Roughness, mm/km	Wet: Effect of Motor Grader Passes, %	Wet: Time Lapse Grading/ Measurement
1 (L)	Gravel	17272	- 52	Same day	5388	- 24	Same day
			- 44	One day		- 25	14 days
			- 52	Two days			
			+ 6	20 days			
1 (L)	Gravel	4318	- 41	Same day	6164	- 38	Same day
			- 8	One day		- 5	14 days
			+138	20 days			
3 (L)	Gravel				3795	- 13	Same day
3 (L)	Gravel				4575	- 18	Same day
106 (L)	Gravel	6630	- 21		3544	- 19	Same day
106 (C)	Gravel	7820	- 17		5568	- 14	Same day
						- 3	Eight days
501 (S)	Gravel				4885	- 1	Same day
9 (S)	Earth	14960[c]	- 22		4960[c]	- 30	Same day

a, Districts are (L) - La Paz, (C) - Cochabamba, and (S) - Santa Cruz

b, Roughness values given for 50 km/hr speed

c, Roughness values for 30 km/hr speed

NOTE; 1 mm = .04 in., 1 km = .62 mi.

Figure 1. Location of Roughness Measurements

Figure 2. Speed Correction Relationships

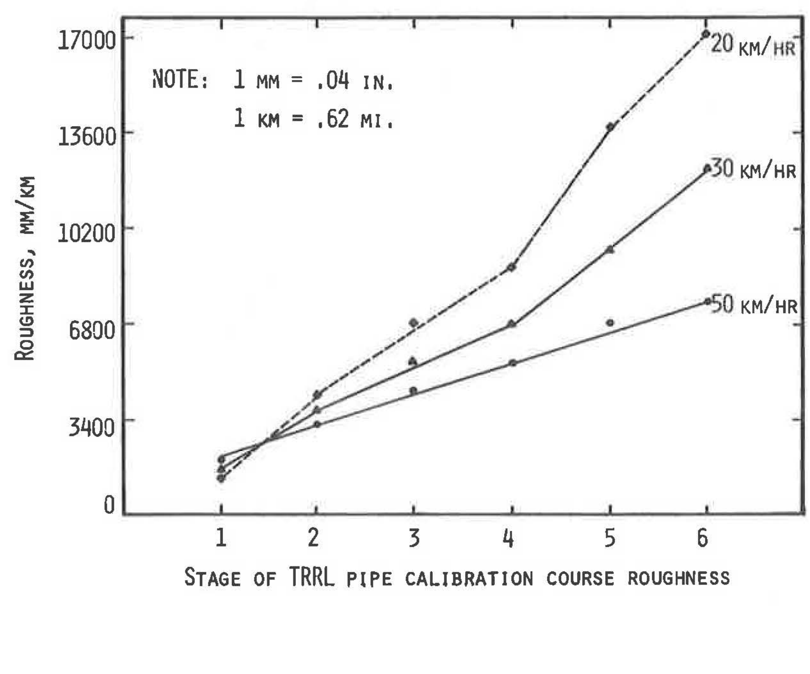


Figure 3. Roughness Versus Speed for Inservice Roads

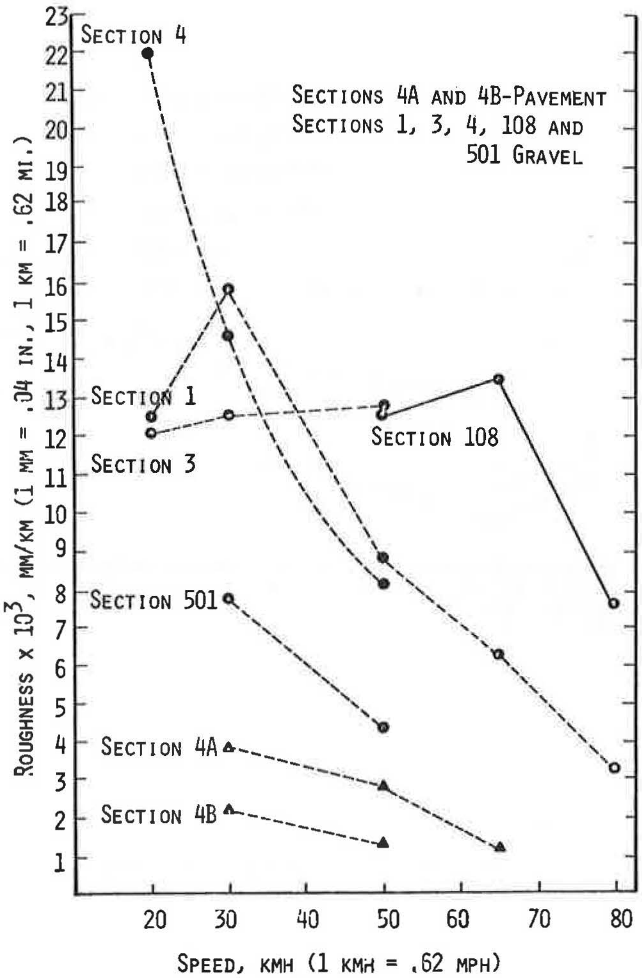


Figure 4. Present Serviceability Rating Versus Roughness for Paved Roads

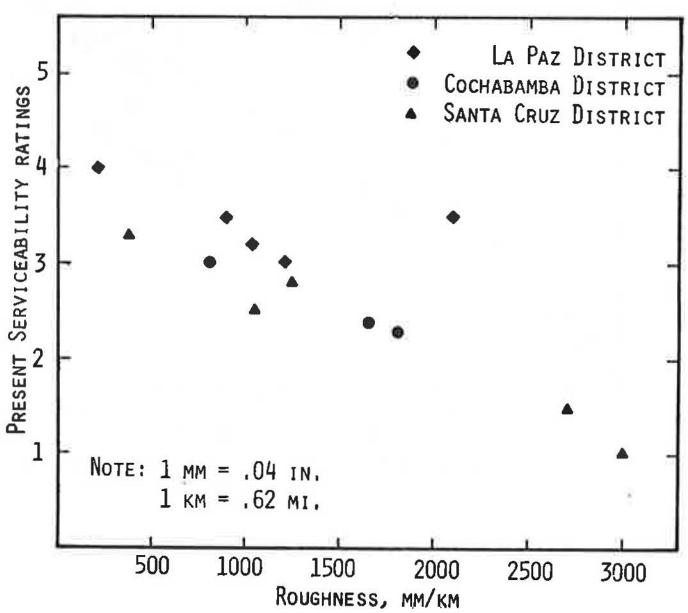


Figure 5. Roughness Versus Time for Gravel Grading Sections

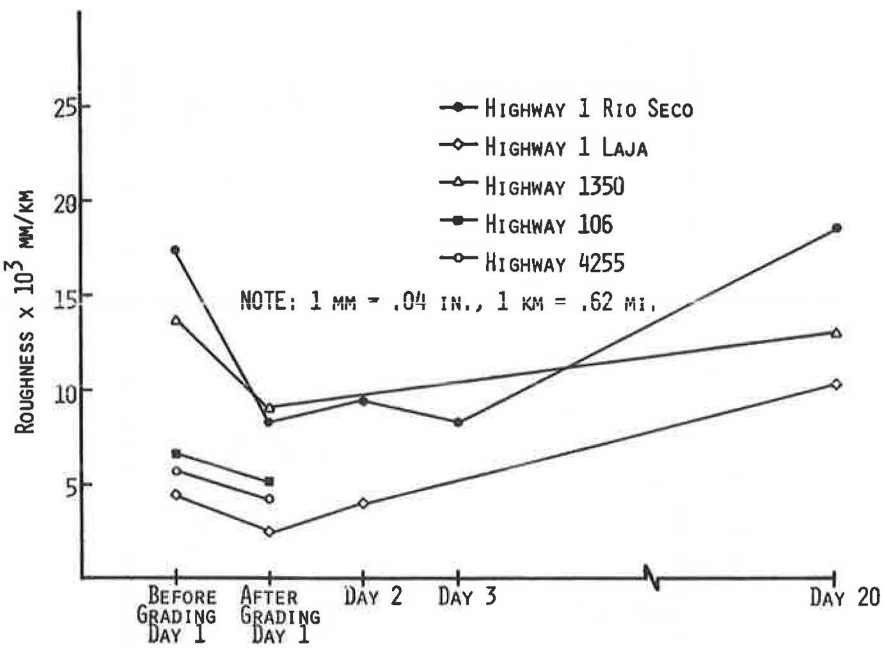
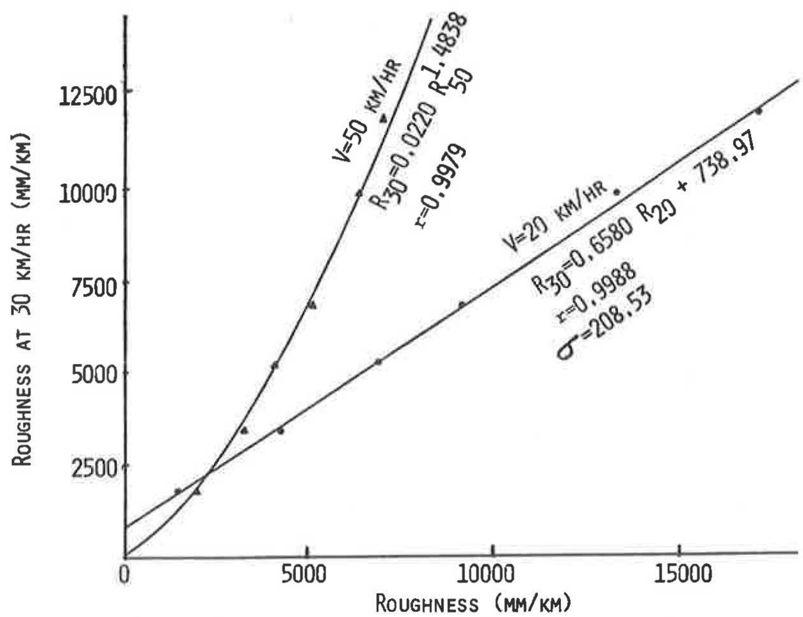


Figure 6. Roughness at 30 km/hr Versus Roughness at 20, 50 km/hr



NOTE: MEASUREMENTS TAKEN ON TRRL CALIBRATION COURSE.

OUTLINE OF A GENERALIZED ROAD ROUGHNESS INDEX FOR WORLDWIDE USE

W. R. Hudson, The University of Texas at Austin

The solution to the problems of providing uniformity in roughness measurements is not an easy one. No perfect answer exists, only a set of intelligent alternatives. It is vital, however, that some type of framework be set up so that coordination can begin. A multifaceted approach is proposed as follows:

1. Develop a Generalized Roughness Index (GRI) which has a sound basis and can provide a pseudo-standard for comparison of all methods.
2. Evaluate the use of an artificial calibration method (such as developed by the Transport and Road Research Laboratory) with a variety of instruments and cases to determine its value, problems, and utility.
3. Apply the concept of a standard rating panel to provide an additional methodology for defining and reproducing the GRI in countries all over the world without the cost of purchasing a stable profilometer, such as the General Motors device.
4. Evaluate the use of rod and level surveys and recommend field equipment to simplify and speed up such surveys for establishing calibration points on a GRI.

It is recommended that a GRI be implemented to test these concepts. Cooperation will be needed among several countries and agencies. Particular attention should be given to coordination of research data from the Kenya, Brazil and India projects, in which the World Bank is involved.

Background

One of the primary operating characteristics of a highway or pavement at any particular time is the level of service that it provides to its users. In turn, the variation of this level of service, or serviceability, with time provides a measure of the road's performance. This performance and the cost and benefit implications thereof are the primary outputs of a pavement management system. User costs are particularly related to road roughness on very rough roads. It was shown by Carey and Irick (5) in 1960 that road surface roughness was the primary variable needed to explain the driver's opinion of the quality of serviceability, or level of service, provided by a road surface, e.g., its desirability for use.

Road roughness can be thought of in many ways. some people talk about smoothness, others, serviceability. The Canadians use riding comfort and there are national committees in the United States to evaluate "riding quality." Still others talk of surface profile. In the European committees of PIARC, the Permanent International Association of Road Congresses, the English term "roughness" has come to be associated with surface texture and skid resistance or hydroplaning. Herein, roughness and smoothness can be defined as opposite ends of the same scale. A general definition of roughness must describe those surface characteristics of a road which affect the riding quality as perceived by the road user.

The availability of a roughness scale is important in terms of evaluating a road and its performance, but it is also very important in terms of evaluating vehicle operation and user costs. A common roughness scale for worldwide use regardless of the level of roughness, e.g., gravel surface or paved surface, is highly important.

Surface Roughness

Serviceability, or ride quality, is largely a function of roughness. Studies made at the AASHO Road Test (8) showed that about 95 percent of the information about the serviceability of a road is contributed by the roughness of its surface profile. That is, the correlation coefficients in the present serviceability, or PSI, equation studies improved only about 5 percent when other factors were added to the index (8). Francis Hveem discusses this problem in several papers. He states that "there is no doubt that mankind has long thought of road smoothness or roughness as being synonymous with pleasant or unpleasant." Road surface roughness is not easily described or defined, and the effects of a given degree of roughness vary considerably with the speed and characteristics of the vehicle using the pavement.

Roughness Defined

Road roughness is a phenomenon present in a road surface that is experienced by the operator and passengers of any vehicle travelling over that surface. Surface roughness is a function of the road surface profile and certain parameters of the vehicle, including tires, suspension, body mounts, seats, etc., as well as the sensibilities of the passenger to acceleration and speed. All of these factors undoubtedly affect the phenomenon of roughness. Safety considerations also influence our acceptance of roughness. Hudson and Haas (10) refer to "pavement roughness" as the "distortion of the pavement surface which contributes to an undersirable or uncomfortable ride." This definition refers to the road surface and divorces itself from other considerations. For purposes of this paper, this definition involving surface distortion will suffice in terms of "road roughness."

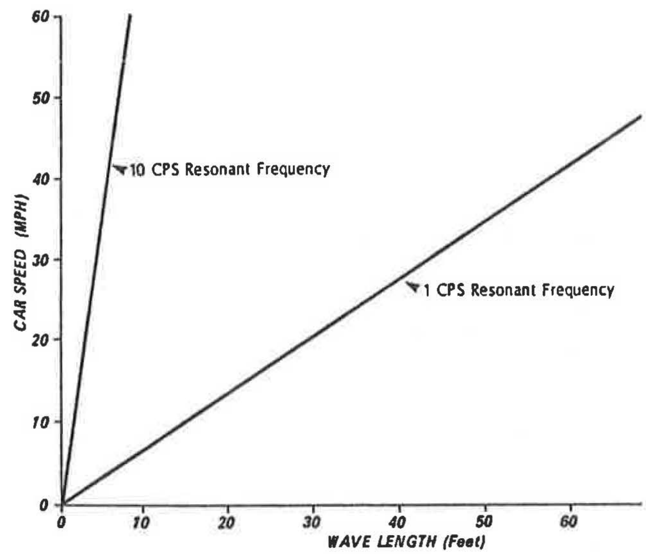
Components of Roughness

To define completely a roughness function some evaluation of the roughness of the entire surface area of the pavement should be made. However, for practical purposes this roughness can be divided into three components; transverse variations, longitudinal variations, and horizontal variations of pavement alignment. In other words, any functional roadway parameter which imparts accelerations to the vehicle or to the riders must be examined. More particularly of interest are those functions which influence the comfort and safety of the rider and/or the deterioration of the vehicle. Previous studies have shown that longitudinal roughness is probably the major contributing factor to undersirable vehicle forces (8). The next greater offender is transverse roughness (e.g., the roll component transmitted to the vehicle). The horizontal curvature of the roadway, which imparts yaw forces to the vehicle, is considered to be the least offensive and the one which is normally handled by following good highway alignment practices. Since most vehicles (approximately 70 percent) travel in a well-defined wheel path with their right wheel located approximately one meter (2-1/2 to 3-1/2 feet) from the outside lane line we conclude that measurements of longitudinal profile in the two respective wheel paths 1.83 meters (six feet) apart might provide the best sampling of roadway surface roughness. Furthermore, comparison between the two wheel paths can provide some measurement of the cross slope or transverse variations which are also important.

A rider in a vehicle passing over a road surface experiences a ride sensation. This ride sensation is a function of the road profile, the vehicle parameters, and the vehicle speed. A variation of any one of these three variables can make a rough road profile appear smooth or vice versa. Therefore, we might say that, from a passenger's viewpoint, roughness is an unfortunate combination of road profile, vehicle parameters, and speed. Riding characteristics of airplanes are also affected by the properties of the pavements and of the aircraft. Accelerations of sufficient magnitude to critically affect safety of aircraft operations are sometimes obtained over poor pavements.

Although some vehicles have hard suspension and others soft, the vehicle parameters (tires, suspension body mounts, seats, etc.) do not vary sufficiently to make a significant change in passenger comfort. With the limitation of relatively fixed vehicle parameters it is apparent that ride sensation is most dependent upon the car excitation generated by the various combinations of road profile and vehicle speed. Most drivers have experienced the sensation of either slowing down or speeding up to improve their ride on a particular road. This indicates that the road has a wave length content which, when driven over at a particular speed, produces an excitation in the vehicle at one of the vehicles resonant frequencies. The typical passenger car has resonant frequencies at between one and ten cycles per second. The relationship between wavelength, car speed, and car resonant frequency is shown in Figure 1. This relationship indicates that at many speeds there is a road wavelength that will cause an excitation at one of the car resonant frequencies. If the amplitude of that wavelength is large, the car ride will be noticeably affected.

Figure 1. Relationship between resonant frequencies of cars, car speed, and pavement surface wavelength.



In general, most passenger car ride characteristics are very much alike, and for any particular road most cars will be driven at about the same speed. With two of these variables held relatively fixed, the excitations into the car and thus the riding characteristics of the car become primarily a function of the wavelength content of the road profile surface.

Surface Roughness Evaluation

Roughness evaluation has received considerable attention from many highway and airport agencies in North America. Roughness is the primary component of serviceability and a large number of different roughness measures are in current use. This concept of preception by the highway user is important. This definition of roughness excludes surface texture and microtexture of surface aggregates since these are not perceived by the user to affect riding quality. Instead they affect skid resistance and other operational characteristics but will be excluded in this paper. The diameter of the surface stone used in pavement surface treatments which causes "noise," is discernible to the user, has an effect on the user's perception, and is roughness by this definition.

Surface Profile

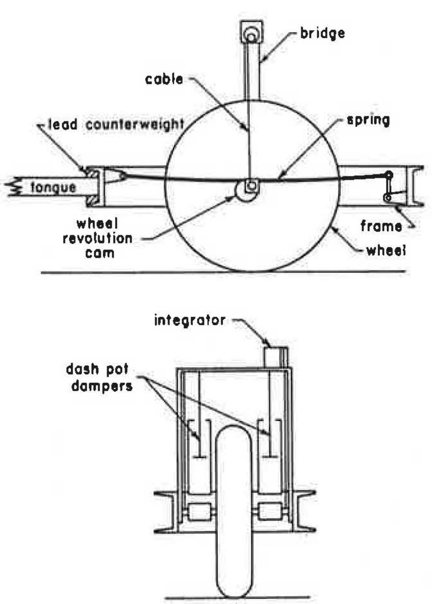
Many authors, such as Darlington (6) and Carey (3), feel that a surface profile is the best way to characterize roughness. In terms of profile, roughness can be defined as "the summation of variations in the surface profile." Profiles in this sense do not include the overall geometry in the road but are limited to wave lingths in the surface that are less than approximately 152.4 meters (500 feet) in length. In Darlington's terms, roughness is "the analysis of the profile or of the random signal known as profile."

Carey in (3) points out four fundamental uses of surface profiles or roughness measurements: as construction quality control tools, to locate abnormal changes in the highway such as drainage or subsurface problems, extreme construction deficiencies, etc., to establish a systemwide basis for allocation of pavement maintenance resources, and to identify pavement serviceability-performance histories.

In summary, a profile is a detailed recording of surface characteristics and roughness or smoothness is a statistic which summarizes these characteristics. Thus, roughness-smoothness is a statistic or number which summarizes the riding quality or surface profile of a road.

How rough is rough? Once the surface characteristics are summarized, it is essential to establish a scale for this statistic or summary value. This can be done in many ways, as pointed out by Darlington (6). Traditionally there are two ways of determining this statistic; mechanical integration and mathematical integration or analysis. The first of these methods is the most common, that is, the use of some mechanical instrument or device, such as the BPR roughometer in Figure 2, to mechanically filter and summarize the data in a specified way. The second method involves recording the profile as faithfully as possible and then analyzing and/or integrating this profile mathematically with some standard mathematical procedure, such as that outlined by Walker and Hudson (31 and 32), Roberts and Hudson (24 and 25), and Darlington (6). The most common methods in current use for mechanical measurement and summary include the BPR Roughometer (15 and 16), the PCA Roadmeter (1 and 2), the Mays Meter (32 and 33), the Chloe Profilometer (4), and the land plane or Profilograph (rolling straight edge) (28).

Figure 2. Schematic Diagram - BPR Roughometer



A number of studies have been made to compare these instruments and a number of references are available, including (6, 11, 16, and 22).

Since so much has been written about the various instruments available, we will not attempt in this short paper to review all these measurement methods in detail. See (14, 16, and 39) for details.

Comparison of Measurement and Summary Techniques

Regardless of the measurement and type of summary techniques used, it is essential that a good reference be established and maintained. It is equally important that accuracy be maintained in summation.

Darlington (6) points out that three basic reference methods have been used historically to measure roughness: the so-called rolling straight edge or land plane, as illustrated in Figure 3, the inertial mass as used in the BPR Roughometer, illustrated in Figure 2, the Mays meter and the PCA meter which the automobile serves as the inertial mass and, finally, an inertial reference profilometer, such as the Surface Dynamics or General Motors Profilometer, where an external reference is provided.

Figure 3. Land plane roughness device sometimes called Profilograph or rolling straight edge.

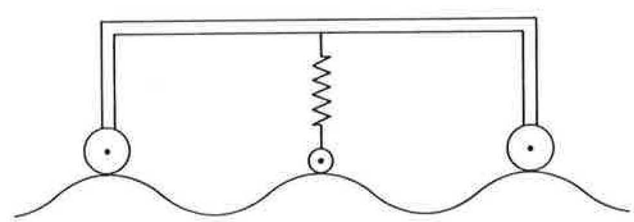


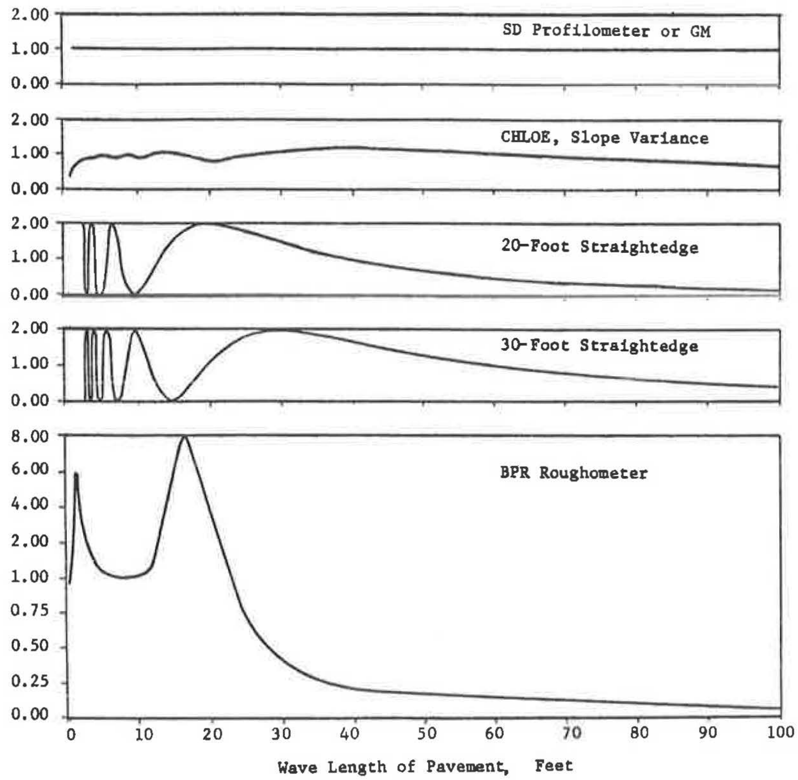
Figure 4 illustrates by means of a Bode plot the transfer function or response of several types of instruments to the input of road roughness. The problem is that the straight edge or land plane device is so erratic in its response that it is relatively useless. This is illustrated in Figure 4, where the effect of roughness wavelengths which are any multiple of the length of the straight edge results in zero output from the device.

Darlington simulated the response of the BPR roughometer (or vibrometer, or seismic reference device) on an analog computer using measured physical characteristics of the instrument. His analysis shows that the roughometer type device yields reasonable results for wave length in the region of approximately 1.22 to 4.26 meters (4 to 14 feet). Wave lengths in the range of 4.26 to 5.48 meters (14 to 18 feet) are badly distorted, and wave lengths beyond 6.70 meters (22 feet) rapidly attenuate to zero response.

The need for compatibility or Generality

As outlined above, diverse measurements of roughness are used around the world. It is not feasible to talk of equality among these measurements since it is not possible to provide compatibility among the various measuring systems if proper consideration is given. This compatibility involves two levels of concern: "External" compatability -- relating to whether the results of one agency's or country's work has quantitative relationship or meaning to those of another agency, and "Internal" Compatibility -- relating to correlating results, achieving repeatability, etc., within an agency.

Figure 4. Theoretical differences between SD Profilometer, Chloe, rolling straight-edges and seismic roughometer



This second aspect of compatibility is well illustrated by the Brazil Project (18) for it is essential that measurements made in all parts of Brazil be compatible with each other even though it is not possible to make all the measurements with a single instrument.

The problem of external compatibility is best illustrated by the fact that results of studies in Kenya can be compared to the findings in Brazil only if there is compatibility between the two sets of roughness data. I feel this can best be accomplished by establishing a "generalized roughness index" which can be used as a compatible base of comparison. This is preferable to selecting any particular measurement system, which itself may be changing and which may not be available to a particular potential using agency.

If a Generalized Roughness Index (GRI) is used, the the matter resolves to one of providing some way of determing the GRI in any particular instance.

In his opening remarks to a National Conference on roughness measurements and correlation in 1972, Mr. W. N. Carey, Jr., Executive Director of the U.S. Transportation Research Board speaks to these problems (3).

> A third use of profile measurements is to establish a systematic statewide basis for allocation of pavement maintenance resources. A word of caution here is in order. In the interest of finding low-cost tools that can be easily available to each highway department district, there is a tendencey to suggest highly simplistic devices. I believe that reliance on these devices may lead to serious mistakes in the development of priorities for maintenance expenditures. . .

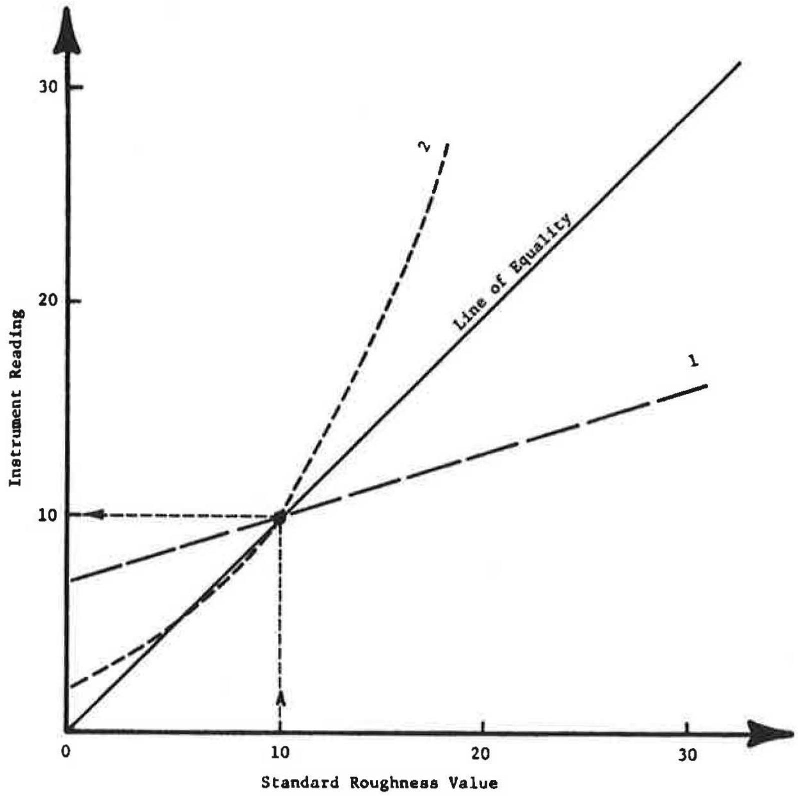
Carey's comment can easily be extended to include low-volume and unpaved road planning in developing countries where roughness can be used not only for allocating maintenance resources but also for ascertaining and considering user and vehicle operating costs. Although the absolute accuracy required for these various purposes may differ in all cases, relative accuracy and compatibility are important.

History of Roughness Calibration and Correlation

The earliest roughness measurements were reported by Hogentogler, as far back as 1923 (42) and early development of the Roughometer was reported in 1926 (19). Even in these early developments the need for calibration was readily recognized. From 1941. when the BPR Roughometer became "standardized," the Bureau of Public Roads (now the Federal Highway Administration) maintained a "standard calibration section" for testing any new or modified BPR Roughometer. It was observed from the beginning that instruments manufactured as nearly alike as possible did not record the same roughness value for the same pavement. The fallacy of a single calibration section is discussed by Hudson and Hain (15).

It is not possible to calibrate a dynamic instrument at a single point over its range and expect the calibration to be satisfactory for use of the instrument over a full range of roughness. This is illustrated in Figure 5 where a standard roughness section with a value of 10 has been set up. We might assume that any other instrument which reads 10 would be calibrated to the standard value. In fact, this assumption is depicted by the solid "line of equality" in the figure. This line assumes that if an instrument reads 10, it is "calibrated" and thus will read 20 when the standard instrument reads 20, 30 when the standard instrument reads 30, etc. Alternatively line No. 1 illustrates a plausible case of a linear relationship where instrument No. 1 is

Figure 5. Single point BPR calibration problems.



calibrated to the standard instrument on the section with value 10. Without additional test points we would not realize that the slope of the calibration line is really different from the assumed line of equality. Dotted line No. 2 illustrates a more complex case of nonlinear relationship which would, of course, also be missed with the single point calibration. Some twenty-four state agencies had BPR Roughometers in use in 1960. Many of these devices have been calibrated by this one-point method and by no other method.

Roughometer Calibration Course - AASHO Road Test

As reported by Hudson and Hain (15) there was a need to use the Roughometer in the AASHO Road Test but it became obvious very early, with the AASHO Profilometer to compare to, that the BPR Roughometer was a variable instrument, difficult to keep in calibration. In our work at the AASHO Road Test we were not only involved in measuring the roughness of all pavements with the AASHO Profilometer and in developing and operating the BPR Roughometer, but also in checking and calibrating at least six roughometers from states such as Michigan, North Dakota, Minnesota, and Wisconsin which brought their instruments to the Road Test for calibration against the AASHO Profilometer for determining serviceability.

Basically the method involved the installation of aluminum bars on the surface of a smooth rigid pavement to establish four separate test sections of different but known roughness. The roughometer could then be checked against the standard sections at any required time.

TRRL Pipe Calibration Course

Another artificial calibration technique has been proposed and used by the Transport and Road Research Laboratory in England. This concept appears to have promise for use as a calibrating device or standardization method around the world. A short note on the moethod is presented in (39). Briefly, the method involves the selection of a smooth pavement section approximately 300 meters (985 feet) long as a standard. This smooth section becomes the smoothest section in a series of 6 calibration sections. Subsequently rougher sections are created by adding artificial bumps to the surface of the standard sections by means of pipes with external diamter of 3.413 centimeters (1.34 inches). A total of six levels of roughness are created. Thus, the problem of one-point calibration is alleviated and yet the calibrating agency need find only one smooth, relatively unchanging pavement section. The absolute profile of this basic smooth standard section can likewise be checked with precise rod and levels on a quarterly or semiannual basis as necessary.

Use of a "Standard" Device for Calibration

Probably the most widely used method of calibration and correlation has been the use of some type of so-called standard device. Really this approach should be divided into two types. The first involves the selection of one replicate from the group of similar devices being used and the use of this replica only for calibration purposes so that it presumably does not "wear out." This is the approach that the BPR took with the check section as outlined earlier. I liken this approach to gold-plating a crowbar. If you have two dozen crowbars and select one of them because it appears to be more perfect in shape and weight than the others and plate it with gold, what do you have? Still a crowbar, albeit a shiny and expensive one. There is little evidence that this type of "standard" device has been successful in true calibration and correlation.

The second type of standard device involves the use of a master device which is itself calibratible or which has a standard of accuracy which is perhaps a magnitude greater than the other devices for which it is to be the master control. The AASHO Road Test Profilometer was such a device which became a standard against which dozens of Chloe Profilometers and BPR Roughometers were calibrated during and soon after the AASHO Road Test. This approach is discussed below as the Texas Calibration Course.

Use of Hydraulic Shaker Table

The General Motors Profilometer was originally developed for obtaining road profile input which could be fed into a vehicle ride simulator for testing vehicle suspensions at the General Motors Proving Ground (26 and 27). Some authorities feel that a similar approach can be used for inputting standard roughness to a machine in an analytically controlled manner to calibrate other devices. This method involves observing the responses of a measuring device in a laboratory with a servo-controlled hydraulic ram resting under each wheel. Known excitation is applied through the hydraulic rams to the vehicle to determine its response. More specifically, the wheels of the vehicle are vibrated by a shaker table in a manner to simulate operation of the vehicle on each of a set of standard test sections. Road profile data obtained with an instrument such as GM Profilometer are used to drive the shaker table. The profile data tape could be used for any number of successive recalibrations over any period of time and, in that sense, would not change.

There is, of course, some question about the correspondence between readings obtained by shaker table and roughness measurements obtained in the field. The major source of discrepancy remains in the fact that the vehicle is moving and wheels are rotating while measurements are being made in the field but not while operating on a shaker table. The dynamic vs. static tire conditions are of particular concern. At the present time the National Cooperative Highway Research Program is undertaking a research project which will undoubtedly investigate the shaker table approach to calibration of roughness devices (21). In general, this method does not seem possible for use worldwide since the shaker table is cumbersome and expensive. If a simple version could be devised it could be duplicated and purchased by interested goups but a great deal of research and development is required and we must await the results of the NCHRP study.

Texas Calibration Course

The Center for Highway Research and the Texas State Department of Highways and Public Transportation use the SDP or General Motors Profilometer as a master calibration device for a series of Mays Meters which are used routinely throughout the state. This approach is reported by Walker, Hudson and Williamson (32, 33, and 34). To some degree, a similar approach has been taken by the Michigan Highway Department, as reported by Holbrook and Darlington (12 and 13). The same approach is being taken at the present time in the UNDP Brazil Study (18). A SDP was purchased and is used for measuring a set of calibration sections. These sections are run regularly by eight Mays Meters to insure that their calibration remains stable. A control chart procedure and regular check procedure similar to that outlined by Williamson is followed (32, 33, and 34).

Basically, Texas maintains a group of 25 pavement sections which vary from smooth to rough. Every three months the profiles of all these sections are measured and analyzed with the SDP Profilometer. In this way, a set of pavements with known roughness are always available for use in checking and calibrating any other roughness instrument. Any instrument which appears to be giving erroneous readings is regularly run on several check sections and the values plotted on a standard control chart. If a device is "out-of-control" on three or four sections then it is thoroughly checked, mechanically repaired, and, if necessary, recalibrated.

Rod and Level Surveys

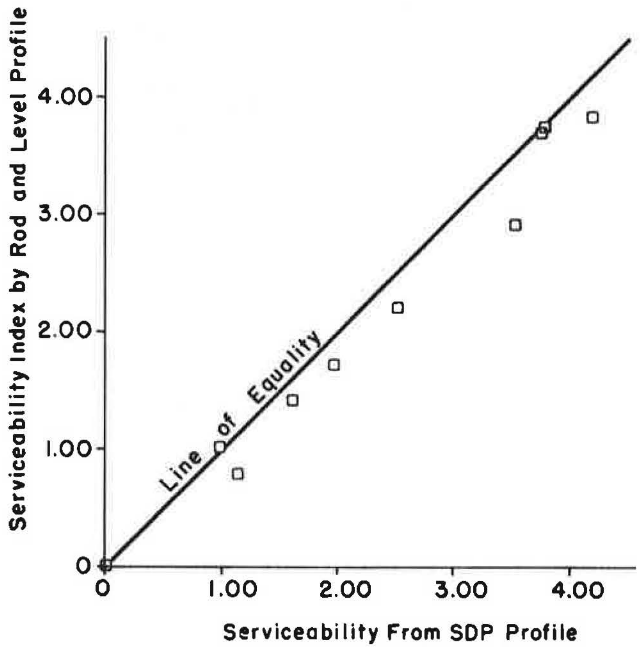
Many people feel that it is possible to establish vehicle roughness calibrations over standard pavement sections by running control rod and level surveys of the calibration sections to see if and how their profiles are changing. There are two basic problems associated with this methodology. First, the response of the vehicle and most roughness measuring instruments to a profile is an integration of everything the measuring instrument sees on the road surface. This is a continuous process and not one involving discrete points such as are used in a rod and level survey. This problem is magnified by the fact that even the best manual leveling techniques make it expensive to make measurements of test sections 300 meters (985 feet) long at spacings closer than about 1/2 meter (1.6 feet). Even in this case a total of 600 measuring points is required each time a calibration section is checked.

Perhaps more difficult than the accuracy and detailed problem outlined above is the need to integrate and/or summarize and anlyze the profile. To date, little has been done in this area. Recently we have investigated the use of second derivations of the profile to yield estimates of vertical accelerations present in the profile. A relationship has, in turn, been developed between vertical accelerations and SI.

Calculations are simple and do not require a large computer facility as is the case with existing profile analyzing methods such as power spectral density, Fourier transform, and digital filtering. Road profile root mean square vertical accelerations have a strong correlation with Mays Meter roughness readings as shown in the study by McKenzie and Srinarawat (40). Figure 6 illustrates a very good agreement in terms of serviceability index from 10 road surface profiles obtained by rod and level method and the Surface Dynamics Profilometer (SDP) (41). This plot also suggests that road profile data from rod and level and SDP are interchangeable and rod and level can be used to provide commonality among road roughness scales presently in use.

Certainly, these discrete rod and level surveys have some practical advantages, particularly in developing countries where labor-intenseive methods are economical. It might be far more practical to obtain detailed, discrete profiles with rod and levels of, say, ten or twelve pavement test sections on a regular basis than to maintain a high-technology, expensive electronic device for continuous profile measurements. Such a method will be practical if data analysis techniques can be developed and automated for easy use of the data.

Figure 6. Comparison of serviceability indices derived from rod and level profile and SDP profile.



Rating Panel Approach - Canadian Good Roads Association

Immediately following the AASHO Road Test, the Canadian Good Roads Association desired to put the findings of the AASHO Road Test into practice. In order to do this, they ran a rather complete survey of the existing roughness of their pavement system. They did not agree totally with the serviceability concept outlined at the AASHO Road Test and they chose to develop a Riding-Comfort Index Scale with values from 1 to 10. This index is basically an evaluation of pavement riding quality or roughness (7, 9, and 10).

After carefully establishing their Riding Comfort Index, a standard procedure was adopted using a small panel of well-trained raters to go from location to location evaluating the riding quality of these pavements and recording this riding quality in a data management system. A great deal of work has been done on rating scales and other subjective evaluations (5, 17, 20, and 24). There are some shortcomings to this approach, but it has the benefits of being practical, relatively inexpensive, and reasonably stable although its precision may be questioned. It certainly fulfills the concept and answers the question, however, raised by Carey in the quote referenced earlier in this paper. This approach deserves further consideration.

Standard Rating Panel

While it is not in present use, I believe that the concept of using a standard panel of pavement riding quality raters to establish a time-and-condition-stable standard roughness scale offers great promise as a practical solution. Yoder and Milhous (37) show in thier studies of rating panels and various instrumentation that rating panels of fifteen persons or more are quite stable in predicting pavement serviceability. Since roughness is so highly correlated with serviceability, there is little doubt that such panels would be equally stable in predicting pavement roughness. Carey and Irick (5) report similar results when comparing panels at the AASHO Road Test, as do Roberts and Hudson (24 and 25).

One major problem exists, what about panels from different cultures? For example, a panel from the United States rides predominately on paved roads. Can it rate accurately on the same scale as a panel from a developing country which rides predominately on gravel roads? How could this dichotomy be solved? Perhaps if as many as three members could be made available to participate in panel ratings in each of the major areas of the world, goegraphic and cultural stability could be evaluated.

This method would never have the precision or detail of physical calibration. However, it might be accurate in terms of insuring that different classes of road roughness are adequately separated.

The following section presents a discussion of the relative merits of these methods for use in establishing a General Roughness Index.

Possible Approaches for Calibration

Evaluation of the concepts for calibration outlined in the last section indicates that three basic methods have strong potential: the use of a shaker table to input artificial roughness in a laboratory, the use of artificial roughness calibration sections, and the use of standard road sections along with a method of evaluating the roughness of these standard calibration sections from time to time.

However, the practical limitations of the problem set forth in this paper apparently preclude any possibility of using a hydraulic shaker table with known roughness inputs to calibrate roughness devices. No such equipment is presently in use for this purpose, even in the United States, and the development and employment of such equipment in the field seems completely infeasible at this point. Therefore, the other two major approaches are discussed in further detail: artificial calibration sections and standard pavement calibration sections.

Artificial Roughness Course

The concept of introducing well defined artificial roughness onto a selected section of smooth pavement in identifiable stages follows the approach of Abaynayaka and TRRL. The approach is certainly feasible since any country in the world could develop at least one smooth, strong section of pavement to serve as the base section. They could then find several pieces of standardized pipe or other material approximately 2 meters (6.6 feet) long to introduce roughness. These two ingredients can be combined in several stages to provide up to six or even more test sections of increasing roughness. The method therefore warrants careful consideration.

The major problems associated with this method are the artificiality of the roughness introduced and the potential of generating resonance or harmonic motion in the measuring vehicle being calibrated. As indicated by the anlysis of Darlington (figure 4), the transfer function of a roughness measuring device is highly dependent upon the wavelength characteristics and amplitude of the roughness in the roadway surface. It yields reasonable readings for wavelengths in the range of 1.22 to 4.28 meters (4 to 14 feet) and it has two resonance frequencies at 0.61 meters (2 feet) and 5.18 meters (17 feet). The response of the instrument to step-inputs might be on the first peak present at very short wavelengths. If some type of resonance is generated in the system, say for roughness level six, then the multiplication amplitude could be even higher. It is entirely possible that the response of an instrument to the roughest calibration section would be, for example, a very large roughness number and yet the instrument might respond different to a very rough gravel road with natural potholes, etc. There is certainly also the possibility that the calibration course can be set up in such a way as to cover the range of interest for most very rough roads and thus to serve adequately as a calibration procedure. The only way to ascertain the answer to this question is to study the problem theoretically and to apply the concept in the field where an alternative method of calibration and checking, such as the SDP or General Motors Profilometer, exists for comparison. This type of comparison check is being made in Brazil and results will be reported soon.

The other problem with this method is that it does not yield to traditional analysis of random data or profiles as outlined by Darlington, Williamson and Walker (6, 36, 30, and 31). It is possible that another type of analysis could be used to evaluate the step function inputs to the roughness profile which will be made by the pipes or artificial bumps. It is desirable that someone follow up on the required analytical approach as a part of the evaluation methodology for this procedure.

Finally, this concept is attractive in the sense that only about six test sections are needed to cover a wide range of roughness and only one basic strong pavement section is needed to provide the base section. Considerable thought, however, needs to be given to the possibility of replication of roughness levels within the artificial calibration course. This could be done by adding two additional roughness levels whose roughness corresponds with a previously selected level, but with new roughness being introduced by an alternate pattern or an alternate means such as a few wider bumps or a rearrangement of the location of the bumps to interrupt regular patterns.

Natural Pavement Calibration Sections

The use of existing pavement sections for calibration of roughness devices is an attractive alternative, but there are problems. The attractiveness seems obvious since the sections are typical of the pavements to be measured in the real world; they contain normal roughness inputs of varying wavelengths and amplitudes over a wide spectrum of conditions. The problems, however, are multifold and must be considered. They include finding sections at extremes of roughness, the changing of roughness with time on a selected test section, the large number of sections usually required, and the considerable time and effort required to check the sections which are normally fairly widely spaced geographically.

Obviously it is not possible to set up a normal pavement section calibration course on which the test pavement roughness will remain constant. All of the pavements are in various degress of deterioration. Most of them were built smooth but they are in the process of change and experience shows that rough pavements change more rapidly than smooth pavements. It is absolutely essential then that for this approach some method of determining the roughness history of each test section with time be developed. This can be done in at least three ways; by true profiles measured "continuously" with instruments such as the SDP profilometer, true profiles measured at discrete increments with precise rod and level techniques, and repeated evaluation of the roughness of the calibration section by a standard rating panel.

Evaluation of the True Profiles - "Continuous." Of the three listed methods, the most attractive seems to be the use of existing pavement with an evaluation of their true profiles. This technique was chosen for use in the Brazil Project where adequate research funding was available to provide a standard profilometer, in this case the SDP profilometer, for making continuous measurements for calibration.

It seems, however, that this approach is impractical at the present time for use worldwide as a calibration standard. The use of a standard roughometer or other "gold-plated" version of a typical machine carried around the world as a standard device is not realistic as shown by experience at the AASHO Road Test and the work by the Center for Highway Research for the Texas Highway Department.

Evaluation of True Profiles - Discrete. It is possible that analytical techniques can be developed to accurately evaluate a discrete rod and level profile of pavement test sections set up for standardization. Field work is underway by Srinarawat and Hudson to evaluate this approach and to compare the accuracy required and the spacing or detail of the measurement points needed to provide adequate information (41).

If the approach is feasible from an analytical point of view, it is possible that field practice can show what type of level instrument and perhaps even what special level rod could be most useful to speed up the process and make it more practically applicable. The U.S. Air Force, for example, has developed a laser profiling system which works on the same basis as a rod and level but which takes automatic readings using a laser beam for a light source (38).

Another point favoring the rod and level approach is the hand labor which is normally available in many of the developing countries for which a roughness calibration is needed. The rod and level crews could make the necessary measurements on a quarterly or triannual basis with relatively little expense whereas in the United States, for example, such an approach might not be as economical as a profilometer.

Thusfar, work by Srinarawat and Hudson seems to indicate that it is possible to interchange machine and rod-level measurements (41).

Roughness Panel. A third approach to establishing and maintaining standard roughness evaluations of calibration test sections is appealing and should be carefully considered. It involves setting up a standard rating panel and developing a Generalized Roughness Index (GRI) which could be used not only for rating and establishing the roughness level of the calibration sections but as a standardized roughness scale for comparing instruments against each other all over the world without having to select any one particular instrument as the "standard."

This approach is far from thoroughly formulated and a great deal of additional thought will be needed before it can be accepted or rejected. However, it is worthy of consideration. If the method works, its value is readily evident. If adequate accuracy and details can be obtained, calibration sections could be set up and evaluated regularly without the expense and detail required for rod and level surveys.

Likewise, the potential pitfalls to the method are obvious. The method would basically be subjective rather than objective, which we, as engineers, always strongly desire. The potential value of the method lies in the question of whetehr or not we can make the subjective rating process objective by carefully selecting and establishing rating panels and rating procedures using up-to-date modern scaling and psychological techniques to overcome some of the subjectivity of the rating approach.

The basic value and acceptability of ratings for judging pavement quality was well established at the AASHO Road Test by Carey and Irick (5) and subsequently by Yoder and Milhous in the significant NCHRP study (37). As outlined previously herein, the Canadian Good Roads Association has also made an excellent practical application of the rating concept (5 and 35).

Another major problem with the roughness rating approach is possible cultural differences amongst countries. One country, for example, such as the United States, has a population accustomed to riding on paved roads which are basically smooth. On the other hand, other countries such as many of the countries in Africa and Latin America, are accumstomed to riding on unpaved roads. There is considerable concern that this cultural or historic difference, which is also by the way aggravated by traditional types and quality of vehicles used, would greatly affect any relationship developed by a rating scheme, and thus would completely invalidate the concept of relative ratings.

Generalized Roughness Index

After a thorough evaluation of the problem of establishing a common basis for comparing roughness measurements all over the world, and specifically comparing roughness measurements in Kenya, Brazil, and India in terms of using data taken from these three research studies and combining it for use in developing improved joint models, it is recommended that a Generalized Roughness Index or a universal roughness index be developed to serve as a basis for comparison instead of the output of any particular roughness device. On the surface this seems an arbitrary intermediate step; however, experience shows otherwise.

At the present time, no simple, robust roughness measuring and evaluation technique exists which is constant enough to become the appropriate "standard." The SDP profilometer might be considered, but work in adopting and using this instrument in Brazil and in comparing it to the Texas instrument manufactured ten years ago shows considerable difference in hardware and data processing techniques. Many people feel we are on the threshold of developing a non-constact probe to replace the road-following wheel for the SDP device. When this happens, you can be assured that the transfer function of this transducer will be different from that of the road-following wheel. Thus, the "standard" would change again. Many other examples could be cited, but for simplicity let it suffice to say that no real "standard" exists.

An example of a similar situation existed in 1962 concerning specifications and measurement of subgrade strength for pavement design. The American Association of State Highway Officials at that time desired to establish a standard design method which would be useable and used by all or at least a large majority of the State Highway Departments. There were many candidate measuring techniques, such as CBR, Texas Triaxial strength, shear modulus, California R-Value, and others. Majority vote would have selected CBR since it was used by more states than any other method. However, comparison of the CBR between states showed that even this so-called "standard" was far from standard since each state made slight modifications in the empirical test procedure. In the face of this diversity, Mr. T. S. Huff, Chief Highway Design Engineer for the Texas Highway Department and Chairman of the AASHTO Committee, recommended that a "soil support value" with a range from zero to ten be set up as the "standard." Each State Highway Department then related its soil test method to the soil support value rather than to some state test procedure. Nationwide information on standard test materials obtained from the AASHO Road Test was used to establish common points.

At this time, 15 years of experience in using the AASHO Interim Design Guide has shown the wisdom of selecting the what-seemed-at-the-time "arbitrary" Soil Support value.

GRI - A Combined Approach

Examination of alternatives indicates that the practical approach to solving this problem will involve some combination of the factors discussed above. To provide realism in the calibration, it is essential that 10 to 12 real pavement sections be included in a calibration course. These can be evaluated on a semiannual basis by rod and level surveys. A detailed methodology will be published by Srinarawat and Hudson within the next year.

To provide a large number of calibration sections of varying roughness and a calibration technique with some commonality around the world, a TRRL calibration course should be added to the calibration procedure. The methodology currently outlined by Abayanaka and the TRRL should be used until a more definitive consensus procedure is developed. Finally, the overall reasonableness of the scale can be assured at any time using a rating panel to ensure that reasonable roughness ratings are established for uniformity. These ratings should involve panels on at least all three or four major research efforts in the world and should include at least three or four common members in each panel in the initial stages of development. These common members could be employees of the World Bank or other research personnel who are involved in one or more of the world-wide research projects and who could beneficially visit other activities, thus providing the necessary commonality of ratings.

The GRI itself should have a relatively large scale, perhaps 0 to 100 and should be generalized with smoothness of existing new highways falling in the range of 10 to 15 and roughness on some of the roughest roads now perceived falling in the 70 to 80 range. This gives adequate room at both ends of the scale for changes and variations not yet observed and in no way detracts from the use of the Index.

Some readers will undoubtedly be disappointed that a firm Index in full detail is not presented here; however, work over the past 10 years shows that there will be several steps required to solve this problem and we believe this paper is a necessary first step in defining the problem so that an intelligent compromise can be reached.

Summary and Recommendations

Solving the problems of providing uniformity in roughness measurements is not easy. No perfect answer exists, only a set of intelligent alternatives. It is vital, however, that a framework be set up so that coordination and use can begin. I proposed a multifacted approach.

1. Develop a GRI which has a sound basis and can provide a pseudo-standard for comparison with any roughness scale existing now or to be developed.
2. Evaluate the use of the TRRL artificial calibration method for a variety of roughness devices and cases to determine its value, problems, and utility.
3. Apply the concept of a standard rating panel to provide an additional methodology for defining and reproducing the GRI in countries all over the world without the cost of purchasing an SDP profilometer or similar equipment.
4. Use rod and level surveys and recommend field equipment to simplify and speed up such surveys for establishing calibration points on a GRI.

It is recommended that action be taken to implement a GRI and to test the concepts set forth above. Cooperation will bee needed among several countries and agencies and a leader, such as the World Bank, is needed. Particular attention should be given to coordination of research data from the Kenya, Brazil, and India projects.

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ENGINEERING ECONOMICS OF THE MAINTENANCE OF EARTH AND GRAVEL ROADS

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This paper presents a methodology for the economic evaluation of maintenance programs for unpaved roads with low traffic volumes (under 250 vpd), a situation commonly encountered in rural areas in developing countries. The technique, drawing heavily on the road deterioration and user cost relationships developed in the IBRD/TRRL Kenya Road Transport Cost Study, involves a dynamic model that relates vehicle operating costs to traffic-induced road deterioration. The proposed methodology requires a two-step procedure: first to determine economically optimal and technically appropriate maintenance strategies; and second to apply these strategies to assess the economic value of the global road maintenance program. The incremental economic analysis used in the methodology permits the differentiation of benefits, in the form of vehicle operating cost savings, between routine and periodic maintenance. The use of the evaluation technique is demonstrated by application to a road maintenance program. Although the proposed method requires the use of multiple regression analysis and elementary calculus, graphical methods can be used as an alternative.

Cost-effective road maintenance practice is becoming a priority objective in most countries of the world since relatively nominal maintenance expenditures (about US$100-1000 per km in case of unpaved roads) can extend the life of existing infrastructure and postpone the need for its renewal. Although the economic value of road maintenance is manifest, its quantification is necessary to determine economically efficient levels of maintenance expenditures. Intensive research by the World Bank over the last decade suggests that the largest benefits of highway maintenance accrue in the form of vehicle operating cost (VOC) savings and that these are often the dominant factor in reaching economically optimal highway maintenance policy choices. In fact, vehicle operating costs on an unpaved road surface in good condition can be about 30-40% lower than if the surface were not adequately maintained.

The traditional approach to quantification of benefits due to good road maintenance takes the form of a static economic model that assumes fixed and often arbitrary levels of VOC under "good" and "poor" road maintenance conditions; the VOC difference comprising the benefits from good maintenance. This subjective method of analysis often results in suboptimal investment decisions relative to the scale and intensity of maintenance operations, such as the frequency of grading operations and the periodicity of gravelling(8).

The proposed method of analyzing road maintenance programs for unpaved roads is based on a dynamic analysis that relates vehicle operating costs to road surface conditions as they are affected by traffic and modified by maintenance operations. Traffic-induced road deterioration is defined in terms of roughness, rut depth, and depth of loose material. The corresponding vehicle speeds and VOC are also estimated as a function of surface condition parameters, because the road geometric and environmental factors influencing VOC remain unchanged under normal maintenance operations.

The Analytical Framework

The mathematical models that relate road surface condition and vehicle operating costs to traffic were developed under the IBRD/TRRL research program in Kenya (2, 3, 5). A review of the background references is necessary for an understanding of the methodology presented in this paper as limitations of space preclude a thorough discussion of these relationships.

Road Deterioration Relationships

Lateritic Gravels Roads

$$R = 3250 + 84\,T - 1.62\,T^2 + 0.016\,T^3 \qquad (1)$$

$$RD = 11 + 0.23\,T - 0.0037T^2 + 0.000073T^3 \qquad (2)$$

$$LD = 1.5 + 14e^{-0.23T} \qquad (3)$$

$$GL_a = 0.94\frac{T_a^2}{T_a^2 + 50}\left(4.2 + 0.092T_a + 3.5\,R_1^{\;2} + 1.88\,VC\right) \qquad (4)$$

Sand-Clay Earth Roads

$$R = 3250 + 785\,T \quad (5)$$

$$RD = 14 + 1.2\,T \quad (6)$$

$$LD = 1.5 + 14e^{-0.23T}\text{; under dry grading with } LD \geq 10.0 \text{ mm.} \quad (7)$$

$$LD = 1.0\text{; under wet grading} \quad (8)$$

Tracks

$$R = 3250 + 1255\,T \quad (9)$$

$$RD = 14 + 1.2\,T \quad (10)$$

$$LD = 1.5 + 14e^{-0.23T}\text{; with } LD \geq 10.0 \text{ mm.} \quad (11)$$

where:

- R = mean roughness (mm/km)
- RD = rut depth (mm)
- LD = depth of loose material (mm)
- T = cumulative traffic volume in both directions since last grading (thousands of vehicles)
- GL_a = annual gravel loss (mm)
- T_a = annual traffic volume in both directions (thousands of vehicles)
- R_1 = annual rainfall measured in meters
- VC = rise and fall, vertical curvature (%)

Vehicle Operating Cost Relationships

In the Kenya Study relationships, if certain physical characteristics of the road (such as geometrics, altitude and moisture regime) are fixed vehicle speeds and consumption of fuel, tires, spare parts, and maintenance labor can be estimated as a function of surface condition descriptors, R, RD, and LD (4). The average road geometric and environmental characteristics assumed in estimating speed and VOC components were: moisture content, 3%; average rise, 30 m/km; average fall, 30 m/km; average horizontal curvature, 175 degrees/km; and average altitude, 375 m. Oil consumption was also estimated from the results of the Kenya Study while for vehicle depreciation and crew costs the method recommended in de Wielle's work (1) was used. Composite VOC, obtained by adding the individual costs of VOC components, were calculated for the four representative vehicles--light goods vehicle (VL), single-unit truck (CMS), medium truck-trailer (CMR) and heavy truck-trailer (CLR)--used in the example demonstrating the application of the proposed analysis method to a road maintenance program. The physical characteristics and costs of these vehicles are shown in Table 1.

Table 1. Vehicle characteristics and costs

Vehicle Type	Light Goods Vehicle (VL)	Single Unit Truck (CMS)	Medium Truck-Trailer (CMR)	Heavy Truck-Trailer (CLR)
Physical Characteristics and Utilization				
Brake Horse Power	86	130	160	250
Payload (t)	1	7	14	24
Gross Vehicle Weight (t)	2.5	13	27	38
Fuel Type	Gas.	Diesel	Diesel	Diesel
Annual Operating Hours	800	800	1,200	1,200
Annual Crew Hours	2,000	2,000	2,000	2,000
Annual Kilometerage	25,000	25,000	25,000	35,000
Average Vehicle Life (yr.)	4	6	6	6
Unit Costs (net of taxes)				
New Vehicle (US$/veh.)	5,765	18,185	31,145	49,485
Tires (US$/tire)	69	395	395	395
Maintenance Labor (US$/hr.)	0.40	0.40	0.40	0.40
Crew Cost (US$/hr.)	0.50	0.40	0.40	0.40
Fuel (US$/litre)	0.33	0.30	0.30	0.30
Lub. Oil (US$/litre)	1.40	1.40	1.40	1.40

Unified Road Deterioration and Vehicle Operating Cost Relationships

A review of Kenya Study relationships showed both road surface deterioration parameters (R, RD, LD) and VOC could be reduced to a common denominator--cumulative traffic volume (T), provided that road geometric and environmental parameters remained fixed. This special characteristic of the two relationships was

Table 2. Vehicle operating cost equations

Road Surface and Vehicle Type	VOC Estimation Equation	Maximum T^a	Number of Observations	R^2	Standard Error of Estimate	Upper Bound Limit on VOC (US$ Equivalent)
Laterite[b]						
Light Goods Vehicle	$VOC=213.32+0.0094T^2$	100	19	0.985	1.85	310
Single Unit Truck	$VOC=392.44+0.0216T^2$	100	19	0.991	3.27	610
Medium Truck-trailer	$VOC=692.98+0.0422T^2$	80	19	0.984	8.53	1020
Heavy Truck-trailer	$VOC=861.93+0.0557T^2$	80	19	0.984	10.95	1225
Sand-Clay[c]						
Light Goods Vehicle	$VOC=207.02+12.87T-0.2496T^2$	30	17	0.987	6.72	370
Single Unit Truck	$VOC=385.96+18.56T-0.2660T^2$	30	17	0.994	8.39	710
Medium Truck-trailer	$VOC=676.29+34.73T-0.4365T^2$	30	17	0.994	16.96	1350
Heavy Truck-trailer	$VOC=833.83+51.40T-0.7947T^2$	30	17	0.994	22.49	1675
Track						
Light Goods Vehicle	$VOC=206.86+21.51T-0.8051T^2+0.003T^4$	30	17	0.986	7.42	370
Single Unit Truck	$VOC=384.54+31.04T-1.0593T^2+0.004T^4$	30	17	0.991	10.32	710
Medium Truck-trailer	$VOC=675.07+58.37T-1.9396T^2+0.008T^4$	30	17	0.992	19.23	1350
Heavy Truck-trailer	$VOC=831.15+86.49T-3.0527T^2+0.012T^4$	30	17	0.990	28.99	1675

[a] T=Cumulative traffic volume between gradings in both directions ('000 vehicles).
[b] Applies to gravel roads with at least 2 cm of laterite surface.
[c] Applies to earth roads and gravel roads with less than 2 cm of laterite surface.

Note: VOC = vehicle operating cost (US$/1,000 km)

Table 3. Vehicle operating costs as a function of traffic and road surface characteristics

Road Surface	T	R	RD	LD	VOC Estimate[a] VL	CMS	CMR	CLR
Gravel Road: (Laterite Surface)	0	3250	11.0	15.5	213.32	392.44	692.88	861.93
	1	3332	11.2	11.1	213.33	392.46	693.02	861.98
	10	3944	13.0	2.9	214.27	394.60	697.20	867.49
	20	4410	14.7	1.6	217.09	401.09	709.86	884.20
	30	4744	16.5	1.5	221.81	411.91	730.96	912.30
	50	5400	22.4	1.5	236.89	446.52	798.49	1001.11
	80	7794	43.1	1.5	273.67	530.89	963.09	1218.24
	100	11450	70.0	1.5	307.61	608.76	-	-
Maximum VOC[b]					310.00	610.00	1020.00	1225.00
Earth Road: (Sand-Clay Surface)	0	3250	14.0	15.5	207.20	385.96	676.29	833.82
	1	4035	15.2	11.1	219.64	404.25	710.58	884.78
	5	7175	20.0	10.1	265.08	472.11	839.02	1079.89
	10	11700	26.0	10.0	310.74	544.95	979.92	1304.11
	20	14000	38.0	10.0	364.53	650.73	1196.23	1543.38
	30	14000	50.0	10.0	373.47	706.97	1325.18	1659.38
Maximum VOC[b]					375.00	710.00	1350.00	1675.00
Earth Track:	0	3250	14.0	15.5	206.86	384.54	675.07	831.15
	1	4505	15.2	11.0	227.57	414.52	731.51	914.58
	5	9525	20.0	10.0	294.81	513.53	919.45	1188.03
	10	14000	26.0	10.0	344.45	593.12	1072.97	1402.98
	20	14000	32.0	10.0	360.80	646.98	1197.22	1535.71
	30	14000	50.0	10.0	373.47	706.97	1325.81	1659.38
Maximum VOC[b]					375.00	710.00	1350.00	1675.00

[a] Vehicle operating cost estimate (US$/1000 km).
[b] Maximum VOC corresponds to an average speed of 10-15 km/hr.

used to formulate a unified relationship that would permit direct VOC estimation as a function of cumulative traffic volume (T), bypassing all intermediate steps requiring estimation of vehicle speed and VOC components as a function of surface condition parameters. The unified VOC estimation equations were obtained by regressing composite VOC with cumulative traffic (T), for three road surface types (Table 2). The interaction among cumulative traffic, road surface condition variables and the related VOC is shown in Table 3.

Estimation of Benefits due to Reduced Vehicle Operating Costs

With the use of VOC equations shown in Table 2, savings in vehicle operating costs during a given time period (normally one year for purposes of discounting) can be estimated directly as follows:

1. Calculate an average VOC equation by weighting the VOC equations for different vehicles by their respective percentages in the traffic distribution.
2. Use the average VOC equation to:
(a) sum the operating costs of vehicles accumulated between gradings during one year, for a given maintenance strategy (e.g. 2 gradings per year and other required routine maintenance);
(b) sum the operating costs of vehicles accumulated between gradings during one year, for an alternate maintenance strategy (e.g. regravelling, 2 gradings per year and other required routine maintenance).
3. Determine incremental benefits (VOC savings) due to the alternate maintenance strategy as the difference between the summed VOC for the two maintenance strategies.

Mathematically, this can be expressed as:

$$\text{VOC Savings} = \int f(T)_1 dT - \int f(T)_2 dT \qquad (12)$$

where: $f(T)_1$ = VOC equation corresponding to maintenance strategy 1
$f(T)_2$ = VOC equation corresponding to alternate maintenance strategy 2

This procedure simplifies the calculation of benefits and does not require discrete addition of vehicle operating costs. For example, VOC benefits from two gradings per year, as compared to one grading per year on a sand-clay earth road, for a traffic stream containing only light vehicles (VL) can be expressed as:

$$B = 1000\int_0^x (207.02 + 12.87T - 0.2496T^2)dT$$

$$- 2(1000)\int_0^{x/2}(207.02 + 12.87T - 0.2496T^2)dT$$

$$= 1000\left(12.8T\left(\frac{x^2}{2} - \frac{x^2}{4}\right) - 0.2496\left(\frac{x^3}{3} - \frac{x^3}{12}\right)\right) \qquad (13)$$

where: x = accumulated number of vehicles per year in thousands
B = VOC savings in US$/1000 km.

Application to Economic Analysis of a Maintenance Program

The foregoing analytical procedure was applied to evaluate the economic value of a maintenance program for a network of 5,300 km of unpaved roads and tracks in West Africa. The area covered by the maintenance program is characterized by a dry climate with an average rainfall of about 1,100 mm per year. The average altitude is about 375 m with a flat to rolling terrain. As deposits of good lateritic gravels are not plentiful, a mechanically stabilised sand-clay mixture has been used as the wearing course on some unpaved roads.

The Maintenance Program

The maintenance program was designed to cover both routine and periodic maintenance (regravelling) activities. Fully mechanized routine maintenance operations consisted of grading, compacting, and dragging with tractor-drawn tires. Other operations such as filling potholes, clearing ditches and culverts, vegetation control, and spot regravelling were placed under the responsibility of road gangs. Each gang consisted of 25 laborers under the direction of a sector chief with responsibility for a road maintenance sector covering 100-200 km.

The physical requirements for the maintenance program included provision of equipment plus an initial stock of spare parts, workshop equipment, construction of equipment sheds, improvements to the workshops, construction of spare parts store, offices for equipment inventory control and inspection, and buildings to serve as administrative centers for the road sectors. To meet the requirements for mechanics, operators, and other skilled staff, a comprehensive training program including technical assistance and equipment for a training center was instituted. Senior administrative and technical staff needs were met by provision of a technical assistance team, whose functions included training of local staff and development of the necessary capability for equipment maintenance, and planning and execution of road maintenance works.

Sequential Economic Evaluation Procedure

The evaluation of the maintenance program followed a sequential procedure involving classification of roads included in the maintenance program, determination of appropriate maintenance strategies and the overall economic assessment of the maintenance program. After the road network was categorized broadly according to its engineering and traffic characteristics, an appropriate maintenance strategy was determined for each road surface type by calculating the incremental VOC savings associated with the improved maintenance operations. This was followed by determination of overall economic benefits (VOC savings), which were then compared with maintenance costs to evaluate the economic returns for the routine and periodic maintenance components of the maintenance program.

Table 4. Classification of road network for economic analysis of the maintenance program

Network Category	Length (km)	Base ADT (vpd)	Distribution of Traffic %VL	%CMS	%CMR	%CLR
Gravel Roads[a]	395	48	45	15	25	15
Gravel Roads	79	38	45	15	25	15
Gravel Roads	276	15	45	15	25	15
Earth Roads[b]	63	40	45	15	25	15
Earth Roads	181	34	45	30	0	25
Earth Roads	175	17	45	30	0	25
Earth Roads	49	34	45	15	25	15
Earth Roads	150	17	45	15	25	15
Earth Roads	259	10	45	30	0	25
Major Tracks[c]	99	29	45	15	25	15
Major Tracks	846	17	45	15	25	15
Major Tracks	72	13	45	55	0	0
Minor Tracks	1544	5	45	55	0	0
Minor Tracks	1130	7	45	15	25	15
TOTAL	5318					

[a] All gravel roads have a wearing course of lateritic gravels.
[b] All earth roads have a wearing course of a mechanically stabilized sand-clay mix.
[c] Most tracks follow the natural ground profile and may have a few drainage structures.

Classification of the Road Network

The unpaved road network of about 5,300 km, varied from all-weather gravel (laterite-surfaced) roads to tracks. The base-level average daily traffic (ADT) on the network ranged from about 50 vpd on the laterite-surfaced roads to less than 10 vpd on the tracks. As the road network had not been functionally classified, it was categorized according to its engineering and traffic characteristics (Table 4). An average traffic growth rate of 3% per annum was assumed over the analysis period.

Determination of Maintenance Strategies

Maintenance policies for unpaved roads included drainage and vegetation control, dragging (with rubber tires), emergency repairs resulting from washouts and weak spots, grading, and resurfacing with gravel. As no definite, quantified models are available to evaluate benefits from four of the basic routine maintenance operations--drainage clearance, vegetation control, shoulder maintenance and surface dragging--it was assumed that a certain level of expenditures for these routine items was required as part of the overall maintenance policies. For grading and gravelling, which consituted the major maintenance operations it was necessary to determine: (i) appropriate grading strategies for various classes of roads included in the maintenance program; and (ii) the traffic level at which surfacing with gravel would be economically justified.

Grading Frequency

The effect of more frequent grading is to improve the condition of the road surface and thereby lower vehicle operating costs. The vehicle operating costs corresponding to various grading frequencies for a given type of road surface were obtained by

$$\Sigma VOC = N \int_0^{x/N} f(T)_w \, dT \tag{14}$$

where:

- ΣVOC = cumulative vehicle operating costs in one year corresponding to a grading frequency, N/year (US$/km);
- x = cumulative number of vehicles during one year in thousands;
- N = grading frequency (number of bladings/year); and
- $f(T)_w$ = average VOC equation obtained by their respective share in the traffic stream.

The incremental benefits (reductions in vehicle operating costs) due to additional grading were obtained as:

$$\Delta VOC = \Sigma VOC_N - \Sigma VOC_{N+1} \tag{15}$$

where:

- ΔVOC = Incremental reduction in vehicle operating costs (US$/km)
- ΣVOC_N = Cumulative VOC per annum for a grading frequency, N/year (US$/year)
- ΣVOC_{N+1} =Cumulative VOC per annum for a grading frequency N+1/year (US$/year).

By equating the incremental benefits associated with progressively increasing grading frequencies with the incremental unit cost of grading (US$80/km); optimal grading frequencies were established for various

Table 5. Effect of grading frequency on vehicle operating costs for lateritic gravel roads

Grading Frequency (number/year)	Cumulative Vehicle Operating Costs, ΣVOC (US$/year)	Incremental Reduction in VOC, ΔVOC (US$/year)
ADT = 10 vpd		
1	1,670.00	-
2	1,669.60	0.40
ADT = 30 vpd		
1	5,020.04	-
2	5,011.38	8.66
ADT = 50 vpd		
1	8,400.98	-
2	8,360.85	40.13
ADT = 70 vpd		
1	11,833.29	-
2	11,723.17	110.12
3	11,702.78	20.39
ADT = 90 vpd		
1	15,377.50	-
2	15,103.47	274.03
3	15,058.09	45.38
ADT = 120 vpd		
1	20,774.76	-
2	20,219.14	555.62
3	20,116.25	102.89
4	20,080.24	36.02
ADT = 150 vpd		
1	26,489.33	-
2	25,404.15	1,085.18
3	25,203.19	200.96
4	25,132.86	70.33
ADT = 200 vpd		
1	36,819.60	-
2	34,247.33	2,572.27
3	33,768.93	478.40
4	33,604.26	164.67
5	33,522.09	80.17
ADT = 250 vpd		
1	48,436.01	-
2	43,412.04	5,023.97
3	42,481.67	930.37
4	42,156.04	325.63
5	42,005.32	150.72
6	41,923.45	81.87
7	41,872.82	50.63

levels of base ADT. Where incremental benefits were larger than US$400/km (the cost of grading with compaction), the grading operation was supplemented with compaction. The incremental (marginal) reduction in VOC with increase in grading frequency for a lateritic gravel surface is shown in Table 5 with levels of base ADT varying from 10-250 vpd. A similar analysis was

Table 6. Economically optimal frequency of grading operations

Road and Surface Type	ADT (vpd)	Optimal Grading Frequency (Gradings/year) Without Compaction[a]	With Compaction[b]	Total
Gravel Road: (laterite)	10	1	0	1
	30	1	0	1
	50	1	0	1
	70	1	1	2
	90	1-2	1	2-3
	120	1	2	3
	150	1-2	2	3-4
	200	2	3	5
	250	3	3	6
Earth Road: (sand-clay)	10	1	0	1
	30	2	2	4
	50	4	3	7
	70	6	4	10
	90	7	5	12
	120	10	7	17
Track: (earth)	5	1	0	1
	10	2	0	2
	30	4	1	5

[a] Unit Cost of grading = US$80/km
[b] Unit Cost of grading with compaction = US$400/km

carried out for earth roads with a sand-clay surface and earth tracks; the results are summarized in Table 6. The optimal grading frequency from an economic standpoint is defined as the breakeven point where incremental reduction in vehicle operating costs due to an additional grading (or grading with compaction) operation is equal to the incremental cost of one grading (or grading with compaction) operation. It was found that at least one grading per year was economically justified on all of the three surfaces whenever the base ADT was more than 5 vpd, when compared with the null alternative (no grading at all). The frequency of grading, however, is a function of surface type and level of traffic; a laterite (gravel) surfaced road requiring a considerably lower frequency of grading than a sand-clay (earth) road or an earth track, for the same level of traffic intensity. The optimal grading frequency on gravel roads was found to be less sensitive to traffic, changing from 1 to 6 gradings/year with ADT varying from 30-250 vpd. For an earth road, the optimum varied from 1 grading/year at an ADT of 10 vpd to about 17 gradings/year for a base ADT of 120 vpd. The optimal grading frequencies obtained by the preceeding analysis were used as benchmarks and, where warranted, modified in the light of local experience and climatic conditions to arrive at operational routine maintenance strategies that maintained an appropriate technical balance between grading frequency and other essential routine maintenance operations, particularly for earth roads. As a result, the grading frequencies for gravel roads were slightly increased while those for earth roads and tracks were reduced (Table 7).

Gravelling

Surfacing an earth road with gravel provides a riding surface which can better withstand the deleterious effects of traffic and environment, thereby permitting all-weather usage. The serviceability of the road is considerably enhanced, while routine maintenance requirements become less stringent. In addition, the better surface quality results in lower vehicle operating costs.

In order to determine the breakeven traffic volume at which surfacing an earth road with gravel or resurfacing an existing gravel road would be economically justified, a benefit/cost analysis was carried out comparing the cost of four gravelling alternatives with benefits resulting from differences in VOC on gravel and earth surfaces with base ADT ranging from 30 to 90 vpd. The gravelling alternatives considered were:

Alternative:

- A - 10 cm thickness; 6 m width with 0.5 m shoulders.
- B - 10 cm thickness; 7 m width with 0.5 m shoulders.
- C - 15 cm thickness; 6 m width with 0.5 m shoulders
- D - 15 cm thickness; 7 m width with 0.5 m shoulders.

Routine maintenance strategies shown in Table 7 were assumed for the two surface types as required. Benefits at a given base ADT were then calculated as:

Table 7. Routine maintenance strategies adopted for the maintenance program

Road Type	Gravel Roads					Earth Roads					Tracks		
Base ADT (vpd):	10	30	50	70	90	10	30	50	70	90	5	10	30
Dry Grading (operations/year)	1	2	2	2	2	1	1	1	2	2	1	1	2
Grading with Compaction (operations/year)	-	-	-	-	-	-	1	1	1	2	-	-	-
Spot Repairs (m^3/km)	-	-	-	-	-	-	25	50	50	50	-	-	25
Light Routine Maintenance[a] (km/year/sector)	-	200	150	100	50	-	150	100	50	50	-	-	200
Dragging[b] (No. of operations)	-	20	30	50	90	-	-	-	-	-	-	-	-

[a] Filling potholes, drainage and vegetation control, shoulder maintenance, etc.
[b] With tractor-drawn tires.

$$B_{PV} = \Sigma_{a=1}^{n} \left[N_E \int_0^{x_a/N_E} f(T_E)\, dT - N_G \int_0^{x_a/N_G} f(T_G)\, dT \right] pwf_a(i,n) \qquad (16)$$

where:

- B_{PV} = present value of difference in vehicle operating costs between gravel and earth surfaces summed over a period of one regravelling cycle (US$/km);
- n = analysis period = regravelling cycle (years);
- x_a = cumulative number of vehicles during year "a" in thousands;
- N_E = grading frequency for earth surface (number/year);
- N_G = grading frequency for gravel surface (number/year);
- $f(T_E)$ = average vehicle operating cost equation for earth road;
- $f(T_G)$ = average vehicle operating cost equation for gravel road; and
- $pwf_a(i,n)$ = present worth factor for period a, discount rate i, and analysis period n.

For this analysis, the average VOC equations for gravel and earth roads are given as:

$$VOC_G = 457.39 + 0.0264T^2 \qquad (17)$$

$$VOC_E = 445.20 + 24.97T - 0.3806T^2 \qquad (18)$$

with 45% VL, 15% CMS, 25% CMR, and 15% CLR with VOC expressed in US$/1000 km. The details of the analysis are presented in Table 8.

The regravelling cycle in years, obtained by dividing the thickness of the gravel surface by the average annual gravel loss, was taken as the analysis period. Discounted VOC savings were obtained as the present value of the difference in vehicle operating costs between gravel and earth surfaces over the analysis period as given by Equation 17. The net present value, then, was given as the difference between VOC savings and the corresponding cost of gravelling. The breakeven ADT for the four gravelling alternatives was taken as the ADT at which the net present value becomes zero. At a discount rate of 12% (assumed to be approximately equal to the opportunity cost of capital), it was shown that regravelling alternative B was economically justified for all roads with a base ADT of at least 47 vpd. Accordingly, 395 km of gravel roads with an average base ADT of 48 vpd (Table 4) were included for gravel surfacing in the maintenance program; the design standards for regravelling comprised a 7m wide running surface, 0.5m wide shoulders, and a 10cm thickness.

Table 8. Benefit cost analysis to determine breakeven ADT for gravelling

Base ADT[a] (vpd)	Average Annual Gravel Loss[b] (cm)	Regravelling Cycle for Gravel Thickness (years)		Cost of Gravelling (US$/km) Alternatives				Present Value of VOC Savings (US$/km) at 12% Discount Rate		Net Present Value (US$/km)			
		10 cm	15 cm	A	B	C	D	A,B	C,D	A	B	C	D
30	0.98	10	15	7,165	8,265	10,745	12,400	3,270	3,960	-3,895	-4,995	-6.785	-8,440
50	1.26	8	12	7,165	8,265	10,745	12,400	8,300	10,390	1,135	35	-355	-2,010
70	1.40	7	11	7,165	8,265	10,745	12,400	9,845	12,860	2,680	1,580	2,115	460
90	1.54	6	10	7,165	8,265	10,745	12,400	10,830	14,950	3,665	2,565	4,205	2,550

Alternative:	A	B	C	D
Breakeven ADT:	43	47	50	58

[a] Traffic growth: 3% per annum.
[b] From equation 4 with R_1 = 1.1 m; VC = 3%.

Economic Evaluation of the Maintenance Program

The economic evaluation of the maintenance program consisted of an engineering-economic assessment of the condition of the road network and the associated vehicle operating costs with and without the maintenance program. Without the maintenance program, it was estimated that effective road maintenance would decrease systematically and eventually cease in about four years. Maintenance output for with and without maintenance program conditions is given in Table 9.

Table 9. Maintenance output with and without maintenance program

Year	Without Maintenance Program Routine Maintenance Gravel Roads (km)	Earth Roads (km)	Tracks (km)	Total (km)	With Maintenance Program Routine Maintenance[a] (km)	Regravelling (km)
1	474	161	965	1,600	-	-
2	474	161	565	1,200	1,800	100
3	474	161	165	800	5,318	100
4	400	-	-	400	5,318	100
5		-			5,318	95
6		-			5,318	-
7		-			5,318	-
8		-			5,318	-
9		-			3,545	-

[a] Distribution of roads and tracks as shown in Table 4.

The economic rates of return for the maintenance program (separately for routine and periodic maintenance components), were calculated by relating the incremental cost of equipment and other maintenance inputs to the corresponding incremental benefits in the form of reduced vehicle operating costs, resulting from improved road maintenance brought about during the economic life of the equipment.

The incidence and magnitude of vehicle operating costs and related benefits as a function of accumulated number of vehicle passes and level of road maintenance with and without the maintenance program is demonstrated in Figure 1. Although the proposed method of economic analysis requires the use of statistical regression analysis and elementary calculus, to estimate VOC savings, it can be seen from Figure 1, that such VOC estimates can be alternately determined by preparing templates consisting of graphed average VOC curves and then measuring the area under these curves for given maintenance strategies.

Benefit/Cost Analysis of Routine Maintenance

Costs for routine maintenance included (i) capital expenditures for maintenance and workshop equipment, an initial supply of spare parts, buildings, and related technical assistance, and (ii) recurrent maintenance expenditures for fuel, spare parts and labor, incremental to the amount spent without the maintenance program. Using procedures discussed in the preceding sections, benefits were calculated in terms of reduced VOC resulting from improved routine maintenance. The benefits were accumulated over the 14 classified sections and grouped in three categories. The economic life of equipment was estimated to average about 8 years while the salvage value of buildings at the end of the analysis period was estimated at 50% of initial cost. Other than the roads to be regravelled under the maintenance program at the rate of about 100 km per annum during the first four years of the maintenance program, gravel roads with less than 2 cm of laterite surface were treated as earth roads for purposes of the economic analysis of routine maintenance. The results of the benefit/cost analysis are presented in Table 10. The economic rate of return for routine maintenance operations under the maintenance program was estimated at 74%

Figure 1: ECONOMIC BENEFITS FROM MAINTENANCE

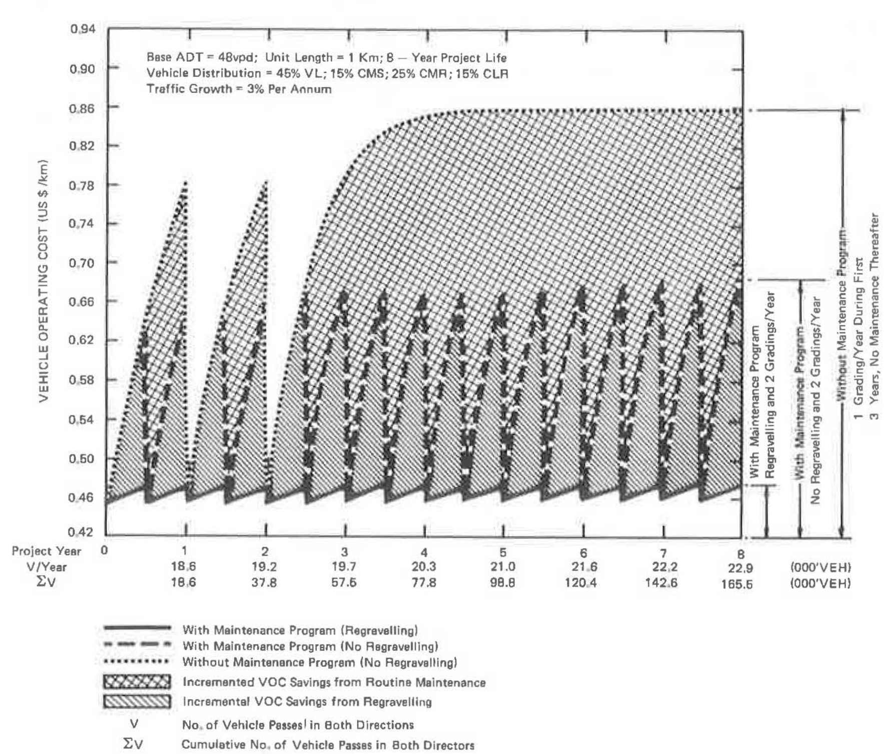

Table 10. Routine maintenance - benefit/cost analysis

	COSTS (US$ million)						BENEFITS (US$ million)			
	Capital Investment			Recurrent Maintenance Costs			Incremental VOC Savings			
Year	Equipment & Spare Parts	Buildings	Technical Assistance	With Project	Without Project	Incremental Costs[a]	Gravel Roads (750 km)	Earth Roads (877 km)	Tracks (3691 km)	Total (5318 km)
1	1.473	0.708	0.621							
2	2.945	1.416	0.621	0.493	0.493	-	0.227	0.231	1.149	1.607
3			0.621	1.479	0.410	1.073	1.116	1.335	4.138	6.589
4				1.479	0.410	1.073	1.098	2.056	4.253	7.407
5				1.479	0.244[a]	1.236	1.079	2.694	4.525	8.298
6				1.479	0.244	1.236	1.351	2.891	4.809	9.050
7				1.479	0.244	1.236	1.831	2.960	4.887	9.678
8				1.479	0.244	1.236	2.869	3.028	4.974	10.871
9				1.479	0.244	1.236	3.232	2.981	5.060	11.270
10		-1.062[b]		1.479	0.244	0.742	2.212	2.120	3.461	7.794

IRR = 74.1%
B/C @ 12% = 3.17

[a] Salaries for permanent road maintenance staff.
[b] Salvage value of buildings.

corresponding to a benefit/cost ratio of 3.17 at a 12% discount rate.

Benefit/Cost Analysis of Periodic Maintenance (Regravelling)

The cost of periodic maintenance included expenditures for equipment and technical assistance, and operational recurrent costs over the 4-year regravelling program. The average regravelling output was estimated at about 100 km per year with a total output of 395 km. A 50% salvage value was applied at the end of the regravelling program, because equipment would have been used for only one-half of its 8-year economic life. Since the regravelling cycle was calculated to be about 8 years, while only about 100 km would be regravelled each year, the residual thickness of the gravel at the end of the analysis period was also assigned a terminal value. The annual gravel loss was estimated to be about 12.6 mm per annum. The benefits from regravelling were taken as the incremental reduction in vehicle operating costs, additional to the reduction effected under routine maintenance, and expressed as:

$$\Delta B = (VOC_2 - VOC_0) - (VOC_1 - VOC_0) \qquad (19)$$

where:

ΔB = Incremental benefits from regravelling (US$ million).

VOC_0 = Vechicle operating costs under the null alternative--one grading/year and other routine maintenance operations for initial three years of the program; no maintenance thereafter (US$ million).

VOC_1 = Vehicle operating costs under maintenance alternative '1',--2 gradings/year and other routine maintenance operations over an 8-year period (US$ million).

VOC_2 = Vehicle operating costs under maintenance alternative '2',--regravelling at 100 km/year over 4 years, 2 gradings/year and other routine maintenance over an 8-year period (US$ million).

Then,

$VOC_1 - VOC_0$ = VOC savings under maintenance alternative '1'.

$VOC_2 - VOC_0$ = VOC savings under maintenance alternative '2'.

or,

$\Delta B = VOC_2 - VOC_1$

The results of the analysis are presented in Table 11. The incremental rate of return for regravelling operations was estimated at 17%, corresponding to a benefit/cost ratio of 1.26 at a discount rate of 12%.

Sensitivity Analysis

The specific risk elements related to the maintenance program were increase in costs and possible shortfalls in the projected maintenance output. A sensitivity analysis was carried to evaluate the effect of these parameters on the economic return of the maintenance program components (Table 12).

If routine maintenance were confined to the most

Table 11. Regravelling - benefit/cost analysis

	Costs (US$ millions)				Benefits VOC Savings (US$ millions)		
Year	Equipment	Technical Assistance	Recurrent Expenditure	Salvage Value of Remaining Gravel Surface	Alternative 1	Alternative 2	Net Incremental Benefits Alt. 1 - Alt. 2
1	1.832						
2		0.080	0.464		0.683	0.802	0.099
3		0.080	0.464		0.718	1.030	0.312
4		0.080	0.464		0.746	1.280	0.534
5	-0.916	0.080	0.464		0.785	1.611	0.820
6					2.648	3.662	1.014
7					2.694	3.763	1.069
8					2.747	3.881	1.134
9				-0.579	2.798	3.999	1.201

IRR = 17.2%
B/C @ 12% = 1.26

Table 12. Sensitivity analysis

	IRR (%)
A. Routine Maintenance - IRR=74.0%; B/C@12%=3.17	
Effect of Reduced Maintenance Output	
Network Maintenance:	
5,300 km[a]	74.1
3,500 km	63.0
1,800 km	26.6
Effect of Reduced Equipment Utilization	
Economic Life of Equipment:	
8 years[a]	74.1
7 years	73.0
6 years	71.3
Effect of Cost Increases	
Increase in Costs:	
5%	70.3
10%	66.9
15%	63.8
20%	60.8
Effect of Increase in Benefits	
Increases in Benefits:	
5%	77.9
10%	81.8
B. Regravelling - IRR=17.2%; B/C@12%=1.26	
Effect of Reduction in Annual Output of Regravelling	
Kilometers Regravelled per Year:	
100 km[a]	17.2
80 km	12.4
Effect of Increase in Cost of Regravelling	
Increase in Costs:	
5%	16.1
10%	15.0
20%	13.0
Effect of Increase in Benefits from Regravelling	
Increase in Benefits:	
5%	18.4
10%	19.5

[a] As assumed under the maintenance program.

important road links (about 3,500 km of roads and tracks), it would have an economic return of about 63%. If the training program failed to produce sufficient personnel to expand maintenance operations, or if a shortage of recurrent funds limited maintenance to current levels (about 1,800 km), routine maintenance would yield an estimated economic return of about 27%. The economic return for routine maintenance was relatively insensitive to reduced equipment life and the corresponding reduction in maintenance output during the later years of the program, showing only a 3 percentage point drop in the rate of return with equipment life reduced from 8 to 6 years. A 20% increase in costs would lower the economic return to 61% while a 5% increase in benefits, a distinct possibility resulting from a probable traffic growth in excess of the assumed 3% would raise the economic return to 78%.

Relative to regravelling operations, a 20% reduction in the annual regravelling output from 100 km to 80 km would lower the economic return to 15% while a 10% increase in benefits would raise it to 20%.

Conclusions

An attempt has been made in this paper to present an improved economic analysis method for the evaluation of road maintenance programs for unpaved roads. The method employs some of the latest research findings related to traffic-induced deterioration of unpaved roads and its effect on vehicle operating costs. This evaluation technique removes much of the subjectivity from estimation of vehicle operating costs as they are affected by the quality and scale of maintenance operations and provides the analyst a tool for determining economically optimal levels of routine and periodic maintenance. The analysis can be carried out with a portable hand calculator without recourse to expensive and time-consuming computer-based models. Some of the salient conclusions of the maintenance program example described in the paper are:

1. Efficiently executed routine maintenance operations yield a very high economic return and can help to offset the need for early renewal of the road infrastructure.
2. Once an earth road is surfaced with gravel, routine maintenance requirements become less stringent and require a lower frequency of grading operations.
3. The optimal grading frequencies resulting from economic analysis should be used only as guidelines in the design of maintenance programs; where necessary, they should be modified to reflect actual operational conditions.
4. The minimum breakeven ADT at which gravel surfacing of earth roads becomes economically justified varies from about 45-60 vpd, depending upon the design standards used.

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EFFECT OF SIMPLE ROAD IMPROVEMENT MEASURES ON VEHICLE OPERATING COSTS IN THE EASTERN CARIBBEAN

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The paper describes the effect on vehicle operating costs of a simple labour-intensive method of rehabilitating and maintaining badly deteriorated bitumen surfaced roads in the Eastern Caribbean. The techniques developed for the rehabilitation and maintenance of the roads are described, and the equipment, materials and manpower required are listed. The rehabilitation and maintenance system is a simple one restricted to providing adequate drainage, filling the potholes in the road and providing a minimum seal over the whole road surface. The reduction in vehicle operating costs resulting from the improvement in the riding quality of the road surface is shown to be sufficient to recover the rehabilitation costs in a very short time even at flows as low as 100 vehicles per day. The majority of the roads included in this scheme are of low strength, have low geometric standards and have traffic flows ranging from 50 to 1500 vehicles per day. All the roads have been trafficked for at least one year since being rehabilitated and some for two years. During this time little or no damage to the surface has taken place.

Introduction

This report describes the cost-effectiveness of a simple labour-intensive method of rehabilitating and maintaining badly deteriorated bituminous road surfaces.

The method used to improve the road surfaces was developed in the Eastern Caribbean island of St Vincent by one of the authors whilst Regional Public Works Advisor to the British Development Division in the Caribbean. Data on vehicle operating costs were obtained from an investigation into vehicle operating costs being undertaken by TRRL in the Eastern Caribbean and from an earlier study(1). The cost-effectiveness of the operation is calculated by comparing the cost of the improvements with the savings in vehicle operating costs resulting from the improved riding quality of the road surfaces.

All the roads included in the analysis are of low strength having a modified structural number of less than 3,(2) and have high horizontal and vertical curvature (at least 300 degrees/km and up to 100m/km respectively). They carry between 50 and 1500 vehicles per day and have all been trafficked for between one and two years since rehabilitation.

As the improvements were carried out to the road surface only there was no change in road geometry but the riding quality of the road surface was improved. Riding quality has been measured using a vehicle mounted bump integrator unit. The roughness values obtained were then converted to the corresponding towed 5th wheel bump integrator value(1). The road surface roughness as measured by this method reduced from 7000mm/km before the improvements to an average of 4000mm/km on completion of the restoration of the road surfaces. Details of these roughness measurements for a representative sample of the roads are given in Table 1, and their locations are shown on the map in Figure 1.

All costs are quoted net of tax in Eastern Caribbean dollars (EC$2.7 = US$1: EC$ 5.2 = £1 sterling).

The situation prior to the Rehabilitation Programme

St Vincent has 320 kilometres (200 miles) of bituminous surfaced roads and, with the exception of 24 kilometres (15 miles) which have been reconstructed over the last 10 years, they had received little if any surface maintenance since they were first bituminised in the period 1950-1968. This neglect had resulted in serious deterioration of the running surface characterised by extensive potholing and crazing, as shown in Figure 2. If a 'pothole' is defined as having a minimum diameter of 150mm (6") and a minimum depth of 25mm (1"), then by 1976 incidences of one pothole per linear metre of road were commonplace, and in many cases the whole width of the road surface had been broken up, forcing traffic to drive on the boulder base. The method of construction adopted when the roads were originally bituminised was basically grouted macadam. A loose layer of 25mm (1") stones was laid over a boulder base, blinded with 13mm (½") stones and then penetrated with bitumen and finished off with a layer of 6mm (¼") to dust. Whilst this method of construction provides a reasonably waterproof finish initially, it has the disadvantage that once water gains access through the finished surface it easily penetrates the porous 25mm (1") layer down to the base, lifting off large areas of surfacing.

Between 1968 and 1976 the St Vincent Government, using grants from the UK Ministry of Overseas Development, reconstructed 8 kilometres (5 miles) of the Leeward (West Coast) Highway and 16 kilometres (10 miles) of the Windward (East Coast) Highway. In addition in 1978 the reconstruction of a further 19 kilometres (12 miles) of the Windward Highway was started using funds provided by the European Development Fund. However with reconstruction costs of the order of $300,000 per kilometre ($½ million per mile) it was clear that because of limitations on funds the remainder of the road network could not be reconstructed in this way.

The amount available for maintenance of bitumen roads in the Recurrent Annual Budget is only $800,000 or $2500 per kilometre ($4000 per mile) approximately and the Roads Maintenance Department found that these funds were sufficient only to repair the worst of the potholes with premixed bituminous macadam. It was not possible to undertake a maintenance and surface dressing programme sufficient to arrest the decline of the road surface condition. To alleviate this situation the UK Ministry of Overseas Development in 1975 approved a grant to establish a Pilot Road Maintenance Unit on St Vincent to train local staff in maintenance procedures and to develop a cheap, labour-intensive method of rehabilitating the road system in order that its continued maintenance could be financed within the recurrent budget allocation.

The Rehabilitation and Maintenance Programme

The rehabilitation and maintenance of a typical broken bituminous road surface comprises the following activities:

1. Clean and improve side and cross drainage.
2. Repair the potholes, bringing the road profile as nearly as possible to its original shape.
3. Surface dress or otherwise waterproof the repaired surface.
4. Maintain the road so repaired by:-
   (a) Keeping the drains clean.
   (b) Repairing minor potholes as they appear and before they become major ones.

Drainage

Since the side drains and road culverts had become blocked and overgrown and numerous landslides had not been cleared, the initial cleaning of the side drains was carried out using a grader, a front end loader, and dump trucks. The cost of this operation ranged from $1250 to $2500 per kilometre ($2000 to $4000 per mile). Thereafter it has been possible to maintain the drains in good order at a cost of $950 per kilometre ($1500 per mile) per year.

Repairing Potholes

The normal method of repairing potholes is to trim off the sides, prime with MCO grade bitumen and fill with a premixed bituminous macadam. This was the method most extensively used in St Vincent. In order to make maximum use of labour the premix was mixed by hand and this was found to be more effective and cheaper than machine mixing. The cost of hand mixed material is about $93 per cu. metre ($70 per cu. yard) as compared with $113 per cu. metre ($85 per cu. yard) for material made by a Spotmix machine and $142 per cu. metre ($107 per cu. yard) for material bought commercially. It was found that 10 to 50 cu. metres of premix were required per kilometre (20 to 100 cu. yards per mile) depending on the state of the road and its width. The cost of providing and laying this material ranged from $1240 to $6200 per kilometre ($2000 to $10,000 per mile).

In order to reduce the costs of patching still further a method of grout patching was evolved (Figure 3). This consisted of brushing clean the area to be repaired, priming with an RC 2 grade bitumen, filling the hole with a suitably graded stone (the grading being dependent on the size of the hole), grouting with RC 2 bitumen and blinding with sand. Although rather crude, this method was quick and produced a watertight patch. It was also less extravagant on materials since the hole was not squared off and was therefore of smaller volume than when filled with premix.

Surface Dressing

It was soon found that roads repaired as described above quickly deteriorated again, especially during the rainy season. Water soon penetrates to the road base through crazing in the bitumen surface, through small potholes which have not been repaired, and through patches. For example the 8 kilometre (5 mile) long Vigi Highway required the expenditure of the following sums for repairing potholes over a twenty month period:-

Date	Cost of repairing 8 kilometres
Nov.75	$10,000
Mar.76	$25,000
Oct.76	$23,000
April/June 77*	$18,000

*immediately prior to surface dressing

Clearly the repair of potholes alone was insufficient to arrest the deterioration and it was necessary to waterproof the road surface by some inexpensive means. Accordingly a programme of surface sealing was started in November 1976. After experimenting with various forms of surface dressing, including spray and chippings and slurry sealing, it was found that the most effective method was labour-intensive sandsealing. This had the following advantages:

1. It could be readily adapted to labour-intensive methods.
2. It used an easily obtainable and cheap surfacing material.
3. It provided a dense, waterproof finish.
4. It caused the minimum disruption to traffic.

The method of sandsealing employed is as follows:

1. RC 2 bitumen is poured on the road surface at a rate of approximately 0.27 litres/sq. metre (0.2 gals/sq. yard).

2. It is then spread with rubber squeejees.
3. The bitumen is then covered with a layer of sand 25 to 30mm thick.
4. The sand is then lightly rolled.

These steps are shown in Figures 4, 5 and 6 and a completed road in Figure 7. At normal daytime temperatures in St Vincent (over 70°F) the RC 2 does not need heating and can therefore be poured straight from the drums.

The cost for a kilometre of 3.7 metre (12 ft) wide road is:

RC 2 Bitumen	6000 litres	@$0.42 per litre	$2500
Sand	100 cu-metres	@$7.50 per cu-metre	$ 750
Roller	6 days	@$35 per day	$ 210
Labour	177 man days	@$7 per day	$1240
Small tools etc			$ 300
		Total	$5000

or approximately $1.35 per sq. metre ($1.10 per sq.yd).

Regular Maintenance

Side Drainage. As mentioned above the recurrent maintenance cost for keeping clear side drains and culverts is approximately $950 per kilometre per year, which equates to one man per 1.6 kilometre (1 mile) per year plus a few hundred dollars extra for emergencies. In some instances the method adopted for this maintenance is to give one labourer the responsibility for keeping clear one kilometre of road, but usually casual gangs are employed to clear specific lengths. In either case the importance of keeping the side drains clear to stop water from getting into the road base cannot be over-emphasised.

Surface Maintenance. The object of the rehabilitation and surface dressing programme is to bring down the costs of regular surface maintenance to manageable proportions and in this the programme has succeeded. The first stretch of 8 kilometres (5 miles) to be sandsealed on the Leeward Highway has not required patching since it was sealed in the period November 1976 to May 1977. The more heavily trafficked Vigi Highway was sandsealed in early 1977 and to date (May 1978) has only required minor pothole patching on one occasion at a total cost of $360.

A mobile patching gange of 8 men has been formed equipped with minor tools such as hand rammers and a flat-bed truck for transport. This gang patrols the sealed roads in a 3-month cycle, patching with premix or grouting any small failures as they occur.

The cost of this gang per day is as follows:

Materials	$150
Labour	$ 80
Vehicle	$ 45
Tools etc	$ 5
	$180

Assuming a working year of 200 days the cost of one gang per year is $36,000. A gang can maintain 100 kilometres (70 miles) of rehabilitated road in this way, hence three units will be capable of covering all the bituminous roads in the country at a cost of $108,000 per year when the 240 kilometres (150 miles) included in the rehabilitation programme has been sealed.

Cost of the Rehabilitation and Maintenance Programme

The average rehabilitation costs are given below but in practice they vary from road to road as can be more clearly seen in Table 1.

1.	Re-establishing earth side-drains	$1900 per kilometre ($3000 per mile)
2.	Patching potholes	$4300 per kilometre ($7000 per mile)
3.	Sandsealing	$5000 per kilometre for 3.7 metre road ($8000 per mile for a 12 ft. wide road) $6200 per kilometre for a 4.6 metre road ($10,000 per mile for a 15 ft wide road)

Maintenance costs after rehabilitation are:-

1.	Drainage	$950 per kilometre ($1500 per mile)
2.	Surface patching after rehabilitation	Average $300 per kilometre ($500 per mile)

1st year	Nil	
2nd year	$ 60 - $ 190 per km	) Depending
	($100 - $ 300 per mile)	on the
3rd year	$190 - $ 620 per km	) traffic
	($300 - $1000 per mile)	volume

Future maintenance costs are a matter of conjecture but the indications are that total recurrent costs for maintenance when the programme is completed will be:

Drainage	320 kilometres (200 miles	@ $ 950 = (@ $1,500)	$300,000
Surface patching	3 patching units	@ $36,000 =	$108,000
			$408,000

This is well within the annual maintenance budget and leaves $400,000 for additional surface dressing, widening and improvements.

The life of the sandseal cannot be predicted with any certainty as the first stretch was only completed in November 1976. However, observation indicates that on lightly trafficked roads a life of 5 to 10 years can be expected.

The Effect of the Improvements on Surface Roughness

The roughness of the roads included in this programme was measured using a vehicle mounted Bump Integrator Unit. Roughness measurements were taken before, during and after the rehabilitation operations. Typical results, together with rehabilitation and surfacing costs, are given in Table 1.

On the worst roads on the island roughness of nearly 10,000mm/km were recorded, whilst those recently completely reconstructed gave readings as low as 2500mm/km. It was found possible in the rehabilitation programme to reduce roads from 7000mm/km down to 4500mm/km by patching, and to achieve a further reduction to 4000mm/km by sand-sealing.

Derivation of the Vehicle Operation Costs

Methodology

The method used to calculate the vehicle operating costs for this evaluation is the same as that used in the TRRL Kenya study(1) where the various components of total vehicle operating costs were considered separately on a quantity rather than a cost basis: the cost at any particular time being obtained by applying the relevant unit costs in operation at that time.

The components of total vehicle operating cost used in compiling the figures calculated for this study were as follows:-

1. Fuel consumption.
2. Oil consumption.
3. Spare parts.
4. Maintenance labour.
5. Tyre consumption.

As the geometry of the road system in St Vincent is the main factor controlling the speed of vehicles rather than the condition of the road surface and distances travelled are short, very little change in journey times between various points on the island has occurred since the road improvements. The average annual useage of vehicles has also changed little and therefore the value of time and overheads has been virtually unchanged, although in the case of the latter there may have been a slight reduction due to the fall in vehicle maintenance requirements.

Vehicle Types Evaluated

Three types of vehicle were considered to be sufficient for the purpose of this study to represent the overall vehicle population travelling on the roads of St Vincent. They are:

1. A "European type" saloon car with a 1600cc petrol engine, three years old and having covered 32,000 kilometres (20,000 miles).
2. A large van with a 2000cc petrol engine and carrying capacity of 1 tonne, three years old and having covered 48,000 kilometres (30,000 miles).
3. A 7 tonne carrying capacity truck with a 5000cc engine, three years old and having covered 64,000 kilometres (40,000 miles).

Vehicle Operating Costs for each Vehicle Type

The following tables give the cost per kilometre, net of tax, for the components of vehicle operating cost included in the analysis both before and after the improvements to the road surfaces at the unit costs prevailing in 1978.

1. 1600cc saloon car (European type)

component	before	after
fuel	0.06	0.06
oil	0.01	0.C1
parts	0.09	0.04
labour	0.04	0.02
tyres	0.08	0.04
total	0.28	0.17

ratio $\frac{\text{before}}{\text{after}} = 1.65$

reduction in cost = $0.11 per km ($0.18 per mile)

2. 2000cc 1 tonne van

component	before	after
fuel	0.07	0.07
oil	0.01	0.01
parts	0.12	0.06
labour	0.04	0.02
tyres	0.09	0.04
total	0.33	0.20

ratio $\frac{\text{before}}{\text{after}} = 1.65$

reduction in cost = $0.13 per km ($0.21 per mile)

3. 5000cc 7 tonne truck

component	before	after
fuel	0.11	0.11
oil	0.02	0.02
parts	0.11	0.07
labour	0.04	0.03
tyres	0.16	0.08
total	0.44	0.31

ratio $\frac{\text{before}}{\text{after}} = 1.42$

reduction in cost = $0.13 per km ($0.21 per mile)

The unit prices used in deriving these figures are:

	Car	Van	Truck
Vehicle cost	$20,000	$25,000	$45,000
Fuel (litre)	$0.49	$0.49	$0.40
Oil (litre)	$0.67	$0.67	$0.67
Labour (hour)	$6	$6	$6
Tyres	$72	$112	$260

At these prevailing units prices the direct savings in operating costs per vehicle kilometre realised by upgrading the road surfaces so that the surface roughness is reduced from 7000mm/km to 4000mm/km is $0.11 for a car, $0.13 for a 1 tonne van and $0.13 for a 7 tonne truck.

The relationships used to calculate the quantities of fuel, oil, spare parts and tyre consumption and to estimate the number of maintenance labour hours, are given in the TRRL Kenya Study report(1) and the Transportation Research Board, Special Report 160(3).

The Cost Effectiveness of the Road Improvements

In order to assess the cost savings likely to result from the improvements made to the roads, it is necessary to take into account both the cost of the initial rehabilitation and the likely future costs of maintaining the roads.

The average cost of rehabilitating one kilometre of road is $11,200 ($18,000 per mile) made up as follows:

improving drainage	$ 1900 per kilometre
filling potholes	$ 4300 per kilometre
sand sealing	$ 5000 per kilometre
total	$11,200 per kilometre

This is a once and for all expenditure provided that the annual maintenance programme described below is adhered to. The costs of this programme are:

clearing drains	$ 950 per kilometre
patching potholes	$ 340 per kilometre
re-sealing	$5000 per kilometre

The first two operations are carried out each year, but resealing is only undertaken when necessary, depending on the level of traffic on each particular road. Although none of the rehabilitated roads have yet reached the stage where further resealing is necessary the condition of those which have been trafficked for two years since being upgraded suggests that further resealing will not be necessary for at least another year for the most heavily trafficked roads. Three years has therefore been taken as the frequency with which the most heavily trafficked roads carrying at least 1000 vehicles a day will require resealing and four, five, six and seven years has been assumed for roads carrying 600-1000, 200-600, 100-200 and less than 100 vehicles a day respectively.

As stated above the savings in vehicle operating costs per vehicle kilometre resulting from the upgrading of the roads are $0.11, $0.13 and $0.13 for cars, light commercial and heavy commercial vehicles respectively. These give an average figure of $0.12 for all vehicles on the basis of the proportions of the different vehicle types in St Vincent.

In the analysis the cost effectiveness of the scheme is examined at various levels of traffic. The costs and benefits are discounted back to the base year in which reconstruction takes place and the number of years it takes to recover the initial reconstruction cost is calculated. The present best estimate of the rate of growth of traffic in St Vincent and the discount rate currently being used to assess road improvement schemes in the LDC's of the Eastern Caribbean are both 10%. Although some information existed on the cost of previous maintenance on the Vigi Highway, there was not sufficient to be able to include this generally in the analysis.

Table 2 compares the costs and benefits for each traffic level considered. It can be seen from the table that the cost of rehabilitating any road with traffic flows of more than 300 vehicles a day is recovered through savings in vehicle operating costs in the first year but it takes progressively longer at lower flows (2 years at 200 veh/d, 5 years at 100 veh/d) until at 50 veh/d it takes 12 years.

This progression is illustrated more clearly in the graph shown in Figure 8.

Summary

The simple labour-intensive methods of road rehabilitation and maintenance described in this paper were developed because of the necessity to find a cheap way of improving the roads of St Vincent. Full scale reconstruction was too expensive to be applied to the majority of the 320 kilometres (200 miles) of surfaced roads in need of rehabilitation.

The methods described will enable all the surfaced roads to be improved and maintained within the present annual maintenance budget, leaving some resources for improvements to the gradient, width and alignment of the roads each year. The savings in vehicle operating costs due to the improvement in the condition of the road surface equal the cost of the improvements to all the roads included in the scheme within two years, and from then on the net annual vehicle operating cost savings will substantially exceed the annual cost of maintaining the roads.

The labour-intensive method of sealing which has been adopted permits a very flexible approach to the management of road maintenance, it being possible to switch a maintenance gang from one area to another at a few hours' notice.

The maintenance technique adopted has greatly reduced the cost of operating vehicles in St Vincent and at the same time has increased the amount of work which can be carried out on the roads within the fixed budget available to the Roads Department.

Acknowledgements

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Table 1

Rehabilitation costs and surface roughness of some individual roads

Reference number	Location of road	Length kms	Rehabilitation costs* per km			Roughness (mm/km)			Vehicles per day
			Drainage	Patching	Sealing	Before patching	After patching	After sealing	
1	Vigi Highway (Mesopotamia - Arnos Vale)	8	$2500	$6200	$6200	7900	4300	3900	1540
2	Leeward Highway (Layou - Barouallie)	6.5	$2000	$2500	$5600	7500	4800	3800	250
3	Montreal Road	4	$1300	$6200	$5000	9300	5800	5400	560
4	Vermont Road	4	$1600	$3000	$5000	7500	4400	4200	140
5	Calder Road	3	$2500	$6200	$5300	9300	4800	4500	180
6	Clare Valley	2.5	$1000	$2000	$5000	7000	4600	4300	110

*Eastern Caribbean dollars. 1978 prices.

Table 2

The time taken to recover rehabilitation costs on low flow roads
(all costs are in EC dollars for 1 km of road)

Traffic flow (Veh/d)	Rehabilitation cost ($\$ \times 10^6$)	Annual maintenance cost ($\$ \times 10^6$)	Annual VOC savings ($\$ \times 10^6$)	Net annual savings per year ($\$ \times 10^6$)	Number of years to recover rehabilitation cost
400	0.0112	0.0023	0.0158	0.0135	0.8
300	0.0112	0.0023	0.0119	0.0096	1.2
200	0.0112	0.0022	0.0079	0.0057	2
150	0.0112	0.0021	0.0060	0.0039	3
100	0.0112	0.0021	0.0040	0.0019	5
50	0.0112	0.0020	0.0020	0.0000	12

Figure 1. MAP of St. Vincent showing the roads included in the rehabilitation scheme.

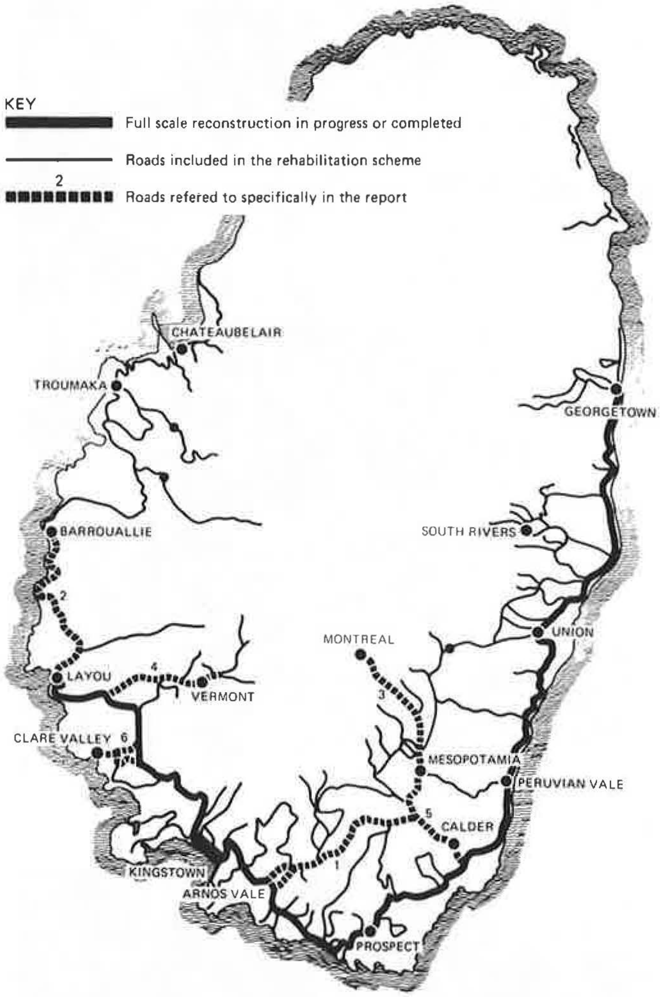

Figure 2.

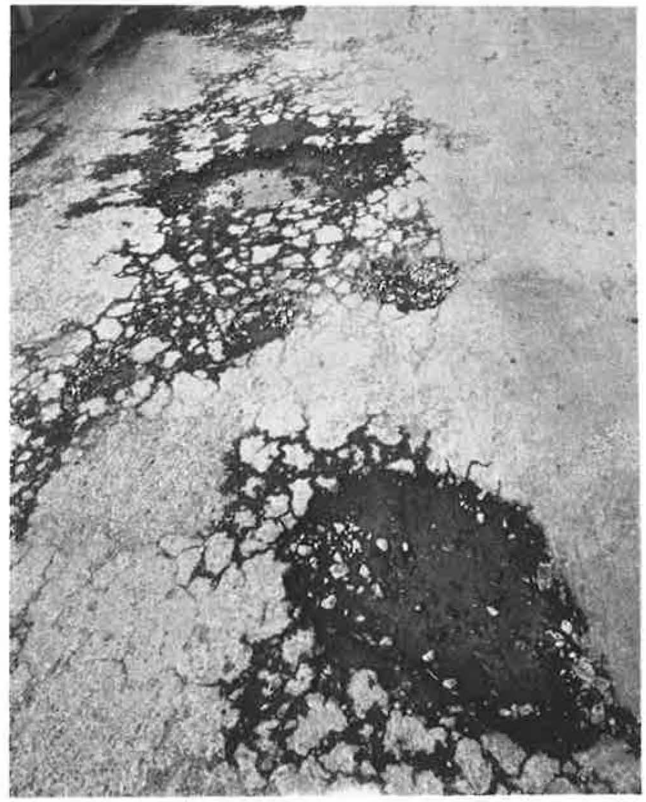

Figure 3.

Figure 4.

Figure 5.

Figure 6.

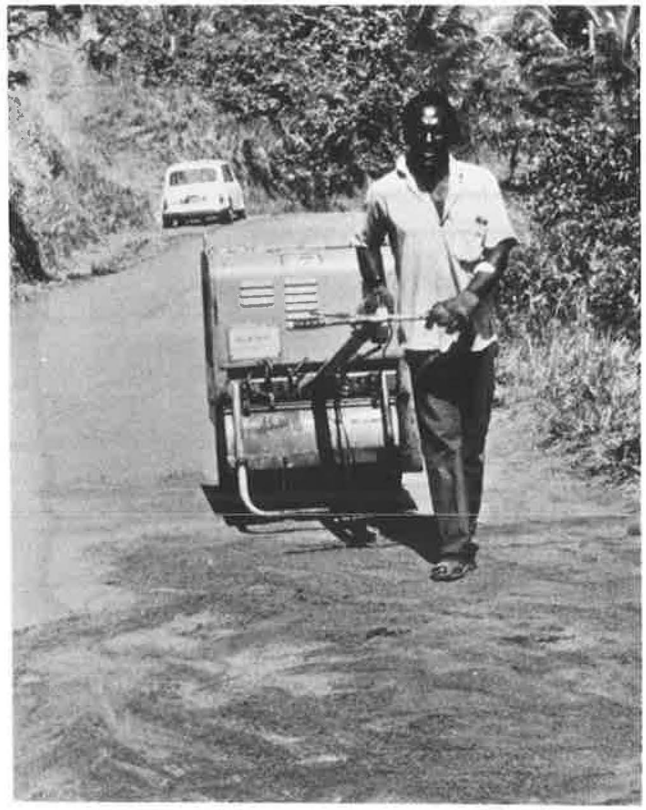

Figure 7.

IMPLEMENTING A PAVEMENT MANAGEMENT SYSTEM IN THE FOREST SERVICE

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During the last six years, the University of Texas at Austin and the U.S. Forest Service have developed and implemented a Pavement Management System (PMS) called LVR into two Regions of the Forest Service. This PMS is capable of designing asphalt and aggregate surfaced roads by considering material, traffic, environmental and economic characteristics. The LVR system has been in use by various Forests to design both recreational and logging roads. Roads have been designed in some Forests of Oregon, Louisiana, Arkansas, Montana and Georgia. Estimates of the savings in both engineering and construction are presently being assembled and evaluation of the system by the user engineers are being solicited. To date a number of programming errors have been discovered and corrected. However, even with these typical implementation problems, most of the user engineers are satisfied with the design process. In several of the Forests, the roads that have been designed will be monitored bi-yearly to determine if the roads are performing as predicted by LVR. Of particular significance are the comments from the user engineers who indicate that the program greatly increases their analytical capabilities and ability to obtain good life cycle estimates for a larger number of candidate designs than would otherwise be possible. After detailed evaluation of the implementation in these two Regions, the Forest Service will make a decision on service-wide implementation.

Introduction

The basic mission of the Forest Service is to carry out Federal responsibility for the wise use of forest and related watershed lands. These lands comprise one-third of the land area of the United States. The Forest Service has direct management responsibility for approximately 757,000 km^2 (187 million acres). The lands are managed for five different and sometimes conflicting purposes:

1. Timber
2. Watershed
3. Forage
4. Wildlife
5. Recreation

To carry out its management responsibilities the Forest Service is building one of the largest and most complex transportation systems in the world. Table 1 gives the magnitude of the road portion of the transportation system along with some generalized cost information. Road building and transportation administration are an important and integral part of the resource management process in the Forest Service.

Table 1. Kilometers and investment of the Forest Service transportation system.

Category	Kilometers[1]/	Approximate Investment (Dollars)
Existing Road System	353,600	3,572,000,000
Planned Additional Kilometers	185,600	6,960,000,000
Approximate Kilometers Constructed or Reconstructed Annually	16,000	379,000,000[2]/
Maintenance	353,600	61,000,000[2]/

[1]/ 1 km = 0.6 mile
[2]/ FY 1978

The surfacing for such an extensive road system represents a sizeable investment. For the 16,000 km (10,000 miles) of roads to be constructed or reconstructed in 1979, the cost of surfacing will be approximately $75,000,000. Costs of this magnitude require constant analysis and scrutiny. Engineers must assure themselves that the investment in surfacing is being spent wisely and efficiently. However the search for cost effectiveness and efficiency is severely hampered by the lack of specific surfacing design procedures for low volume roads.

For the most part present day pavement design technology was developed from and for roads carrying much higher volumes of traffic. Major research programs on pavement design were conceived, sponsored, and formulated to solve problems relating to high type pavements. Design theories and test roads paid scant attention to aggregate surfaced or bituminous surface treated roads. Yet these type roads make up the vast majority of the Forest Service transportation system.

In the early 1970's the Forest Service adopted a modified version of the AASHTO Interim Guide as the basis for determining structural design thickness for aggregate surfacing. The modification was based on a combination of (1) the AASHTO structural design equations (1), and (2) the thickness design charts developed by the U.S. Army Corps of Engineers for unsurfaced roads and airfields (2). For bituminous surfaced roads the Forest Service design procedure is essentially the same as the AASHTO Interim Guide.

Although a surfacing design procedure was adopted into Forest Service management procedures, it was readily apparent the procedures did not adequately treat nor consider many variables, constraints, and uncertainities. Furthermore, because road design in the Forest Service is decentralized, the surfacing design process was often greatly influenced by the prevailing practices that existed at many different geographic locations. Surfacing design was not being done in a uniform, consistent manner.

A critical review of the aforementioned factors led to the conclusion that a significant problem existed in surfacing design in the Forest Service. The factors can be summarized as follows:

1. Approximately $75,000,000 million dollars was being spent annually for surfacing on Forest Service roads.
2. Existing structural design procedures were inadequate and not responsive to low volume road situations.
3. Decentralized operations were resulting in inconsistent, non-uniform and widely varying procedures for structural design.

Forest Service managers assessed this situation and recognized the need for an indepth comprehensive review to determine whether surfacing design could be strengthened and substantially improved throughout the service. The first step in this comprehensive review was to initiate a study to define the problem and the ramifications of that problem. This study took the form of an analysis to determine if a systems approach was appropriate for the design and analysis of Forest Service roads.

Problem Identification and Definition

The first step was to further identify and explore the problem by acquiring detailed background information and investigating the present state-of-the-art for surfacing design in the Forest Service as well as other low volume pavement design concepts. An assessment of the Forest Service needs for a pavement management system showed that emphasis was placed on (1) optimizing the total pavement investment, (2) providing pavement performance prediction methods for planning purposes, (3) optimizing resource management efforts, (4) providing a tool for evaluating the effectiveness of specific pavement designs, and (5) unifying design efforts within the Forest Service. Problem definition was achieved by detailing the special constraints and considerations characteristic of low-volume forest road designs as compared with the design of "higher type" roads.

Following detailed identification of the problem, work was begun to formulate a conceptual system for designing and managing the surfacing of Forest Service roads. Wherever possible pavement management subsystems for low-volume roads were defined. Where definition of a subsystem was not possible because of the need for further research, relevant questions and ideas were formulated for consideration in the eventual development of the subsystems definition.

An essential part of the initial evaluation of subsystems was the interaction and exchange of information between the University of Texas research staff and Forest Service personnel that took place during a five day "brainstorming session." Many ideas and discussions were presented at this meeting including those on (1) the decision making process within the Forest Service, (2) decision criteria, (3) terminology, (4) system input variables, (5) pavement performance, (6) pavement failure, and (7) special constraints and considerations for Forest Service roads (3).

In order to determine the relative significance of the variables discussed at this session, a rating and ranking of the pertinent ideas and their importance to the proposed pavement management system were completed by the conference attendees. The results of this "importance rating" were then analyzed and the information obtained was used in developing a conceptual pavement management system for Forest Service Roads (3 and 4).

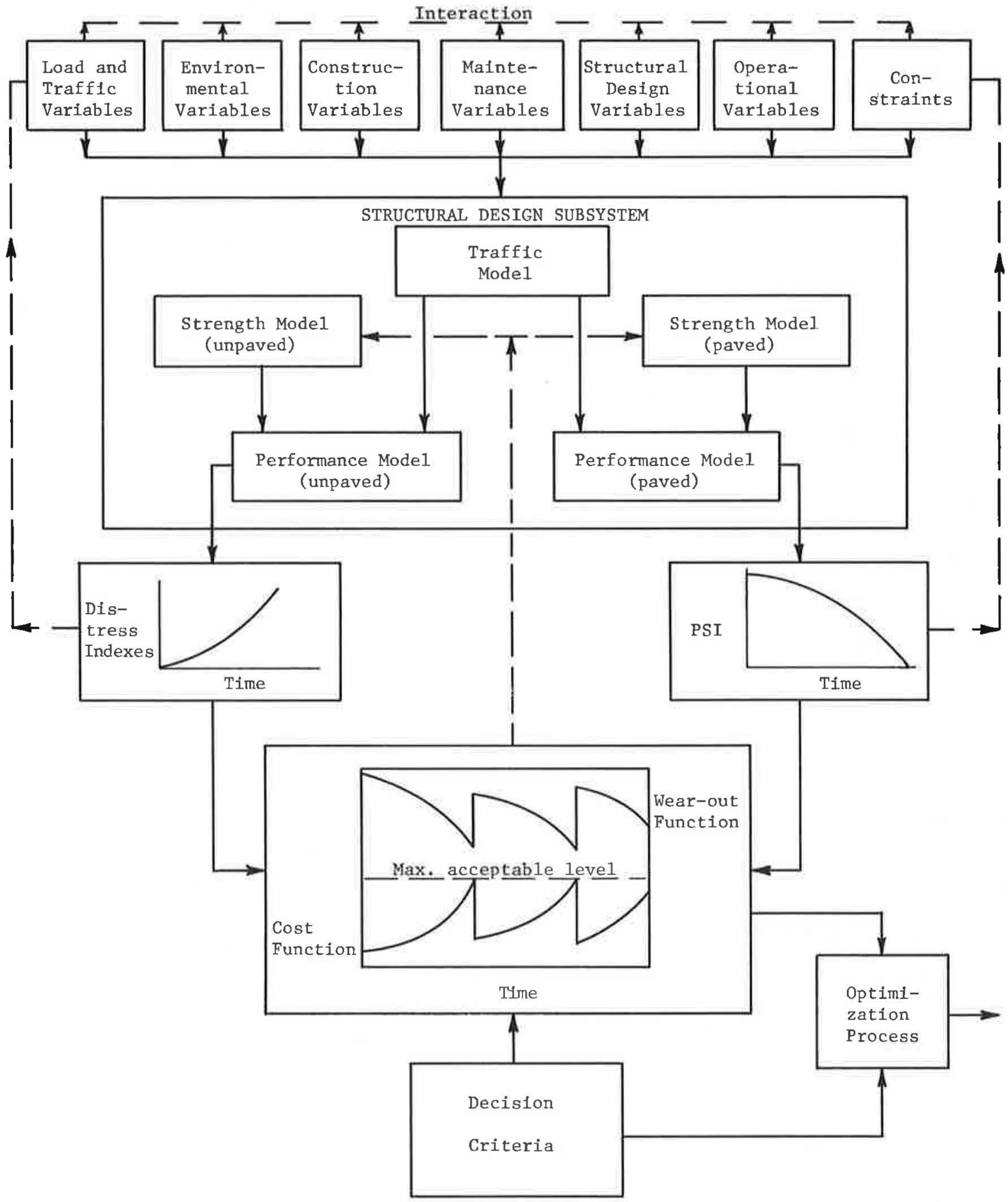
Conceptual System

The combination of structural design models, output representation, and evaluation criteria with the previously defined major components of a pavement management system for low-volume Forest Service roads resulted in the conceptual system as shown in Figure 1.

As can be seen in the diagram, the first step in the systems design process is the collection of all necessary input data. Once this is completed a summation of the predicted traffic and loads that will travel over the proposed road during the analysis period is calculated using the traffic model with pertinent input data. A structural strength for a design of given materials and layer thickness values are calculated using one of the two strength models. This information is then used in a performance model to determine (1) the wear-out function of the structure in terms of either distress or a present serviceability index, and (2) the performance of the structure in relation to a cost function in serving traffic needs.

When the measurable distress on a road reaches a level corresponding to a maximum acceptable cost level as determined by the decision criteria, some form of maintenance will be required to return the structure to an acceptable distress level. The structure is then re-evaluated according to the type of maintenance designated, and the extended life of the pavement is determined. This re-evaluation process is indicated by the dotted line from the serviceability-age history to the strength

Figure 1. Conceptual pavement management system.



models in Figure 1. This process of extending the life of the pavement through maintenance activities is continued for the predesignated design life of the road structure.

The total design and management evaluation process can be carried out for many different design and maintenance strategies, each goes through an optimization process and then is evaluated, compared, and arrayed for the subsequent decision.

LVR Program Capabilities

The pavement management system that was developed for the U.S. Forest Service consists of a single computer program, identified as LVR, that can be used to design asphalt concrete, surface treated, and aggregate surfaced roads. However, since the program will only design for a single road surface type at a time, in order to compare an aggregate surfaced road with a bituminous surfaced road, it is necessary either (1) to make a run for an aggregate surface, modify the input data slightly and rerun the program for a bituminous surface, or (2) to stack both sets of input data and obtain separate outputs for bituminous surfaced and for aggregate surfaced designs in one run of the program.

A brief description of the capabilities of the program follows, however details of the program and the various options are described in Reference 5.

Bituminous Surfaced Roads

The bituminous surfaced road design portion of the program uses the AASHTO structural design equation for flexible pavements (1, 6, and 7). This equation, which is currently being used by the U.S. Forest Service (8), is based on the concept of the Present Serviceability Index (PSI) of a pavement.

Using the bituminous surfaced road model, the user can design and compare single and multi-layered pavement structures of either asphalt concrete or bituminous surfaced treated surfaces.

Aggregate Surfaced Roads

Like the bituminous surfaced road design previously described, the aggregate surfaced design uses the current U.S. Forest Service method which is based on a combination of the AASHTO structural design equation for flexible pavements (1, 6, and 7), and the U.S. Army Corps of Engineers Thickness Design Charts (2). This method has been further modified to account for aggregate loss in the aggregate surface layer due to traffic action.

Failure of a candidate structure is defined as any of three events representing the time at which (1) the PSI reaches the minimum acceptable level, or (2) a 5.08 cm (2-inch) wheel path rut develops, or (3) the reduced thickness of the surface layer due to aggregate loss reaches a minimum acceptable value as specified by the user. This triple failure criteria is discussed later.

Using the aggregate surface design model, the user can design and compare single and multi-layered structures of either of two strategies of aggregate surfaced road. One strategy is the use of only aggregate surfacing during the design period; whereas the second strategy provides for a post construction bituminous surface treatment at a later time during the analysis period.

Failure Criteria

The principal surfacing material of Forest Service roads is aggregate. Bituminous surface treatments and asphalt concrete are used to a lesser extent. These surfacing materials fail in vastly different ways. Because of this problem, separate sets of failure criteria are used by the program for bituminous and aggregate surfaced roads.

Bituminous Surfaced Roads Failure Criterion

The performance of bituminous surfaced roads is based on the results of the AASHTO Road Test as presented in the 1972 Edition of the AASHTO Interim Guides for Design of Pavements (1) and in NCHRP Reports 128 (6) and 139 (7). In these reports, failure of a bituminous surfaced road is defined as the time at which the Present Serviceability Index of a pavement reaches the minimally acceptable value, P_t. This concept is demonstrated pictorially in the top portion of Figure 2.

Aggregate Surfaced Roads Failure Criteria

Unlike a bituminous surfaced road with its single failure criterion, the performance of an aggregate surfaced road is based on a triple failure criteria. The first component of the triple criteria is the PSI concept which is applied in the same manner for aggregate surfaced as for bituminous surfaced roads.

The second component of the triple failure criteria is related to rutting. Failure in this case occurs at the time when a 5.08 cm (2-inch) rut develops in the wheelpath. This criterion was developed and reported (2) by the U.S. Army Corps of Engineers and is discussed in more detail in Reference 5.

The third and final component of the triple failure criteria is based on failure due to excessive aggregate loss, which results when the thickness of the top layer is reduced to a user specified minimally acceptable level. The amount of aggregate loss as a function of time is either predicted by the Lund (9) aggregate loss model or specified directly; the choice is based on user preference. The aggregate loss models are discussed in more detail in Reference 5.

The resulting failure time is then the minimum of the times calculated from:

1. The rutting model as used by the U.S. Forest Service which involves computing the failure time due to rutting as the maximum of either
   a. The failure time predicted by way of the rutting model briefly discussed above or
   b. The failure time predicted by the AASHTO performance model.
2. The time at which excessive aggregate surfacing loss has occurred.

The rutting model, like the AASHTO performance model, was originally intended to be used to compute the design thickness needed to carry a certain number of 18-kip (80 kN) equivalent single axle loads under given circumstances. Given the thicknesses of the layers, the layer coefficients, and other necessary information, however, both models can be used to compute the number of 18-kip (80 kN) equivalent single axle loads which will have been accumulated when failure occurs. The number of these loads, then, can be converted to failure time by using a non-linear traffic model. An illustrative application of the triple failure criteria is shown in Figure 2.

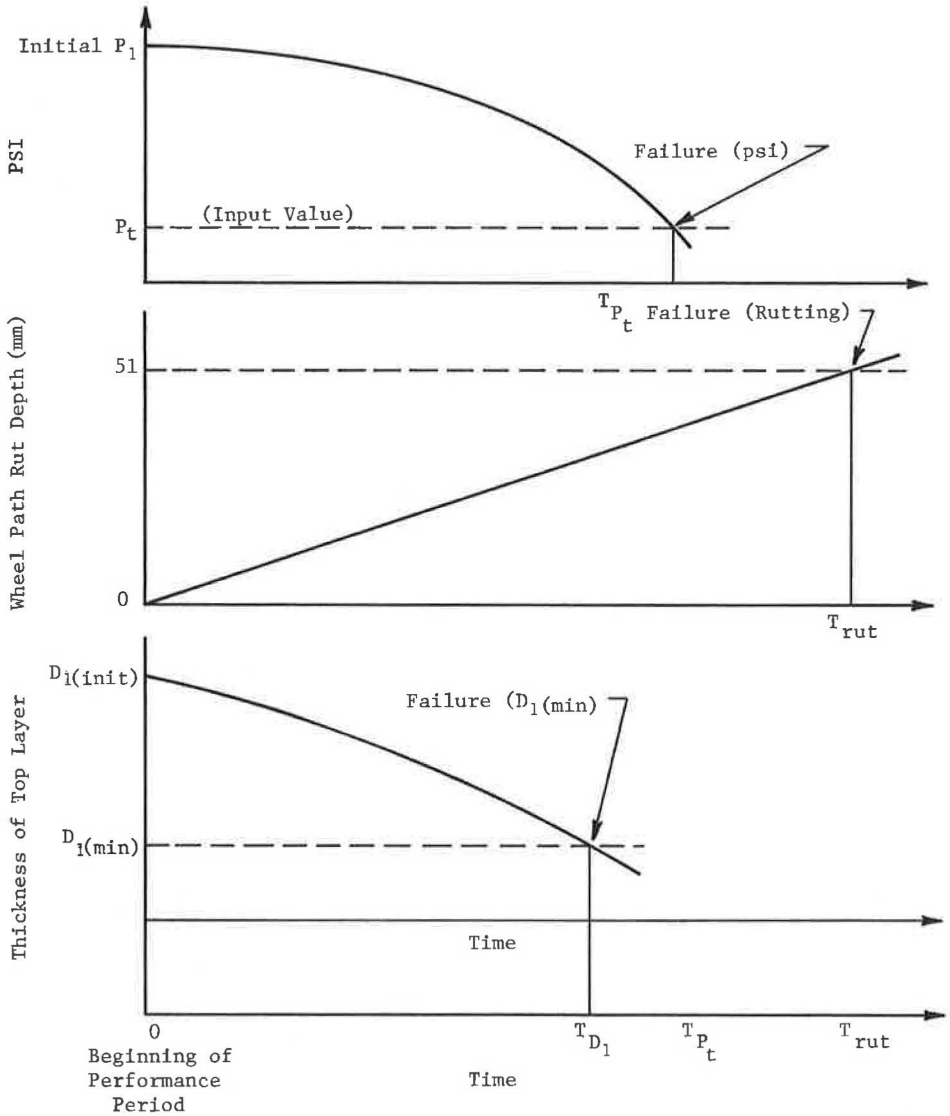
Implementation Procedure

The procedure for implementing LVR into Forest Service usage is visually described by the flow chart in Figure 3. The basic philosophy in this procedure was to (1) begin the trial usage at an early stage for "hands on" experience, (2) utilize experience of the Forest Service users as a guide for program revisions, and (3) conduct model analyses at the same time in order to improve the program.

To initiate the implementation, the first training session was held with Region 6 users in Portland, Oregon on December 20 and 21, 1976. During trial usage the program was examined to determine if everything was working properly. Interaction between Forest Service users and University of Texas project staff was very important in working out "bugs", developing the confidence of the users, and in answering various procedural questions. This interaction provided the information needed for revisions in the program.

After the trial usage was under way, the project staff began to make a more detailed analysis of the program. Information from the trial usage was helpful in selecting areas of needed study. It soon became apparent that the

Figure 2. Failure criteria for bituminous and aggregate surfaced roads (4).



P_t ~ minimally acceptable level of PSI

$D_{1(init)}$ ~ Initial thickness of top layer

$D_{1(min)}$ ~ minimum allowable thickness of top layer

T_{P_t} ~ Time at which psi equals P_t

T_{rut} ~ Time at which a 51 mm (2 in.) rut develops in the wheelpath

T_{D_1} ~ Time at which thickness of top layer equal D_{1min}

rutting and aggregate loss models would have to be studied in more detail. A sensitivity analysis on the program played a major role in examining LVR input variables. The sensitivity analysis was used to make program revisions and to analyze the significance of input variables.

Other activities during the implementation period involved conducting training sessions in several other Forest Service Regions, developing a user's manual, and providing a detailed documentation of the LVR program.

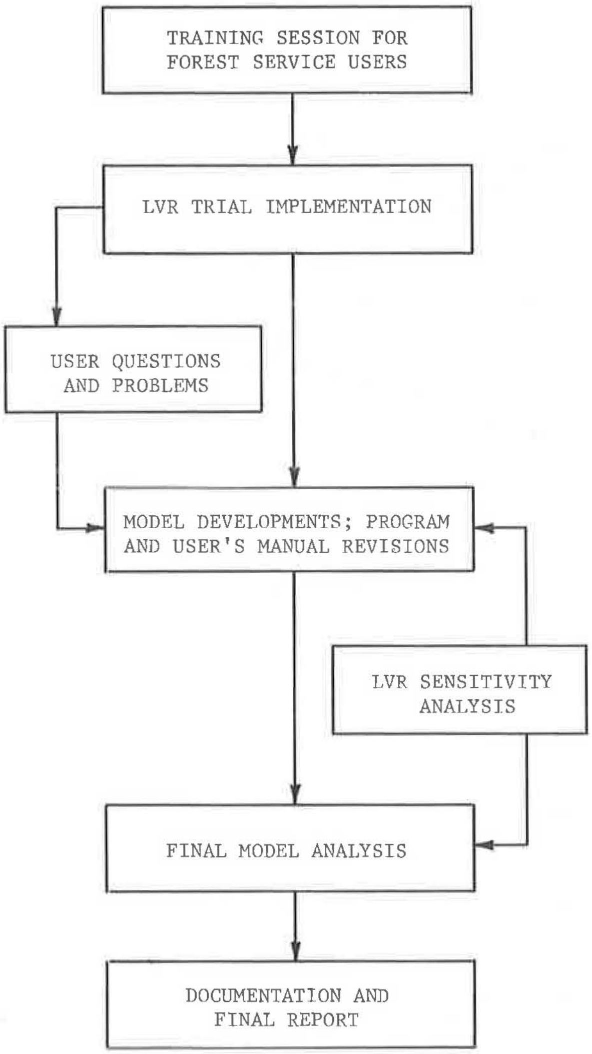
LVR Training Sessions

A total of four training sessions were given by the project staff to introduce LVR to new Forest Service users. Two of these sessions were held in Portland, Oregon (Region 6), one in Atlanta, Georgia (Region 8), and one in Missoula, Montana (Regions 1, 2, 3, 4 and 9). A total of 70 Forest Service "students" attended the sessions.

Usually, the first day of the session included project background, a discussion of the systems approach, discussions of the models included in the program and a detailed discussion of the LVR User's Manual. The second day usually included discussion and coding of an example problem, which was prepared and executed by the participants, and scheduled time for selected individual problems of interest.

At some of the sessions, the students had difficulties setting up input files at the Forest Service Computer Center, located in Fort Collins,

Figure 3. LVR implementation flow chart.



Colorado. As a result of this problem, many of the participants indicated a desire for more "hands on" time with the computer in order to make runs using data brought from their respective Forests. At each training session an evaluation was conducted to get feed back on the adequacy of teaching aids, handout materials and presentation techniques. The results of this evaluation showed that the methods, materials and techniques were very satisfactory. In general, the project staff felt that the training sessions went very well, the participants were very cooperative and eager to learn about to use the new program. Some participants suggested future sessions be offered to refresh experienced users and introduce new ones.

LVR Model Modifications

An important benefit of the implementation procedure was the feedback of Forest Service user experience with LVR. This feedback often pointed out modifications that were necessary to correct or improve the program. Listed below is a summary of some of the changes that were made in the program as a result of this feedback.

Deflection Design Procedure. At the request of Region 6, a deflection design procedure being used in that Region was incorporated into LVR. The model was developed by Region 6 using regression analysis techniques. The model is set up to predict the time of pavement failure in a manner similar to that of the AASHTO model previously incorporated into the program. The deflection model uses pavement deflection data as obtained from a Dynaflect, and, as presently structured, is for use only with bituminous pavements.

Structural Model for Aggregate Roads. As a result of a request from the Ouachita National Forest, a change was made in the AASHTO structural model as it applies to aggregate surfaced roads. The change involved the layer checking scheme that insures the surface layer is adequately thick to support the loads for the soil support value of the underlying granular layer on the subgrade. Since the surface layer for an aggregate road does not perform the same function, structurally, as the surface layer of an asphalt road, this check was deemed inappropriate and is now bypassed when designing aggregate surfaced roads.

Non-Traffic Deterioration Parameters. During the early sensitivity analysis, certain effects caused by non-traffic deterioration parameters were noted and documented. It was determined that when the program indicates that there are no feasible designs for a given set of input data, it may be because the swelling clay parameters P2P (minimum level of PSI due to non-traffic deterioration) and BONE (rate of deterioration) force the serviceability index to drop to the unacceptable level of P2 too quickly. If P2P is less than P2, it may be impossible to remedy this by relaxing any of the constraints related to cost, traffic, or construction. To inform the user of this situation when it occurs, the following message was incorporated in the program: "Non-traffic deterioration parameters are too restrictive for all possible designs. Decrease time to first overlay or reconsider values for P2, P2P, and Bone."

Changes in Program Code. Minor modifications were made, usually the result of queries or "bugs," found in the execution of the program. Examples where changes were made include the cumulative traffic model, the cumulative aggregate loss function, and the rehabilitation strategy for aggregate surfaced roads. As these problems were exposed, the program code was checked to determine if the models were executing correctly. Corrections were made and the program was rerun to verify that it was operating properly.

Documentation

A User's Manual was prepared during the development phase of this project. The manual was continuously updated as changes occurred in the program or suggestions were made to improve the input instructions. To aid in understanding the LVR program, and to make changes in the program easier, a three level documentation of LVR was also developed for the Forest Service.

A detailed computer-generated flow chart, along with a complete cross-referenced listing on the Fortran code, is provided primarily for the computer analyst or engineer who wants to know either exactly how the program operates or desires to make a model change for updating the program. The second level of the documentation is for the engineer who wants to have a knowledge of where the different models are located within the program, but does not want to go through the very detailed flow chart. This includes a brief description of each subroutine and a list of key models along with appropriate references on model development. For the engineer/administrator who is interested only in the general logic of how the program works, a conceptual flow chart was developed with brief explanations of the subroutines. These different levels of documentation should provide appropriate coverage of the program for each of the anticipated types of users of the documentation.

Trial Implementation

The trial implementation of the LVR program was designed to give Forest Service engineers, planners, and managers an opportunity to use and evaluate the program over a period of time. Trial implementation would also test the program, reveal problems, and provide useful experience under Forest Service applications in various parts of the country.

Implementation was initiated in Regions 6 and 8. Training sessions were held in December of 1976 for Region 6 and March of 1977 for Region 8. These two regions provided a range of Forest Service road types. Because of high timber resources in Region 6 the transportation system consists of mostly log hauling roads. The design condition in Region 6 usually involves heavy axle loads and relatively high traffic volumes. In Region 8, on the other hand, there is a relatively low volume of log hauling traffic, but high recreational use of the Forest Service transportation system. Also, because of high timber resource values, Region 6 tended to have more money available for roadway construction and maintenance than Region 8. It was felt that Regions 6 and 8 would provide a wide range of conditions for implementing the LVR program.

During the implementation period, there was increased interest from other regions to use LVR. Because of this interest, trial implementation of LVR was expanded to include certain other regions. It was felt that this additional implementation would benefit the study by increasing the experience base of the trial implementation, and at the same time benefit the Forest Service by introducing more users to the new system. The project staff conducted a training session in October of 1977 in Region 1 (Missoula, Montana) which included engineers from Regions 1, 2, 3, 4, and 9. Subsequent follow up discussions with these regions indicated that the program was being implemented in a number of forests in these respective regions.

During the implementation period the project staff served as consultants to users who had questions or problems with the program. This interaction served two purposes. First, it assisted the user in his understanding and operation of the program, and secondly, it was a valuable feedback source for analyzing problems and making changes to the program.

Results of Implementation

During the trial implementation of LVR, over 70 Forest Service personnel from 30 forests and 7 Regions were introduced to the program. This represents a cross-section of planners, engineers, and managers currently working in many different parts of the country. With this amount of exposure, it was felt that LVR would be tested against most, if not all, possible applications of Forest Service usage.

To ascertain the type of usage that LVR had received and any additional needs for model revision and development, the project staff conducted a questionnaire survey with some of the users in all Regions where LVR had been introduced. Depending on the Region, this survey was generally sent out after one year of trial implementation. The following is a general summary of the comments from the questionnaire.

LVR Usage

The general response from Forest Service personnel around the country was the belief that LVR was a good program and that more frequent use was expected in the future. However, the program had received less than the planned amount of use. Some users had problems with data processing, some did not have the time or resources to experiment with the program, while others did not as yet have authorization to use LVR. In some cases, trained personnel had been transfered to other duties since the training session and were no longer designing pavements. Three or four users simply did not like the program, but they were a small minority. Most gave the overall program high marks and said they planned to use it more in the future.

Problems with Models

The reply from most users concerning bituminous surfacing design was very favorable. Most had satisfactory results and few problems in executing the program. For aggregate surfaced roads, however, many users had unsatisfactory experiences. Problems ranged from unreasonable designs to excess computer execution time. Many questioned the accuracy of the aggregate loss and rut depth models which had also been of concern to the project staff and future actions are being considered by the Forest Service to correct this problem.

One common item mentioned by many users was the intention to use deflection design methods more in the future. With the number of new aggregate surfaced roads to be built, and an interest in determining overlay strategies for existing bituminous roads, it was suggested that the deflection model be expanded and improved.

Suggestions from Forest Service personnel concerning changes and additional capabilities of the models were very useful. The suggestions ranged from providing input for dust abatement cost to interaction with the Forest Service computer based Road Design System (RDS). These suggestions are being considered for present and future development.

Management's Opinion

To secure the opinion of Forest Service management, the questionnaire study included representatives of every Regional Office that participated in the trial implementation. In general, the response was very favorable towards the program, and as at the forest level, these users planned to make more use of LVR in the future.

One familiar concern was having adequate personnel and resources to spend the extra time often necessary when implementing new methods. Another concern involved the ability of the Regional Office to maintain in-house staff capable of training new users and handling user problems.

One very important comment involved the maintenance of the program itself. It was stated that considerable attention will be required to keep the models up to date, and if this is not done on a continuing basis, the program may become obsolete in 3 to 5 years. This comment is also applicable to any design method.

Overall, the Regional Office personnel showed a strong interest in continuing the use of LVR in the future. They were very aware of the importance of up-to-date information and model maintenance in the future performance of the program.

Expansion of Implementation

As a result of this trial implementation of LVR, the Forest Service now has the program operational in selected areas across the country. Because the short period of implementation has limited the usage of the program, the Forest Service plans to expand its implementation of the pavement management system. With more participation, an increased data base could be used to generate more meaningful and beneficial results. Throughout the questionnaire survey the users remarked about the importance of new pavement information. This was particularly true in two areas: aggregate road design and vehicle operating cost. Some stressed the importance of a standard road rating system. Others remarked about the unknown relationship between gravel loss, blading frequency, environment, material type, and traffic. Other users wanted information on vehicle operating costs, particularly when making economic comparisons between aggregate and bituminous surfaced roads. With LVR now operational for the Forest Service, it could be a focal point for gathering and analyzing new information during an expanded implementation period.

Forest Service Use of LVR

LVR is a natural asset to the forest engineer who is interested in maximizing the benefits of road surfacing while working within constraints of materials and cost. LVR allows the use of many kinds of constraints and decision criteria. This leads to various optimization techniques that can pick out favorable alternatives and simplify the final decision process.

Because of the large transportation system under Forest Service jurisdiction, LVR may also be an important tool for management and financial planning. The pavement management system allows the predicted performance characteristics of the surfacing to be used to predict future financial needs and manpower requirements. In many cases expenditures are based on immediate needs, and there is little opportunity to establish long-range plans. As a result there may be no funds for upgrading when a pavement deteriorates below a certain level. In other years, funds may be available, even though a pavement may not require upgrading. The capability of optimizing the expenditure of available funds is obviously an important one. The ability to consider many constraints and variables during design, to optimize available funds and materials for construction, and to assist in management and financial planning are important characteristics of LVR which will be particularly beneficial to the Forest Service users.

Summary

The procedure by which this pavement management system was developed has been long and arduous. However, the regularity and thoroughness of the interaction between Forest Service personnel and the researchers have provided significant information for use in development of the program and user's manual. The end result is that the LVR program provides the Forest Service with a powerful surfacing design and roadway management tool. Its eventual service wide implementation will result in significant additional time to enable the engineer to make project selection decisions using the best available objective information. The engineer should have more time to carefully evaluate the subjective factors affecting the acceptability of a design rather than spending available time making routine design calculations.

It should be emphasized that the program is operational at the Fort Collins, Colorado Computer Center and is receiving limited use in most of the Forest Service. Currently, certain improvements are under development and others, that are in response to implementation feedback, are being planned. Design of several projects has been completed and some jobs are already under construction. It is expected that a significant number of additional design projects will be completed using LVR during the next few years as service wide implementation is realized.

One other important feature of this work is that the program is modular in form and can be updated as new developments occur. This modular feature permits incorporation of new models into the program as they are developed. The strategy for development of new models comes from user feedback and sensitivity analyses.

In this research, we have successfully accomplished the completion of a conceptual study, the development of models necessary to complete the system program, and the completion of a trial implementation phase. Yet, the most difficult part of this work has not been completed: the accomplishment of service wide implementation of LVR which involves changing an established procedure in a very large engineering organization. Such a procedural change is always a time consuming and difficult process and is the part of improving design procedures where failure most often occurs. The researchers have finished their work and the sponsor representatives who are most familiar with the work have evaluated and accepted the product. Now, the engineer who has no vested interest in the work is confronted with a different procedure without the advantage of personal knowledge of the background of the work nor its capabilities. He now becomes the critical element in the implementation process. How he responds to the implementation attempts will be influenced by his perception of the value of the system to his own workload, and the availability of specialized training to teach him how to use the procedure and answer his questions on trial solutions. Such coordination and training support work will be critical in evaluating the probability for successful service wide implementation.

Acknowledgments

The success of this study is due in no small part to the contributions of the engineers and administrators of both the U.S.D.A. Forest Service and Council for Advanced Transportation of the University of Texas at Austin. Without their significant sacrifice of time and energy, the difficult cooperative decisions that were made could not have been made. The result of this work was the successful development of a pavement design and management system that met most of the objectives established for these projects. The assistance and cooperation of all those involved, including the student-engineers who provided valuable feedback, is gratefully acknowledged and the success of the projects was enhanced by these cooperative efforts.

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HIGHWAY SAFETY REQUIREMENTS FOR LOW-VOLUME RURAL ROADS

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This paper summarizes research that was undertaken to reevaluate the safety needs on low-volume rural roads. Based on a series of functional analyses relating safety performance to specific design and operational elements, a set of revised guidelines was developed. The revised guidelines apply to total roadway width, horizontal curvature, roadside design, speed signs, curve warning signs, centerline markings, and no-passing stripes. These guidelines are proposed to supplement the existing national policies, with each revised guideline either replacing or clarifying the existing national guideline. The widespread application of the revised guidelines should provide for more consistent design and traffic control of low-volume rural roads consonant with a rational balance between highway investment, highway safety, and traffic service.

Low-volume rural roads, those carrying 400 vehicles per day or less, are the backbone of the U. S. rural economy. State "farm-to-market" roads, county roads and township roads provide the accessibility required by agricultural commerce. Also, forest roads and park roads are necessary for the operation, maintenance, and accessibility of national forests and parks.

Although low-volume rural (LVR) roads only carry about 8 percent of the total U. S. highway travel, their economic importance in the national highway program is recognized because they constitute 2 out of every 3 kilometers of public highway. Because they are the largest single class of highway, objective guidelines for their design and operation are imperative to achieve a reasonable balance between cost and safety effectiveness. The bulk of the present LVR road system has been built using design and operational practices that have evolved from subjective experience and judgment rather than from an objective evaluation of quantifiable performance.

National guidelines for the design of LVR roads are contained in the 1971 AASHTO publication "Geometric Design Guide for Local Roads and Streets." For traffic control devices, the basic guidelines are presented in the "Manual of Uniform Traffic Control Devices." But, because these national guidelines reflect more the safety needs of primary highways, their application to the reconstruction of existing LVR roads is continually being questioned in a time when local highway agencies must spend a majority of their limited funds for highway maintenance.

In designing and operating highways for safety, LVR roads have one intrinsic edge over higher-volume highways because of a considerably lower probability of vehicle-to-vehicle collisions. The basic requirements for the minimization of single-vehicle consequences, however, are similar for all roads. In this area, maximum safety requires wide roadways and shoulders, clear and flat roadsides, gentle alignment, and high quality traffic controls and informational signing.

When considering safety on LVR roads, local highway agencies have been faced with a dilemma. On one hand, the agency would like to provide the same high-type design and operational features as on the primary highway system. On the other hand, the cost of providing this degree of safety often conflicts with the agency's philosophy of economic expediency. Because of this dilemma, LVR roads have historically been designed and operated at minimal cost with minimal overt attention to safety.

Now, the basic scenario of the highway program is changing from the massive road building campaign of the 1950s and 60s toward a concerted effort to rehabilitate existing highways. As this new emphasis mounts, the tendency is for federal matching funds to require that highways, regardless of their functional classification, be redesigned to meet all current standards. And, current standards tend more to reflect the needs of primary highways and, therefore, could require extensive and costly reconstruction of existing LVR roads. Highway agencies express increasing concern on this trend because it would force them to spend unreasonably large amounts of money for the rehabilitation of LVR roads. The alternative, which is more likely, however, is for the highway agencies to avoid these apparently unjustified costs by not implementing any LVR road improvements at all.

What this discussion points to is the need for objective design and traffic control guidelines that will strike a rational balance between maximum safety and minimum cost for LVR roads. With these guidelines, highway agencies could determine where and when to improve LVR roads within the framework of highway rehabilitation for the entire highway system.

Table 1. General Accident Statistics.

	Local Rural Roads	Total Roads	Percent Local of Total
Kilometers[a]	3,555,857	6,141,343	58.0
Million vehicle kilometers/ year	136,248	2,140,268	6.4
Average ADT	105	955	9.1
Fatal accidents/year	4,299	39,993	11.0
Injury accidents/year	156,528	1,861,131	8.4
Fatal accidents/million vehicle kilometers	3.16	1.87	169.0
Fatal and injury acci ents/ million vehicle kilometers	1.18	0.89	132.0
Fatal and injury accidents/ kilometer year	0.0452	0.311	14.6

[a]One kilometer = 0.62 miles

The objectives of this research were to:

1. Evaluate existing geometric design and traffic control guidelines, requirements and criteria with regard to their applicability and relevancy to the safety of roads carrying low traffic volumes (under 400 vehicles per day) at normal and reduced speeds.
2. Identify design and traffic control elements for which modifications of guidelines should be considered and recommend interim safety guidelines for low-volume rural roads.
3. Develop a systematic approach for collecting additional information related to safety requirements for low-volume rural roads.

Current Safety Performance of Low-Volume Rural Roads

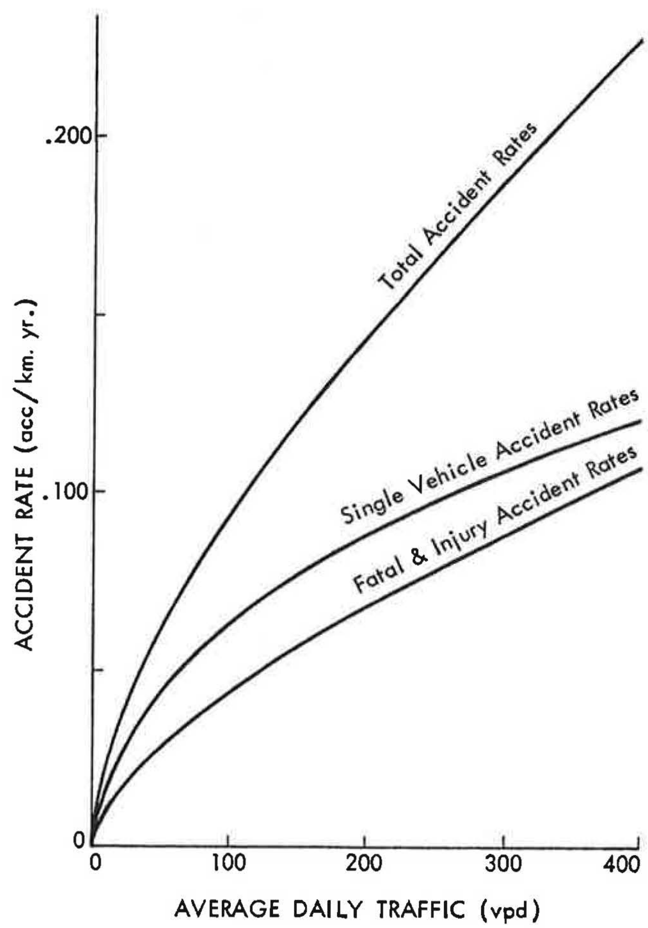
In analyzing the safety requirements for LVR roads, it is first important to dimension their current safety performance.

Table 1 shows national statistics for "Local-Rural" roads, which are basically county and township roads with an average ADT of 105 vpd. Although these roads constitute 58 percent of the total U.S. public road system, they experience only 11 percent of the fatal accidents and 8.4 percent of the injury accidents. These statistics indicate that the average frequency of fatal plus injury accidents is one every 22.1 kilometers (13.7 miles) per year on these LVR roads.

These national statistics, together with some other empirical data found in the literature were used to generate the best-fit curves of total accidents, injury plus fatal accidents, and single-vehicle accidents shown in Figure 1. The total accident rates range from 0.061 accidents/km/year (0.098 accidents/mile/year) at 500 vpd to 0.228 accidents/km/year (0.367 accidents/mile/year) at 400 vpd. In other words, the average road carrying 50 vpd will have one accident per year for every 16.4 kilometers (10.2 miles), and the average road carrying 400 vpd will have one accident per year for every 4.3 kilometers (2.7 miles).

The generated rates for injury plus fatal accidents are 47 percent of the total accident rates. The generated rates for single-vehicle accident as a percentage of total accidents range from 52.9 percent at 400 vpd to 71.4 percent at 50 vpd.

Figure 1. Estimated safety performance of existing LVR roads.



Another dimension of safety performance is the proportion of total hazard contributed by various accident types. Defining hazard as the annual number of fatal and injury accidents per kilometer, and weighting the different kinds of accidents by their average severity (percent of fatal plus injury accidents), single-vehicle accidents were found to have the majority contribution to LVR road hazard. The percent of the total fatal and injury accidents contributed by single-vehicle accidents ranges from 61.1 percent for roadways with 400 vpd to 77.8 percent for roadways with 50 vpd.

Another way of looking at the current safety performance of LVR roads is to evaluate the impact of accident costs. Using National Highway Traffic Safety Administration costs by accident severity class and applying the percent of accidents by severity class determined for Figure 1, the average cost of an accident on LVR roads was estimated at $9,500. Applying this average cost to the generated accident rates yields an average cost of accidents per kilometer of LVR road ranging from $413 ($665/mile) for a road carrying 50 vpd to $2,217 ($3,570/mile) for a road carrying 400 vpd.

These accident rates and costs begin to draw a picture of LVR roads that indicates the difficulty of making safety improvements that have any recognizable impact on the overall safety performance of LVR roads. Given a goal of a 25 percent reduction in accidents, an average of only $190 per year could be justified per kilometer of LVR road if a cost-benefit balance is to be achieved. What this suggests is that, even with the safety-conservative (high) unit values used for the cost of accidents, only relatively low-cost kinds of improvements can be justified on most LVR roads.

Interrogation of National Policies

The interrogation of national policies on geometric design and traffic control elements was conducted early in the project period to identify the standards, criteria, and guidelines currently applicable to LVR roads and to evaluate their functional suitability to the safety performance of LVR roads.

The 1971 Manual on Uniform Traffic Control Devices (MUTCD) is the national policy on traffic control devices. If the MUTCD is interpreted literally, the only traffic control devices that are mandatory on LVR roads are crossbucks at railroad grade crossings. All other devices have generalized warrants or otherwise discretionary application. In evaluating the application of the MUTCD to LVR roads, five traffic control devices appeared to require further clarification regarding their safety requirements on LVR roads. These devices, which are discussed further in the next section of this chapter are: speed signs, stop signs, curve warning signs, centerline markings, and no-passing stripes. Although most of the other regulatory and warning devices might apply under certain circumstances on LVR roads, this application must remain discretionary because of their unclear relationship to safety performance.

The national policies on the geometric design of LVR roads are contained in two AASHTO publications, "A Policy on the Geometric Design of Rural Highways, 1965" (AASHTO Bluebook) and "Geometric Design Guide for Local Roads and Streets, 1970." The AASHTO Local Road Guide mainly summarizes the parts of the 1965 AASHTO Bluebook pertaining to LVR roads.

The major differences in the Local Road Guide relate to the specification of design speeds. In this change, minimum design speeds ranging from 32.2 kph (20 mph) to 80.5 kph (50 mph) are specified depending on the ADT and type of terrain on the LVR road. Lower ADT's and more severe terrain justify lower minimum design speeds, and higher ADT's and more level terrain justify higher minimum design speeds. These design speed specifications allow a balance between the objectives of safety, service, and economy consistent with roadway function and expected operating speeds.

The design elements identified as pertinent to LVR roads and the general evaluation of suitability of their AASHTO guidelines to the safety performance of LVR roads are as follows:

1. Suitable Safety Requirements for LVR Roads/ Requirements Based On General Analysis of Tradeoffs Between Safety, Service, and Economy.
   - Highway Grade
   - Cross Slope
   - Shoulder Cross Slope
   - Structure Width
2. Suitable Safety Requirements for LVR Roads/ Requirements Based On Objective Functional Analysis of Safety Performance Using Design Speed as Basic Criterion.
   - Stopping Sight Distance
   - Passing Sight Distance
   - Corner Sight Distance
   - Horizontal Curvature
   - Vertical Curvature
3. Questionable Requirements for LVR Roads/ Requirements Not Based On Analysis of Trade-offs Between Safety, Service and Economy.
   - Total Road Width (traveled way plus shoulders)
   - Shoulder Width
   - Roadside Design (guardrail, curbs, side slopes, clear zone, etc.)

The elements in the third category are discussed further in the next section of this paper. For the most part, these design requirements call for dimensions that are much greater than those needed for acceptable safety at a reasonable cost.

Development of Revised Requirements

The development of revised safety requirements was undertaken for the eight traffic control and geometric design elements that were identified as having questionable national standards or guidelines as they apply to LVR roads. The elements evaluated were speed signs, curve warning signs, stop signs, centerline markings, no-passing stripes, roadway width, shoulder width, and roadside safety design.

Revised safety requirements were developed for most of these elements based on functional analyses, probability of conflict analyses, and cost-effectiveness analyses. The analyses were conducted using available data where possible and safety-conservative assumptions where data were not available. The term "safety-conservative" refers to assumptions that overtly favor safety in the analysis. By so doing, if errors are made in deciding appropriate requirements for design and operational elements, the errors will favor safety at the expense of highway investment, rather than the opposite.

The following discussion summarizes these developments.

Speed Signs

For most highways, drivers tend to judge their appropriate safe speed according to the geometric design, traffic characteristics, and roadside development of the highway. This would suggest for LVR roads, that because of minimum roadside friction and relatively infrequent encounters with other vehicles, that geometric design elements are the primary determinants of vehicle speeds. Without the other controls, however, drivers might tend to overdrive LVR roads except where directly influenced by physical constraints such as horizontal curvature. For this reason, speed limit signs keyed to the design speed of the highway appear to be an important adjunct to the safe operation of LVR roads.

In keeping with the proposed premise of a direct correspondence between design speeds and operating speeds, all LVR roads should have regulatory speed limit signs displaying their design speed. Signs should be placed at frequent enough intervals so that drivers will see them for almost all expected trips. Also, the speed limit should have zoned values that change as often as needed to maintain correspondence with localized general design speeds.

This practice will provide a consistent display and guide to drivers indicating the maximum operating speed for LVR roads. For drivers who are good judges of geometric design conditions for setting their maximum operating speed, the speed limit signs will reinforce their judgement. For drivers who normally overdrive the geometrics, the speed limit signs will provide a persistent reminder of why they continually experience discomfort.

Shoulder Need

An evaluation of several studies in the literature indicates conflicting results regarding the general safety effectiveness of highway shoulders. Then too, further analyses of some of the studies, which show that accident rates decrease with increasing shoulder width, indicates that the studies lacked statistical control for traffic volume. Therefore, what was really found was the relationship that shows decreasing accident rates with increasing traffic volumes.

The primary function of shoulders are to provide additional width for tracking corrections, head-on clearances, emergency stops, and leisure stops. The analysis of the tracking and head-on clearance requirements, treated separately in the next section of this report, indicates that shoulders are needed to satisfy reasonable tracking error recovery at design speeds above 72.5 kph (45 mph).

A Poisson probability analysis was used to evaluate the need for shoulders to accommodate emergency and leisure stops. The relative hazard of a highway with no shoulders can be estimated by evaluating the additional conflicts created by vehicles stopped on the traveled way rather than on a shoulder. Vehicles stopped on the traveled way present a hazard, first, to following vehicles and, second, to opposing vehicles when following vehicles pull into the left-lane to pass the stopped vehicle. The hazard to following vehicles, per se, is judged as insignificant if adequate stopping sight distance has been provided. With adequate stopping sight distance, the following vehicle driver should have more than enough time and distance to either stop or pull into the left-lane. The critical situation, therefore, involves the head-on conflicts created by a stopped vehicle.

The expected conflict rates were calculated using values for the frequency of emergency and leisure stops reported in the literature and by assuming Poisson arrivals for both following and oncoming vehicles. The expected conflict rates range from one every 27 years per kilometer of 50 vpd roadway to 19 per year per kilometer of 400 vpd roadway. An order of magnitude comparison shows that a road carrying 3,000 vpd is expected to have about 2,200 of these conflicts per kilometer per year. This would suggest that the hazard associated with stopped vehicles on LVR roads is relatively insignficant.

The conflict rate for the higher volume LVR roads might be considered as justifying shoulders to accommodate stopped vehicles. But, as discussed earlier, the next section of this report already shows justification for shoulders for the higher (more critical) design speeds, which generally correspond with the higher ADT categories. Therefore, no separate justification for shoulders based on shadowing stopped vehicles is recommended for LVR roads.

Total Road Width

Total road width is defined here as the width of traveled way plus shoulders, if present. Table 7 of the AASHTO "Geometric Design Guide for Local Roads and Streets," indicates a previous lack of functional analysis regarding road width. To say that the road width requirement for 50 vpd at 32.2 kph (20 mph) is the same as for 400 vpd at 80.5 kph (50 mph) seem inconsistent both with relative safety and with economic efficiency. What is apparently needed is a safety analysis that would relate road width to design speed. Also, if road width was related to design speed like the current requirements for horizontal and vertical alignment, the driver would be able to more readily relate his maximum safe speed to what he sees.

Two traffic conditions are readily apparent in analyzing safety requirements for total roadway width. These are (1) the clearances needed when two opposing vehicles pass, and (2) the lateral width needed to make a tracking correction without encroaching on the roadside.

The clearance requirement is the summation of two vehicle widths, two outside clearances, and one inside clearance. At very low speeds, the total roadway width need only be slightly more than the width of two vehicles. As speeds increase, the lateral margin for error is sensitive to the speed, requiring greater road widths to accommodate the safe passing of opposing vehicles.

The tracking requirement is a function of the initial lateral position of the vehicle, the speed of the vehicle, the perception-reaction time of the driver, the skid resistance of the pavement, and the angle of the tracking correction needed. As speeds increase, the ability to avoid a roadside encroachment is very sensitive to speed, requiring greater road widths to accommodate safe vehicle tracking.

Roadway width requirements for safe tracking were computed for various design speeds such that the tracking correction recovery at all encroachment angles was equivalent to that provided by a 11-meter (36-foot) roadway width (two 3.66-meter lanes and 1.83-meter shoulders) at 96.6 kph (60 mph). Lateral clearances to opposing vehicles were related to design speed and traffic volume such that the total roadway width accommodated reasonable frequencies of

Table 2. Minimum Road Width Requirements

Design Speed (kph)[c]	Total Road Width Requirements (Meters)[a b]			
	Lower % Busses & Trucks (as specified below)		Higher % Busses & Trucks (as specified below)	
	< 28% for 0- 50 ADT < 12% for 51-100 ADT < 7% for 101-200 ADT NA for 201-400 ADT		≥ 28% for 0- 50 ADT ≥ 12% for 51-100 ADT ≥ 7% for 101-200 ADT All% for 201-400 ADT	
	Infrequent Trips by Farm Machinery[d]	Frequent Trips by Farm Machinery[d]	Infrequent Trips by Farm Machinery	Frequent Trips by Farm Machinery
32.2 kph	5.5m	6.7m	6.1m	7.3m
40.2	6.1	7.3	6.7	7.9
48.3	6.1	7.3	6.7	7.9
56.3	6.7	7.3	7.3	7.9
64.4	6.7	7.9	7.3	8.5
72.4	7.9	7.9	7.9	8.5
80.5	9.2	9.2	9.2	9.2

[a] 1m = 3.28 ft.
[b] Widths above 7.3 meters (24 ft.) include appropriate shoulder widths.
[c] 1 kph = 0.621 mph.
[d] The determination of "frequent" and "infrequent" are at the discretion of the designer.

head-on meetings of busses and/or large trucks.

Table 2 presents the proposed revisions for total roadway width on LVR roads. Where values exceed 7.3-meters (24-feet), shoulders should be provided as part of the total width. When comparing with the sum of pavement width plus shoulder width proposed in the AASHTO Local Road Guide, the revised values are smaller for the lower design speeds and larger for the higher design speeds.

Four values of total roadway width are given for each design speed in Table 2. These four values derive from four different combinations of design vehicle widths used in the head-on clearance determination. With the design speed established, selecting the appropriate road width value depends, first, on the percent that busses and large trucks are of the highway ADT, and, second, whether the movement of large farm machinery is frequent enough to justify a wider roadway. The deciding values for the percentage of busses and large trucks are different for each ADT range as shown at the top of the table. In considering whether to design for the movement of large farm machinery, the definitions of "frequent" and "infrequent" are left to the discretion of the designer.

Curve Design and Warning Signs

A vehicle tracking model similar to that used to evaluate total road width on tangent was used for horizontal curves. The initial idea was to evaluate the adequacy of these previously developed widths for vehicle tracking on horizontal curves. What was found was that, rather than width, vehicle tracking was most sensitive to degree of curve.

The modification to the general design of highway curves on LVR roads relates to curves with design speeds that are lower than the general highway design speed. Although this practice is not generally recommended, it may be the only practical alternative in mountainous terrain, for example. Then too, many older existing highways have a highway curve with a design speed lower than the general highway design speed. These are the curves that are usually marked with curve warning signs and advisory speed plates.

The same kind of tracking analysis described above demonstrates, for the total roadway widths proposed in Table 2, that certain highway curves with design speeds below the general highway design speed can satisfactorily accommodate recovery from tracking errors by vehicles traveling at the highway design speed. Table 3 presents this correspondence as an allowable but not generally recommended practice. A highway curve with this allowable tolerance should always be marked with curve warning signs and with advisory speed plates displaying the design speed of the highway curve.

Stop Signs

The accident reduction effectiveness of installing two-way stop signs was predicted using (1) a Poisson probability of conflict analysis to estimate the annual number of right-angle accidents for various combinations of intersecting traffic volumes on LVR roads; and (2) an estimate of the per-

Table 3. Minimum Design Speeds for Horizontal curves that deviate from the general design speed of the highway but display curve warning signs and advisory speed plates.

Highway Design Speed (kph)[a]	Minimum Design Speed of Deviant Curve (kph)
32.2	32.2
48.3	40.2
64.4	48.3
80.5	56.3

[a] 1 kph = 0.621 mph

centage of accidents reduced for two-way stop control taken from NCHRP Report 162. Table 4 shows the predicted accident reduction for two-way stop control for various traffic volume combinations.

Table 4. Expected annual accident reduction of two-way stop control at LVR road intersections.

Road A ADT	Road B ADT 50	100	200	400
50	.0029	.0058	.0117	.0234
100	.0058	.0117	.0234	.0468
200	.0117	.0234	.0468	.0936
400	.0234	0.468	.0936	.1872

Using these effectiveness measures, the average accident cost of $9,500, an estimate of the annualized cost of stop sign installation, and the increased operating cost of $0.021 per vehicle stop reported by Anderson et. al., the benefit-cost of two-way stop sign installation was evaluated on LVR roads. The benefit-cost analysis indicates that the increased costs are greater than the safety benefit even for a 100% reduction in right-angle accidents. Therefore, stop signs are not generally justified at the intersection of two LVR roads.

Because this analysis was based on average expected values, it should be recognized that the present discretionary warrants for stop control in the MUTCD are appropriate for LVR roads. Special problems with sight restrictions or with the assignment of right-of-way, particularly when a LVR road intersects a higher-volume through highway, should warrant consideration of stop control on LVR roads.

Centerline Markings

No empirical data are available to show the safety effectiveness of centerline stripes on two-lane highways. The primary function of the centerline stripe is to guide drivers in judging the proper clearance interval to opposing vehicles.

To visualize the nature of the problem, a Poisson probability analysis was used to predict the expected number of head-on meetings for various LVR road traffic volumes. This yields the following expected rates.

ADT	Expected Number of Head-on Meetings Per Kilometer Per Day
50	0.9
100	3.9
200	15.6
300	34.8
400	62.1

With these rates, many trips are taken on LVR roads without meeting an opposing vehicle.

The need for centerline markings was also evaluated on a benefit-cost basis using the accident rates and costs generated previously and assuming a 5% reduction in total accidents as reported in NCHRP Report 162. Using a centerline cost of $124/km ($200/mile), a 1.5-year life marking, and the $9,500 average cost of accidents, the benefit-cost balance was found at an ADT of 300 vpd. Therefore, centerline markings are warranted on paved LVR roads when the ADT equals or exceeds 300 vpd.

No Passing Stripes

No empirical data are available on the safety effectiveness of no-passing stripes. The primary function of no-passing stripes is to prevent passing maneuvers where limited sight distance would make passing unduly hazardous.

To visualize the nature of the problem, a Poisson probability analysis was used to predict the expected number of head-on conflicts created by passing maneuvers. This yields the following expected rates for various LVR road traffic volumes:

ADT	Expected Annual Number of Passing Conflicts Per Kilometer
50	0.01
100	0.11
200	0.89
300	2.99
400	6.87

If we compute similar conflict rates for higher traffic volumes, the expected number for 2000 vpd for example is 900 per kilometer per year, or well over 100 times that for 400 vpd.

Based on the safety-conservative conflict rates calculated and the order of magnitude comparison with higher volume roadways, no-passing stripes do not appear to be justified on LVR roads. This is particularly true because as demonstrated in a study by Jones, drivers tend to decide to pass more on the availability of adequate passing sight distance than on the presence or absence of no-passing stripes.

A benefit-cost analysis similar to that used for centerline markings was also conducted for no-passing stripes. This analysis indicates that the balance between striping cost and accident benefits is at traffic volumes much higher than 400 vpd.

Roadside Design

The AASHTO Local Road Guide presents general guidelines for roadside design for safety. It's suggestions of 3.1-6.1 meter (10-15 foot) roadside clear zones and 4:1 or flatter side slopes are related to desirable safety performance and should be retained. These suggested values, however, are recognized as idealistic objectives in a "more is better" continuum as applied to existing LVR roads with limited rights-of-way.

A more realistic approach to roadside safety design on LVR roads depends on achieving a balance between the cost and safety effectiveness of the design treatment. For this purpose, these guidelines recommend: (1) the use of the roadside hazard model presented in a report by Glennon to compare the relative hazard reduction of various roadside safety treatments; (2) the use of a multiplier of 4 to modify the referenced model for highway curves; (3) the use of the accident cost values by severity type presented in NCHRP Reprot 162 to compute the benefits of the various hazard reductions; and (4) the application of local values for the cost of roadside safety treatments to compute the benefit-cost balance for the various roadside treatments.

Although the application of this procedure to LVR roads (using typical cost ranges for various treatments) indicates that individual roadside safety treatments yield very small safety contributions, some low-cost improvements do appear to be cost-effective especially on highway curves. For example, on highway curves, tree removal and breakaway signposts, utility poles, and mailboxes appear to be cost-effective for all LVR road traffic volumes and all reasonable unit costs of treatment. On highway tangents, these same improvements do not appear as cost-effective except for the higher (say, greater than 200 vpd) LVR road traffic volumes.

Guardrail placement on steep slopes, the removal of unnecessary guardrail on flat slopes, and the flattening of steep but low embankments also appear to be cost-effective on highway curves for the higher LVR road traffic volumes. All other kinds of roadside safety treatments including placing guardrail at fixed objects and moving fixed objects laterally do not appear to be cost-effective.

Recommended Research

The intent of Task 3 of this research was to recommend follow-on data collection activities leading to multi-variate analysis relating highway design and traffic control elements to highway accidents on LVR roads. Review of several researches, however, demonstrates the futility of these kinds of studies, even for primary highways. And, of course, several of the probability analyses of this report clearly demonstrate that the frequencies of various critical events on LVR roads are very much smaller than on primary highways. Because of these factors, dependency on discrete empirical studies to isolate any functional relationships would not only be cost-prohibitive but potentially fruitless.

Although the multi-variate analyses described above do not appear feasible, some other kinds of studies might be helpful to either verify or modify the revised safety requirements developed in this project. For example, several of the developments used the safety-conservative assumption (either expressed or implied) that LVR roads have a 50-50 directional traffic split during all periods of the day. A study of continuous traffic counts on LVR roads with different ADT's would not only show just how conservative the 50-50 assumption is, but would also measure hourly volumes to verify the efficacy of the average hourly volumes and 18-hour traffic-flow periods assumed.

Two basic kinds of studies are recommended to verify, modify, or add further depth to the developments of this research. One study would collect accident data on LVR roads to draw a clearer picture of the current safety performance of LVR roads. The other study would collect on-site data of traffic characteristics on LVR roads for the purpose of verifying the revised safety requirements. A brief discussion of these studies is given below.

Accident Studies

Accident studies could be conducted at one of three levels of detail. The first level would compile accident data on LVR roads in general. The second level would attempt to further classify these data by several traffic volume categories for LVR roads. And, the third level would add to the second level by relating the accident data to some general quality measure (e.g., high, medium, or low-type design) of individual roads. In proceeding from each level to the next higher level, the difficulty and effort involved in collecting data becomes more demanding and the feasibility of study becomes more uncertain because of limitations on existing data sources.

For the first level of study, accident data could be obtained from those states such as Missouri and North Carolina that have both many kilometers of LVR roads on the state highway system and accident records for those roads. Although the states may not be able to completely isolate LVR roads (400 vpd or less), some other classification may provide a sample that is mostly LVR roads.

The kinds of data desired for the first level of study include accident type, severity, and location. These kinds of data would provide general statistics on the proportions of the various accident types and would allow relative comparisons such as: (1) single versus multiple-vehicle accidents; (2) intersection versus mainline accidents; and (3) accidents on curves versus accidents on tangents. Adequate statistical reliability for this level of study would require a sample of about 16,000 kilometer-years of accident data.

Although the second level of study would add considerable depth to the first level, its feasibility is not clear. Collecting traffic volume data for LVR roads is not a routine task in most jurisdictions. Therefore, some method such as using personal estimates by local highway agency personnel might have to be developed. If feasible, this level would allow classifying the comparative data of the first level into discrete traffic volume categories for LVR roads.

The third level of study, of course, is both the most desirable and the most difficult to accomplish. The goal here is to further classify the data of the second level to generally relate safety performance to some measure of design quality. Although most state highway agencies usually develop sufficiency ratings for their highway system, these ratings do not usually extend to LVR roads. Therefore, some form of either personal estimates by local highway agency personnel or on-site inspection by the project staff might be necessary.

One aspect of data collection that might ease the burden and make the second and possibly the third level of study more feasible, especially if secondary data sources are not available, is if several years of accident data are available. This would limit the number of kilometers of roadway for which some form of primary data would be necessary.

Another potential form of accident study would involve, say, 10 to 20 LVR road sections in a complete case-study analysis. Although this form of study would not be as statistically tractable as the three-level study described above, it could provide some valuable insights on the safety performance of LVR roads.

Traffic Characteristics Studies

This study could be designed to measure several traffic characteristics to verify the adequacy of several assumptions used the development of the revised safety requirements in this report. Highway sites could be instrumented with sensors and a multi-channel recorder to simultaneously measure speed, speed profile, lateral placement, hourly volume, directional split, vehicle type, etc.

For complete statistical tractability, about 320 days of data collection would be necessary. This would include, for example, an average of 4 days of data at 4 sites each of four different design classifications within five categories of LVR traffic volumes. Although this is a very expensive kind of research, the fact that several kinds of data can be collected simultaneously makes the data collection very cost-effective, especially since these kinds of data are not presently available. Then too, one possible modification to the general experimental plan described above is to eliminate one or more of the lowest volume categories. Based on the orders of magnitude of various probabilities calculated in this report, the data from the higher volume categories could probably be extrapolated to make reasonable estimates for these lower volume categories. Another expediency in the total study of LVR roads would be to conduct the accident studies and traffic characteristics studies together such that both the selection of study sites and the collection of primary and secondary data could be done simultaneously.

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A DURABLE REFLECTIVE SIGN SYSTEM FOR LOW-VOLUME ROADS

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Some reflective signs on National Forest land are subjected to extreme temperatures and snow burial. Field units noted the reflective sheeting peeling from these signs after only one winter. In 1972 the Forest Service Missoula Equipment Development Center (MEDC) began testing outdoor signs of various substrates, reflective sheeting, application techniques, and clear coatings. The goal was to find the right combination of materials and manufacturing processes to produce a reflective sign that would remain maintenance-free for 7 years. The 3M Co. agreed to take part in the testing. After five winters of outdoor exposure, several combinations of substrate, reflective sheeting, application techniques, edge seal, and clear coatings were rated free of structural failure that would require maintenance. It was recommended that outdoor reflective signs for Forest Service use be manufactured as follows: (1) Engineering-grade sheeting and letters with heat-activated adhesive (HA) on aluminum substrate--cycled twice through the heat vacuum applicator and clear coated. (2) Engineering-grade sheeting (HA) and pressure-sensitive (PS) or HA letters on high-density overlay (HDO) plywood substrate--cycled twice through the heat vacuum applicator. When PS letters are used, the sheeting is cycled through the heat vacuum applicator once before applying the message and once after. (3) High-intensity sheeting (PS) with heat-activated letters on HDO plywood substrate--cycled once through the heat vacuum applicator. Signs are equally durable with silk-screened letters. The top edges of all signs are protected with Scotchcal brand transparent film.

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Special thanks to Forest Service Regional, Forest, and District personnel of the Southwestern, California, and Pacific Northwest Regions for providing test sites and installing and monitoring test signs, regardless of weather.

Field Evaluation

Reflective signs have been installed on National Forest land to conform with the Manual on Uniform Traffic Control Devices and the Highway Safety Act of 1966. When installation began on a broad scale, field units noted the reflective sheeting peeling along the edges after as little as one winter. This deterioration appeared to be more common at higher elevations, but also occurred at lower sites when signs were subjected to extreme temperature change, snow burial, ultraviolet rays, or all three.

Investigation by the Forest Service Missoula Equipment Development Center (MEDC) revealed that snow burial, extreme temperature change, and ultraviolet rays combine to destroy reflective signing in three basic ways:

1. Peeling of the legend (message) from the background sheeting.
2. Peeling of the reflective sheeting from the substrate (base material).
3. Separating (delaminating) of the layers that make up the reflective sheeting and legend. The first evidence of this type of failure is crazing of the reflective material--minute cracks that cause peeling or delamination if cracks become large enough.

It was important to find a solution to the deterioration for two reasons:

1. Reflectorized signs were adopted to increase traffic safety, because they communicate clearly to a driver day or night; peeling and delamination destroy the reflectorized sheeting, making the sign less effective.
2. Reflectorized signs are expensive --about $8.50 a square foot on the average--and maintenance or replacement due to premature failures is extremely costly.

MEDC began a project in 1972 to outdoor test signs manufactured of various combinations of substrates, reflective sheeting materials, application techniques, and clear coatings. The goal was to find the right combination of materials and manufacturing techniques to give Forest Service units a rugged reflective sign that would remain in service 7 years without maintenance.

The Forest Service was not the only agency experiencing problems with reflectorized signs. The Federal Highway Administration, Bureau of Land Management, National Park Service, Federal Prison Industries, Inc., and various transportation departments at the State and county level were concerned and asked to participate as observers in the outdoor tests. The American Plywood Association was also interested in the tests and asked to participate.

The 3M Co., manufacturer of much reflective sheeting in Forest Service signs, agreed to take an active part in the testing; and an agreement was entered into regarding the responsibilities of both the Forest Service and 3M. In 1976 two other major reflective sheeting manufacturers, Avery International and Mitsubishi/ Seibu International, entered into similar cooperative agreements to test samples of their reflective materials.

In 1977 a cooperative agreement was also completed with Finnish Plywood Association USA to evaluate the durability of the products of the three sheeting manufacturers on Finnish birch plywood overlayed with phenolic resin. Reflexite Corp. will enter testing in the summer of 1979.

This paper is divided into two parts. Part I describes the testing of 3M Co. products over the past five winters and makes specific recommendations for manufacturing signs with 3M products. Part II discusses Avery and Mitsubishi/Seibu materials, which have undergone outdoor testing for 2 years.

Part I--3M Co.

Test Objective

The objective is to test and evaluate as many different materials combinations as practical with accepted manufacturing processes, to provide information needed to produce a durable sign that will remain in service for 7 years without maintenance.

Test Plan

In the fall of 1972, 3M Co. representatives and MEDC personnel met to design a cooperative test plan. At this meeting, guidelines for selecting sign materials, manufacturing processes, and testing methods were agreed upon. Test sites also were selected based on snowfall, extreme temperature change, and exposure to ultraviolet rays. Sites chosen were Hopewell Lake, N. Mex.--elevation, 3048 m (10,000 ft); Donner Summit, Calif.-elevation, 2134 m (7,000 ft); Mount Adams, Wash.--elevation, 1402 m (4,600 ft).

Test Sign Materials and Manufacturing. 3M and MEDC selected 50 different combinations of background sheeting, application techniques, top edge treatments, and clear coatings for initial testing. Aluminum and HDO (high-density overlay) plywood, the two basic sign substrates, were used for test signs.

The 3M Co. reflective background sheeting was engineering-grade brown with heat-activated adhesive; high-intensity green with heat-activated adhesive; engineering-grade brown with pressure-sensitive adhesive; high-intensity green with pressure-sensitive adhesive. Letters were high-intensity silver with heat-activated adhesive; engineering-grade silver with pressure-sensitive adhesive; and Control-Tac engineering-grade silver. Some letters were silk screened on the reflective sheeting.

To simulate the legend (message), the letters "E," "O," and "N" were selected because they represented the geometrical shapes found in the alphabet.

It was agreed that test signs would be produced by an impartial independent contractor. 3M would negotiate a contract with the sign manufacturer, provide all materials, and pay manufacturing costs. It was important that the sign contractor be willing to have 3M and MEDC representatives monitor the entire manufacturing process and provide technical advice. Ojo Caliente Craftsmen Cooperative, Ojo Caliente, N. Mex., was selected to produce the signs. This firm had worked closely with the Forest Service on other sign testing projects.

Evaluation of Test Signs. The Forest Service and 3M agreed on this system for evaluating test signs:

1. E--Excellent to good durability with no structural failures that would require maintenance (fig. 1).
2. L--Legible; sign message legible but maintenance would be required to prevent deterioration and for esthetic purposes (fig. 2).
3. NL--Not Legible; message unreadable, requiring immediate replacement or complete repair (fig. 3).

Figure 1. Test sign shows no structural failure that would require maintenance.

Figure 2. Test sign message is legible but maintenance would be required for esthetic purposes and to prevent further deterioration.

Figure 3. Test sign message is becoming unreadable and sign requires immediate replacement or repair.

These definitions of sign failure also were agreed to:

1. Peeling--Results when sheeting peels from the substrate; in the case of sign legend (message), when letters peel from background sheeting.
2. Delamination--Separation within the reflective sheeting.
3. Crazing--Fine cracks within the reflective sheeting. If cracks become large enough to break surface coating, peeling or delamination results.

Evaluations were to be performed as early as possible each spring by technical representatives from 3M and MEDC. Each sample would be photographed and evaluated; any difference of opinion between 3M and MEDC personnel would be settled at the test site, with MEDC reserving the right to make the final recommendation for reporting purposes. Signs that failed and would provide no further information would be removed from the test by MEDC.

Minimum critera for a sign combination to be considered successful were agreed upon: five of the six signs in the combination (two installed at each test site) would have to be rated "E" and the remaining one rated at least "L."

Outdoor Weathering Tests

Some 300 test signs were manufactured in October 1972, using the 50 different materials combinations and application techniques (tables 1-3). One hundred signs each were installed at the California and Washington test sites in December 1972. Severe weather delayed the installation of the last 100 signs at the Hopewell Lake, N. Mex., site until April 1973.

All test signs measured 20 by 35 cm (8 by 14 in). They were installed 46 to 61 cm (18 to 24 in) above ground on wood posts then transferred later to steel U-channel posts set in rows (fig. 4). Signs faced south for maximum ultraviolet exposure in summer.

In the fall of 1974 seven new sign combinations of HDO and MDO (medium density overlay) plywood and ABS (acrylonitrile butadiene styrene) plastic substrates were manufactured and installed at each test site (table 4). The MDO plywood and ABS substrates were selected to provide the Forest Service with weathering data on additional substrates

Table 1.-- Reflective signs of 3M Co. materials on aluminum substrate, placed at test sites, 1972-73.

Sign No.	Reflective materials		Edge treatment				Clear coatings				Heat application	
	Legend[a]	Sheeting[a]	Corners and edges square	Film #639[b]	#700 clear[c]	#800 clear[c]	#700 legend only[c]	#800 legend only[c]	#700 complete sign[c]	#800 complete sign[c]	Normal application	Double cycles
1F1	2270	2279	x									x
1F2	2270	2279	x	x			x					x
1F3	3270	2279	x						x		x	
1F4	3270	3279	x				x				x	
1F5	9270	3279	x	x							x	
1F6	9270	3279	x						x		x	
1F7	Silk screen	3870	x								x	
1F8	3870	2877	x	x							x	
1F9	2870	2877	x			x				x		x
1F10	2870	3877	x				x				x	
1F11	Silk screen	3870	x	x							x	
1F12	3870	3877	x			x				x	x	
1F13	2270	2279	x		x							x
1F14	2270	3270	x		x		x				x	
1F15	3870	3877	x								x	
1F16	3870	3877	x		x			x			x	

NOTE: • Numbers refer to 3M Co. product numbers.
• Background sheeting placed one-half inch below top edge of sign on Nos. 13, 14, 15, 16.

[a] The following reflective materials were used for letters and sheeting:

2270 = heat-activated, engineering-grade silver;
2279 = heat-activated, engineering-grade brown;
2870 = heat-activated, high-intensity silver;
2877 = heat-activated, high-intensity green;
3270 = pressure-sensitive, engineering-grade, silver;
3279 = pressure-sensitive, engineering-grade brown;
3870 = pressure-sensitive, high-intensity silver;
3877 = pressure-sensitive, high-intensity green;
9270 = Control-Tac engineering-grade silver.

[b] Scotchcal brand transparent film (#639) placed along top edge of sign for added protection against delamination; on signs 1F14 and 1F16, film placed on all edges.

[c] Scotchlite brand process color, #700 series used as clears and edge sealers for engineering-grade sheeting; #800 series used as clears and edge sealers for high-intensity sheeting.

Table 2.--Reflective signs of 3M Co. (high-intensity sheeting) on HDO plywood substrate, placed at test sites, 1972-73.

Sign No.	Reflective materials: Legend[a]	Sheeting[a]	Edge treatment: Corners and edges square	Corners and edges rounded	Film[b] #639	#4150[c] Clear	Paint	Clear coatings: #800 legend only	#830 complete sign	#831 complete sign	Heat Application: Normal
2F1	3870	2877	x								x
2F2	3870	2877	x		x			x			x
2F3	Silk screen	2877	x			x					x
2F4	2870	2877	x					x	x		x
2F5	Silk screen	2877		x							x
2F6	2870	2877		x		x		x			x
2F7	2870	2877		x					x		x
2F8	3870	3877	x					x			
2F9	3870	3877	x		x						
2F10	3870	3877	x			x		x			
2F11	2870	3877	x							x	
2F12	2870	3877		x				x			x
2F13	2870	3877		x		x					x
2F14	2870	3877		x				x		x	x
2F15	2870	3877		x			x				x
2F16	3870	2877		x			x	x			x

NOTE: • Numbers refer to 3M Co. product numbers.

2F15 - All edges and back received one coat of brown long oil primer and one coat of Benjamin Moore Co.'s polysilicone enamel (brown) before sheeting and second coat of enamel after sheeting.

2F16 - All edges and back received one coat of short oil primer and one coat of Benjamin Moore Co.'s polysilicone enamel (brown) before sheeting and second coat of enamel after sheeting.

[a] 2870 = heat-activated, high-intensity silver;
2877 = heat-activated, high-intensity green;
3870 = pressure-sensitive, high-intensity silver;
3877 = pressure-sensitive, high-intensity green.

[b] Scotchcal brand transparent film (#639) placed along top edge of signs for added protection against delamination.

[c] Scotchlite brand process color, #4150 series clears used as edge treatment for high-intensity sheeting.

Table 3.--Reflective signs of 3M Co. materials (engineering-grade sheeting) on HDO plywood substrate, placed at test sites, 1972-73.

Sign No.	Reflective materials: Legend[a]	Sheeting[a]	Edge treatment: Corners and edges square	Corners and edges rounded	Film 639[b]	#700 clear	Painted[c]	Clear coatings: #700 legend only	#700 complete sign	Heat application: Normal application	Double cycle
3F1	2270	2279	x				x				x
3F2	2270	2279	x		x			x			x
3F3	2270	2279	x			x					x
3F4	3270	2279	x					x	x	x	
3F5	3270	2279	x				x			x	
3F6	3270	2279		x				x		x	
3F7	9270	2279		x		x		x		x	
3F8	9270	2279		x					x	x	
3F9	9270	2279		x			x	x			x
3F10	2270	3279	x				x				x
3F11	2270	3279	x		x			x			x
3F12	2270	3279	x			x				x	
3F13	3270	3279	x			x			x		
3F14	3270	3279	x				x				
3F15	3270	3279		x							
3F16	9270	3279		x		x		x			
3F17	9270	3279		x					x		
3F18	9270	3279		x			x				

NOTE: Numbers refer to 3M Co. product numbers

[a] 2270 = heat-activated, engineering-grade silver;
2279 = heat-activated, engineering-grade brown;
3270 = pressure-sensitive, engineering-grade silver;
3279 = pressure-sensitive, engineering-grade brown;
9270 = Control-Tac engineering-grade silver.

[b] Scotchcal brand transparent film (#639) placed along top edge of signs for added protection against delamination.

[c] 3F1 = Fuller Co. long oil base prime, 1 coat (4 $\pm$ 1/2 mil), then 4 coats Fuller oil base paint applied to edges and back of finished sign.

3F5 = Same as 3F1 except Benjamin Moore Co. short oil prime.

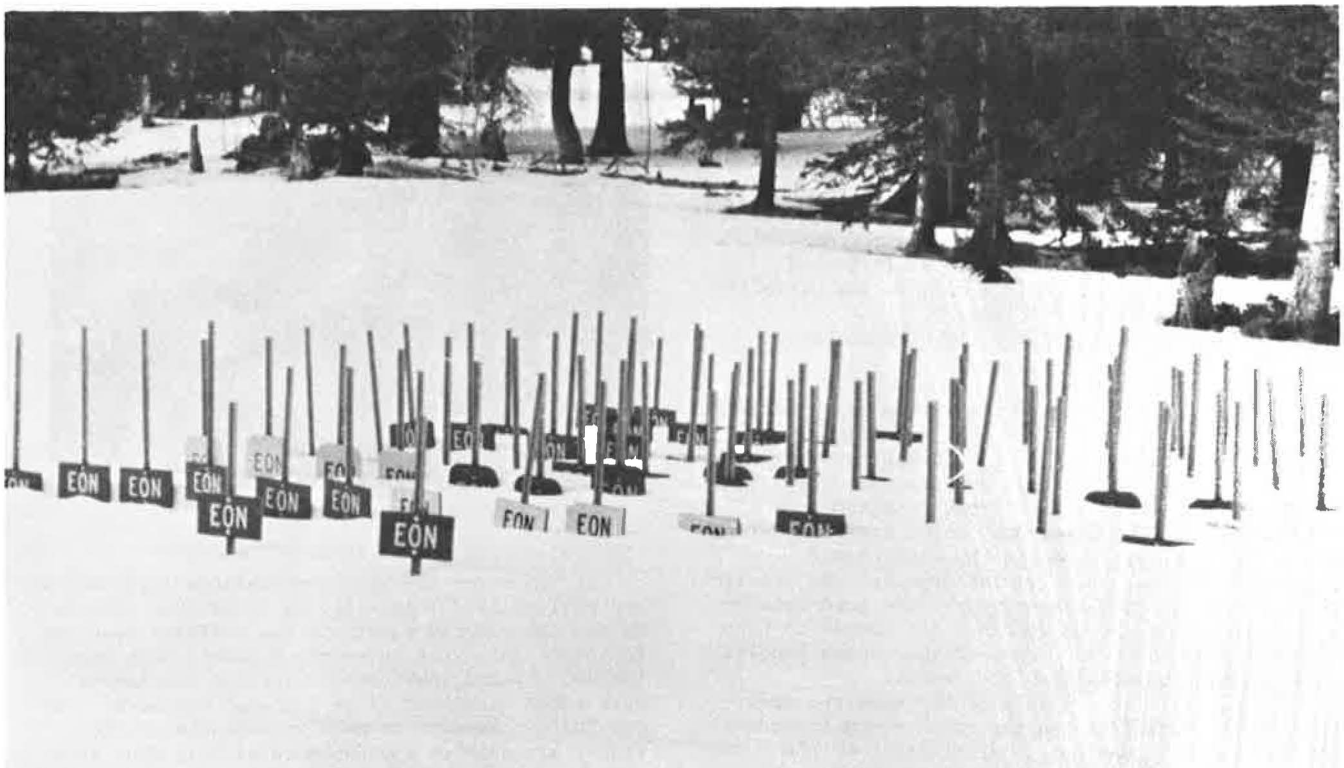
3F9 = Same as 3F1 except primer and 2 coats of paint before sheeting application; 2 coats after.

3F10 = Fuller Co. long oil base prime (1 coat) and Benjamin Moore Co. polysilicone enamel (brown) (4-1/2 $\pm$ 1/2 mil); 4 coats applied to edges and back of finished sign.

3F14 = Benjamin Moore Co. short oil base prime (1 coat) and polysilicone enamel (brown) (4-1/2 $\pm$ 1/2 mil); 4 coats applied to edges and back of finished sign.

3F18 = Benjamin Moore Co. short oil base prime (1 coat) and Fuller oil base paint (2 coats) before sheeting application; 2 coats after sheeting application.

Figure 4. 3M Company reflective signs undergoing outdoor exposure testing at Hopewell Lake, New Mexico, test site.



and manufacturing alternatives. One combination (4F3) had one letter coated with verathane; two combinations (4F6 and 4F7) had the top edge protected with two types of aluminum extrusions that were screwed to the sign. Sheeting for all combinations was applied with only a single cycle through the heat vacuum applicator, to determine the protective qualities of the verathane and the aluminum edging. The legend on all except 4F2 was hand applied.

In September 1977 12 new sign combinations using aluminum and HDO plywood were installed at the test sites to evaluate two new films (SJ8582X and SJ8583X) designed specifically to protect the top edge of a sign. Because the interest was in the bond between the substrate and the sheeting, no legends were put on the signs.

In the spring of 1978, 24 more combinations were installed to evaluate improved high-intensity and engineering-grade sheeting on aluminum and HDO substrates. Four combinations of 3M Co. materials on Finnish plywood were installed in 1977 and 1978.

Test Results

The original 50 sign combinations were evaluated in 1974, 1975, 1976, and 1977. By 1977, only six combinations had weathered well enough to meet the minimum criteria of five signs rated "E," with the sixth sign rated at least "L" (fig. 5).

The six combinations were 1F2, 2F13, 3F2, 3F3, 3F10, and 3F11. 3F3 and 3F10 were then eliminated because they largely duplicated 3F2 and 3F11.

1F2 was the only successful combination that had an aluminum substrate (table 1). 2F13 was the only successful combination that had green high-intensity grade sheeting (fig. 6) (table 2). The remaining successful combinations (table 3) had brown engineering-grade sheeting, one with pressure-sensitive adhesives and one with heat-activated adhesives.

Figure 5. 3F11 test sign rated excellent after 6 years of outdoor exposure.

Figure 6. 2F13 was the only successful sign combination that had green high-intensity grade sheeting.

3F2 had Scotchcal brand transparent film (#639) on the top edge; the legend had been clear coated. 3F3 had been clear coated on the top edge. Analysis of other combinations indicated that the film was more effective than the clear coat in protecting the top edge of a sign. It was found that placing a coat of clear over heat-activated letters was unnecessary. 3F10 had the top edge simply painted, but this was found to be unsatisfactory in many other combinations.

The manufacturing variables--background sheeting, application techniques, edge seals, top edge treatment, clear coating--used in each of the four combinations were compared to identical variables in failed signs. In no case had these manufacturing variables been the cause of a sign failure.

Five years of outdoor testing also revealed these findings:

1. Peeling of background sheeting was generally more severe on aluminum than on plywood (fig. 7). Severe crazing occurred on high-intensity sheeting where clear coating had been applied.

2. The #639 film taped across the top edge of some test signs for added protection against peeling was deteriorating; it showed the most wear at Hopewell Lake where ultraviolet exposure was greatest. But the film continued to preserve the bond between the sheeting and the substrate. (It proved to have a useful life of 4 to 5 years--3 years when exposed to large amounts of ultraviolet rays.)

3. Placing the top edge of the sheeting down 1.27 cm (1/2 in) from the top on aluminum substrate did not prevent peeling or delamination of the sheeting. Of eight signs made this way for each test site, at Mount Adams, six were rated "L" and two were removed; at Hopewell Lake, four were rated "E" and four "L"; three were rated "E" and five "L" at Donner Summit.

4. Silk screened legends weathered well. There were no failures.

Figure 7. Background sheeting generally peeled more severely from aluminum than plywood.

Of the seven new sign combinations installed in the fall of 1974 (table 4), we found that legends on the ABS plastic substrate had suffered moderate to severe delamination at the Hopewell Lake site; the MDO plywood combinations (except one sample) were rated excellent after 3 years' exposure. The one failure appears to be more a result of the higher ultraviolet ray exposure at this test site. The plywood signs with aluminum over the top edge have not peeled at the top to date.

The samples with verathane over one letter have not failed in peeling, delamination, or crazing, but the letter is turning yellow after 3 years.

Samples installed in 1977 and 1978 do not have enough exposure to evaluate durability.

Table 4.--Reflective signs of 3M Co. materials, placed at test sites, 1974.

	Reflective Materials		Substrates			Edge Treatments				
Sign No.	Legend [a]	Sheeting [a]	ABS Plastic	MDO Plywood	HDO Plywood	Corners & Edges Square	Corners & Edges Round	Flat Aluminum Over Top	Round Aluminum Over Top	Paint
4F1	3270		X							
4F2	2270		X							
4F3	3270	2279			X		X			X
4F4	3270	2279		X		X				X
4F5	3270	2279		X		X				X
4F6	3270	2270			X		X	X		X
4F7	3270	2279			X		X		X	X

Note: Numbers refer to 3M Co. product mumbers.

4F3 - Short oil base primer; Fuller oil base paint (4-½ $\pm$ ½ mil.) on edges and back; one letter coated with Verathane.

4F4 - All edges and back received five coats (4½ mils.) of Benjamin Moore Co.'s. polysilicone enamel (brown) before reflective materials applied.

4F5 - Same as 4F4 except enamel added after reflective materials applied.

4F6 - Short oil base primer, Fuller oil base paint (4½ $\pm$ ½ mil.).

4F7 - Same as 4F6 except for difference of alluminum top edge.

Two samples of 4F1, 4F2, 4F3, 4F6, and 4F7 were put at each test site; three samples of 4F4 and 4F5 were put at each site.

[a] 2270 = heat-activated, engineering-grade silver;
2279 = heat-activated, engineering-grade brown;
3270 = pressure-sensitive, engineering-grade silver.

Discussion

Test findings indicate that the service life of reflective signs can be extended, using specific combinations of materials and manufacturing processes.

Based on 5 years of experience, the test procedure adopted appears to be effective in isolating premature failures. Its primary value is in testing products under real-use conditions, which cannot be simulated in the laboratory with today's technology.

In addition to rating each sign, careful examination was made of other features and notes were taken to document findings. Failures were examined in an attempt to correlate them with the unique materials combinations and manufacturing process.

One observation was that the life of plywood veneers are extended by using plywood substrates with rounded edges and painting them with polysilicone paint; the paint does not protect the top edge of reflective sheeting, however.

Another observation was that few of the edge seals prevented moisture from entering the plywood for the 5-year observation period. Paint over a short oil base primer peeled in 2 years; over a long oil base primer, in 5 years. Three clears were used to seal the plywood edge: #700, #800, and #4150. Plywood veneers treated with these were checking and cracking within 2 years.

From these observations, and from observing signs produced according to Southwestern Region specifications, it can be shown that four coats of Benjamin Moore polysilicone paint without any primer will protect plywood edges from moisture for more than 5 years. The edges must also be rounded to help reduce the stresses in the paint film.

Conclusions

1. Specific combinations of substrates, reflective sheeting, top edge treatments, applications techniques, and clear coatings are available to substantially increase the service life of reflective signs. Four sign combinations met the desired criteria and should have a service life comparable to test goals.

2. Bonding the sheeting 1.3 cm (1/2 in) below the top of aluminum sheeting does not improve longevity.

3. MDO plywood looks like a promising substrate material, but additional exposure is needed before it should be adopted for use.

4. Improvements are needed before ABS plastics can be adopted for use.

5. An aluminum extrusion placed over the top edge of a sign appears to increase longevity. It is extremely costly and was made a part of the testing only to gain data on a method of sign protection that might be resorted to in the most extreme outdoor conditions when all other sign combinations had failed.

6. The Scotchcal brand sprint film (#639) is durable and can be used effectively but an ultraviolet inhibitor is needed to extend service life to 7 years.

7. Silk screened letters are durable. They can be considered an alternative to letters of reflective sheeting. Sheeting manufacturers' recommendations must be followed when applying the ink.

8. If a verathane coating over letters is durable, an ultraviolet inhibitor will need to be added to the verathane.

9. Although the bond between paint and sheeting is not adequate, the top edge bond is protected with #639 film.

10. Plywood edges should be rounded and painted to insure that moisture does not get into the sign so that checking and cracking of veneers can be avoided. Benjamin Moore polysilicone paint without primer will protect edges from moisture for more than 5 years.

Recommendations

The recommendations that follow are based on 5 years of outdoor testing. Manufacturing and maintenance costs were considered during testing and are reflected in the recommendations. For example, black HDO, which is impervious to moisture is recommended, reducing maintenance costs; the optional use of group 1 B grade veneers on both sides of the substrate, instead of exterior-marine grade, to lower cost; and the elimination of a special primer for the edge paint.

1. It is recommended that outdoor reflective signs for Forest Service use be manufactured with these combinations of materials and processes (they are recommended equally, and their order of presentation has nc significance):

Reflective Sign of Aluminum

Substrate: No. 6061-T6 plate stock conforming to ASTM Standard B209.

Background Sheeting: 3M Co.'s engineering-grade brown with heat-activated adhesives (#2279).

Legend: 3M Co.'s engineering-grade silver with heat-activated adhesive (#2270) or silk screened.

Manufacturing Process: (1) Double cycle through heat vacuum applicator. (2) Coat the legend with 3M Co.'s Scotchlite brand process color (#700) clear (follow instructions on container). Clear coating not required for silk-screen legend.

Top Edge Treatment: Apply 3M Co.'s Scotchcal transparent film (#639) over top edge of sign. For adequate protection, 7.6-cm-wide (3 in) film is recommended. For ease of handling and cleanliness, it should be applied in 61-cm-long (24 in) strip. On larger signs, begin taping from each outside edge and tape toward center of sign. Film should overlap at the center of sign 5 or more cm (2 in). The purpose of the film is to provide complete protection to the top edge of the sign to help prevent the sheeting from peeling from the substrate.

Reflective Sign of HDO (High-Density) Overlay Plywood and High-Intensity Sheeting

Substrate: HDO front and back. All Douglas-fir, exterior-marine grade, conforming to product standard PSI-74; or all Douglas-fir exterior plywood, PSI-74, group 1, with B grade veneers on both sides. HDO must be a 60-60 nonoiled resin impregnated fiber, black in color. Each panel should be edge-branded, marine-grade HDO EXT PSI-74 or HDO B-B G 1 EXT PS 1-74, 5-ply, 1.3 cm (1/2 in); or 7-ply, 1.9 cm (3/4 in). (Thickness will vary depending on sign size, as defined in the Forest Service procurement and manufacturing specification.)

Background Sheeting: 3M Co.'s high-intensity green with pressure-sensitive adhesive (#3877).

Legend: 3M Co.'s high-intensity silver with heat-activated adhesive (#2870) or silk screened.

Manufacturing Process: (1) Cut plywood blank. (2) Round or bevel edges to a radius of .24 cm (3/32 in); round corners. (3) Finish-sand all edges and the panel face (HDO). (4) Clean all surfaces with a tack rag. (5) Before sheeting, apply two coats of Benjamin Moore Co.'s #120-60 polysilicone enamel as a primer to all edges. (6) Apply sheeting to substrate. (7) Apply two more coats of enamel after sheeting has been applied to substrate. (8) Apply legend and cycle once through heat vacuum applicator. Do not apply clears.

Top Edge Treatment: Apply 3M Co.'s Scotchcal transparent film (#639) over top edge of sign. For adequate protection of 1.9 cm (3/4 in) plywood, 7.6 cm-wide (3 in) film is recommended. For ease of handling and cleanliness, apply in 61 centimeter-long (24 in) strips. On larger signs, begin taping from each outside edge and tape toward center of sign. Film should overlap at the center of the sign at least 5 cm (2 in). The purpose of the film is to provide complete protection to the top edge of the sign to help prevent the sheeting from peeling from the substrate.

Reflective Sign of HDO Plywood and Engineering-Grade Sheeting

Substrate: HDO front and back. All Douglas-fir, exterior-marine grade, conforming to product standard PSI-74; or all Douglas-fir exterior plywood, PSI-74, group 1, with B grade veneers on both sides. HDO must be a 60-60 nonoiled resin impregnated fiber, black in color. Each panel should be edge-branded, marine-grade HDO EXT PSI-74 or HDO B-B G 1 EXT PS 1-74, 5-ply, 1.3 cm (1/2 in); or 7-ply, 1.9 cm (3/4 in). (Thickness will vary depending on sign size, as defined in the Forest Service procurement and manufacturing specification.)

Background Sheeting: 3M Co.'s engineering-grade brown with heat-activated adhesives (#2279).

Legend: 3M Co.'s engineering-grade silver with heat-activated adhesives (#2270); or 3M Co.'s engineering-grade silver with pressure-sensitive adhesives (#2270); or silk-screened.

Manufacturing Process: (1) Cut plywood blank. (2) Round or bevel edges to a radius of .24 cm (3/32 in); round corners. (3) Finish-sand all edges and the panel face (HDO). (4) Clean all surfaces with a tack rag. (5) Before sheeting, apply two coats of Benjamin Moore Co.'s #120-60 polysilicone enamel as a primer to all edges. (6) Apply sheeting to substrate. (7) Apply two more coats of enamel to edges after sheeting has been applied to substrate. (8) Apply legend. If using heat-activated letters, cycle sign twice through the heat vacuum applicator. Do not apply clears. If using pressure-sensitive letters, cycle sheeting through heat vacuum applicator once before applying legend and once after. Do not apply clears.

Top Edge Treatment: Apply 3M Co.'s Scotchcal transparent film (#639) over top edge of sign. For adequate protection of 1.9 cm (3/4 in plywood, 7.6-cm-wide (3 in) film is recommended. For ease of handling and cleanliness, apply in 61-cm-long (24 in) strips. On larger signs, begin taping from each outside edge and tape toward center of sign. Film should overlap at the center of the sign at least 2 inches. The purpose of the film is to provide complete protection to the top edge of the sign to help prevent the sheeting from peeling from the substrate.

Part II--Avery International & Mitsubishi/Seibu International

Since the Forest Service and 3M Co. began their cooperative testing the reflective sign materials 5 years ago, other companies have entered the reflective sheeting field. For this reason, the Forest Service invited these manufacturers to supply reflective materials for testing as 3M wasdoing. Two companies, Avery International and Mitsubishi/Seibu International, concluded cooperative agreements in 1976 with the Forest Service.

A test plan identical to the one agreed to by 3M was adopted, and separate test plots for the signs of both companies were set up at the existing test sites in California, New Mexico, and Washington. Plots were physically separated from each other and the 3M Co. plots but provided the same exposure to the elements.

Signs of Avery and Mitsubishi/Seibu materials were manufactured and installed at the test sites in November 1976. Signs of aluminum and plywood measured 20 by 35 cm (8 by 14 in); those of ABS plastic, 30.5 by 30.5 cm (12 by 12 in). Signs were installed 46 to 61 cm (18 to 24 in) above ground on steel U-channel posts set in rows. Signs faced south for maximum exposure to ultraviolet rays.

Signs were evaluated for the first time in June 1977. A discussion of each company's signs and an evaluation of them after one winter's exposure follows. Because this testing has been underway for only a short time, no conclusions or recommendations are included in Part II of this paper.

Avery International

Some 28 different sign combinations were produced by Ojo Caliente Craftsmen Cooperative, under MEDC and Avery supervision, with substrates of HDO and MDO plywood, aluminum, and ABS plastic. Heat-activated and pressure-sensitive engineering-grade white and green reflective sheeting was used. Avery does not now manufacture a brown sheeting.) The letters "E," "O," and "N" were selected to simulate the legend. Letters were either reflective sheeting or silk screened on the signs. Some edges were square, others were rounded. Scotchcal film (#639) was used as a top edge treatment, and polysilicone paint and various clears were used to seal the edges. Some of the HDO plywood signs were treated with Scotchlite brand process color #700 series clears; others were not to verify if the adhesives were durable without added protection.

In all, 168 test samples were installed at the three sites in November 1976.

The first evaluation took place in June 1977. After one winter of outdoor exposure, sheeting had peeled on several of the ABS plastic and aluminum substrates.

In the fall of 1977 three combinations of prototype sheeting and letters were installed at the three test sites.

By the spring of 1978 all combinations installed at both the Mount Adams test site and the Donner Summit test site demonstrated some degree of failure (fig. 8). However, at the Hopewell Lake site, only a few of the combinations exhibited any failure. As a result, Avery removed signs except those installed in 1977 from all sites.

In response to these failures, the company developed new materials, modified original formulations, including adding ultraviolet ray inhibitors, and adding tapes to the top edge of signs. Forty new sign combinations were manufactured and installed in October 1978 (fig. 9). In addition, six combinations using Finnish Plywood substrate were also manufactured and installed at the same time.

Mitsubishi/Seibu International

MEDC and Mitsubishi/Seibu International representatives selected 42 sign combinations for testing. These were manufactured by Ojo Caliente Craftsmen Cooperative under the supervision of MEDC and company personnel. Substrates included HDO and MDO plywood, aluminum, and ABS plastic. Heat-activated and pressure-sensitive, engineering-grade brown and silver reflective sheeting underwent testing. "E," "O," and "N" were chosen to simulate the legend; in addition to the precut letters of reflective sheeting, some letters were silk screened. The edges of most plywood signs were painted with four coats of Benjamin Moore Co.'s polysilicone brown enamel; the edges of three signs were left unpainted and treated with a clear edge seal. In addition, the top edges of two plywood sign combinations were taped with Scotchcal transparent film (#639). Some signs were treated with clear coating; others were not to verify if adhesives were durable without the added protections.

Some 252 test samples were installed at the three sites in November 1976 (84 signs per site) (fig. 10). The first evaluation took place in June 1977. After one winter of outdoor exposure, we found that some of the sheeting was peeling from the ABS substrate. In the evaluation in 1978 peeling continued on the ABS plastic.

In the fall of 1977 Mitsubishi/Seibu requested the manufacture and installation of three more sign combinations (18 test signs). These signs were installed at Hopewell Lake and Donner Summit in October 1977 and at Mount Adams in Spring 1978. Combinations using Finnish plywood were installed in 1978.

Figure 8. Signs of Avery International materials after 2 years of outdoor exposure.

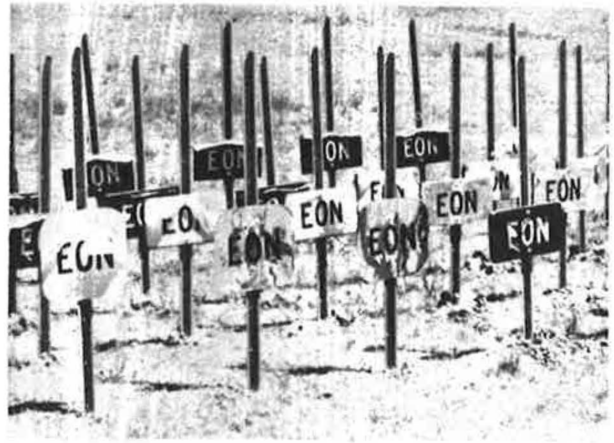


Figure 9. New Avery International sign combinations installed in October 1978.

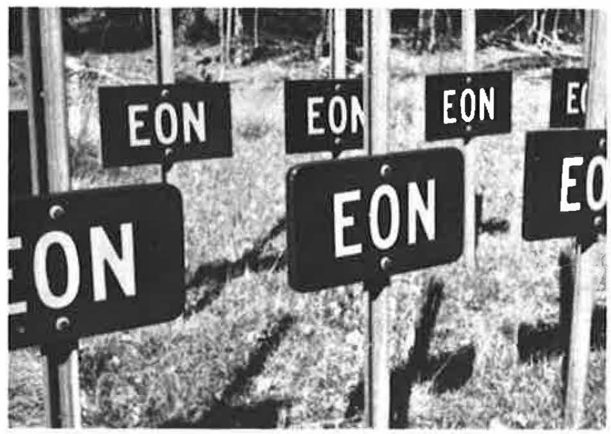
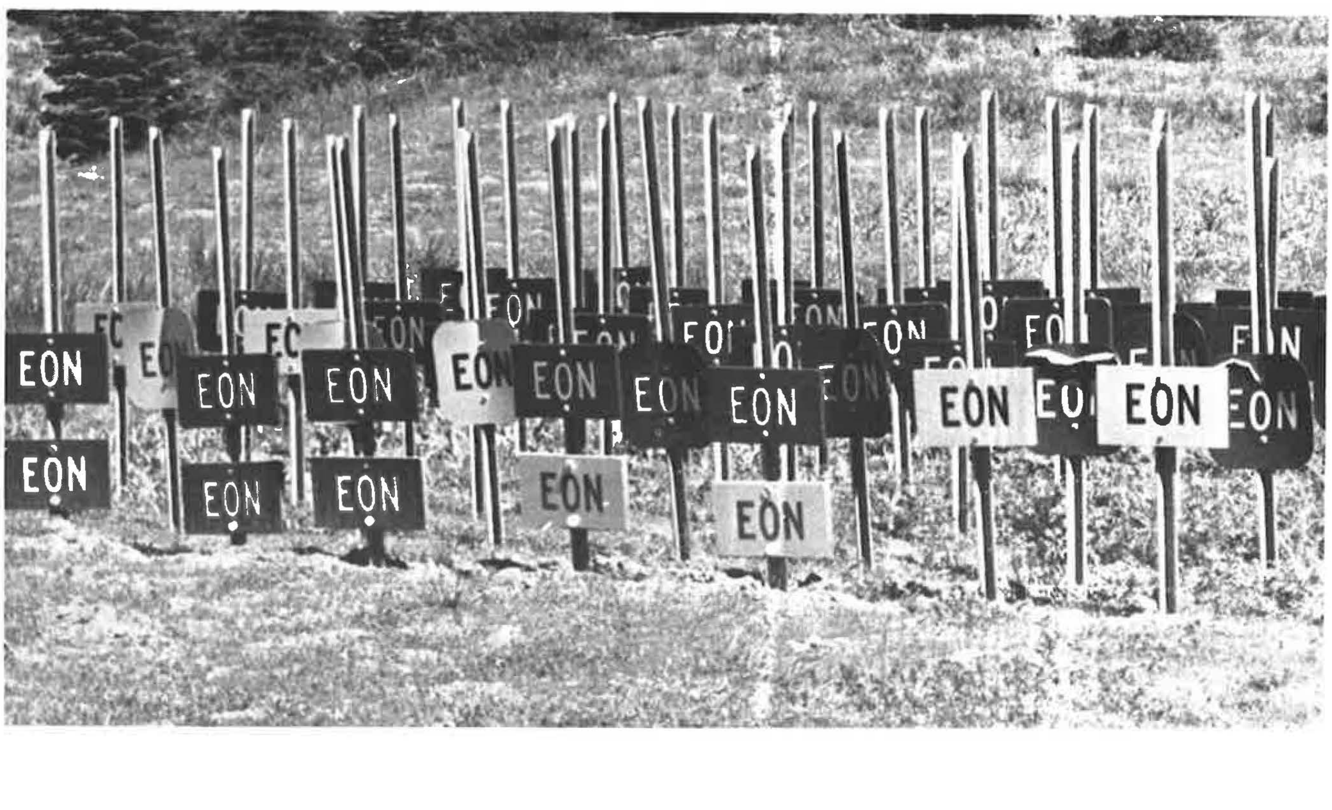


Figure 10. Signs of Mitsubishi/Seibu International materials after 2 years of outdoor exposure.



A PRELIMINARY EVALUATION OF PAVED AND UNPAVED ROAD PERFORMANCE IN BRAZIL

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The study of unpaved and paved road performance was a principal part of the "Research of the Interrelationships of Road Construction, Maintenance and User Costs" conducted in Brazil during the period 1975 to 1979. The paper outlines the experimental design methodology and measurement techniques for the pavement and maintenance studies. Preliminary results of the performance of unpaved and paved roads, monitored on 30 unpaved sections and 65 paved sections in Brazil, are discussed. Equations predicting roughness, rut depth and gravel loss, are presented for unpaved roads. These performance parameters are a function of average daily traffic, horizontal alignment, vertical geometry, wearing course material type, maintenance and wet or dry season. Preliminary findings of analyses of roughness and rut depth on paved roads are also discussed.

The complexity of the road network/traffic load dynamic system has caused engineers and planners to use piecewise solutions to this overall systems problem. Part of the problem was the lack of information on the costs of the components of highway transportation, i.e., the costs of highway construction and highway maintenance and user costs. This is a problem which has confronted governments of developed and developing countries alike, as well as major financing agencies such as the World Bank. The current research project in Brazil was planned to respond to these needs and is sponsored by the Brazilian Government with aid from the United Nations Development Program.

The minimization of total transportation costs may be achieved by the type of model shown in Figure 1 (1). In the overall model pavement performance plays an integral and important role because it influences all the cost components of the highway model.

In the past many paved road performance relationships were developed in the United States (2), Canada (3) and Europe. These relationships are applicable to countries with well developed transportation systems, with temperature climates and pavement materials which are derived from alluvial or glacial deposits. Considerable uncertainty exists in translating these relationships directly to developing countries, particularly those in the tropics, where pavements are constructed with materials which have been influenced by tropical weathering. With the exception of the Kenya Study (4), relatively little recent research has been conducted on the performance of unpaved roads, which constitute the major proportion of the road network in developing countries. Thus a major aspect of the study in Brazil was to study unpaved, as well as paved, road performance and behavior. Elements of pavement performance and behavior which are addressed in this preliminary evaluation of the results are roughness, rut depth, gravel loss and loose material on unpaved roads and roughness, rut depth, and cracking and patching on paved roads.

Design of Experiments

The experimental design matrix for the study of unpaved roads includes four factors. These were average daily traffic, vertical alignment and horizontal curvature at two levels, and surface type at three levels, as is shown in Figure 2. The surface type materials studied were laterite, quartzite and sections without a surfacing, whose material was defined as containing more than 35 percent material passing the 0.074mm sieve. Besides the 29 sections in the main factorial, a further 19 sections were studied at intermediate levels to permit curvature relationships to be developed in regression analyses.

All the sections were used to investigate their performance under minimal maintenance, i.e., the Resident Engineers were requested to withhold blading as long as was feasible. In addition to this evaluation, 10 sections in close proximity to Brasilia for which more than one year's data had been collected under the minimal maintenance conditions, were selected to study the influence of maintenance. Each section was divided into two subsections; one subsection was bladed every two weeks and the other subsection every six weeks. Thus, in effect three maintenance policies were investigated.

Based on previous studies (4), (5) six factors were selected for study on paved roads - surfacing type, base type, average daily traffic (ADT), state of rehabilitation, age of the original surfacing or overlay with asphaltic concrete and vertical alignment. The paved road experimental design matrix, contains the six factors at two levels. Figure 3 shows the matrix and the section numbers in the

Figure 1. Flow chart for highway cost model (1)

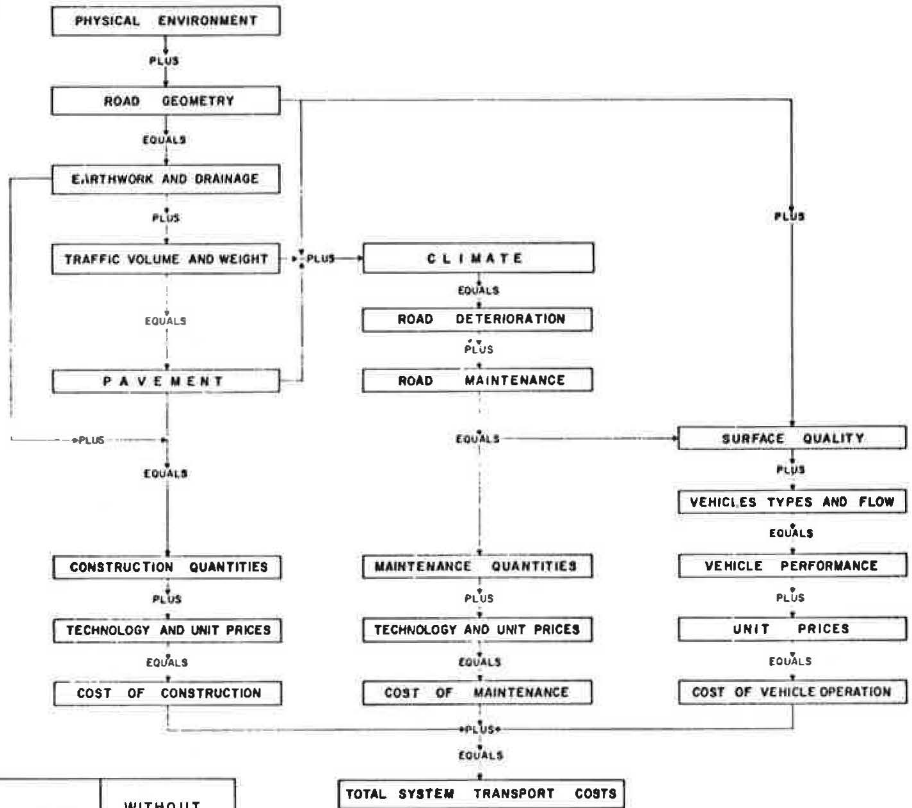


Figure 2. Unpaved road design matrix

Surfacing material / Traffic / Vertical geometry / Horiz. geom.		Laterite		Quartzite		Without surfacing	
		< 100	> 350	< 100	> 350	< 100	> 350
CURVE R < 250 m	≥ 6 %	203 209	312	213 215	315	217	
	0 - 1½ %	204	252 300	263	264	218	
TANGENT	≥ 6 %	206 262	253	307 261	255	205 216	
	0 - 1½ %	202	251 302	259	254 304	201	313

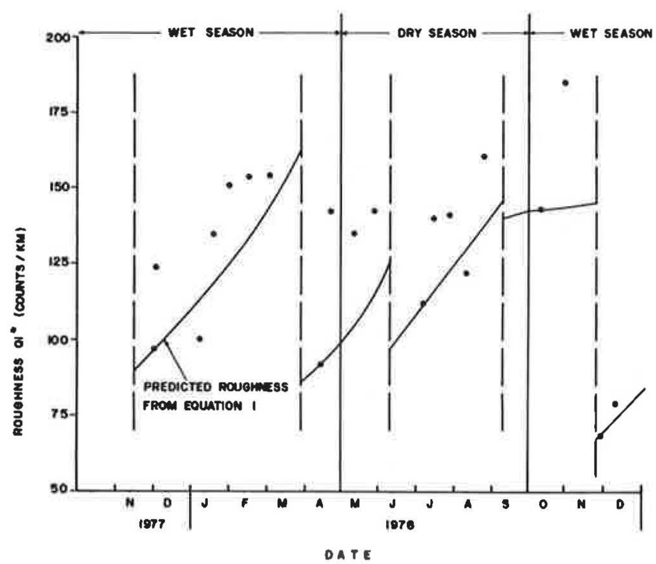
THE NUMBERS IN EACH CELL ARE THE SECTION NUMBERS.

Figure 3. Paved road design matrix

Surfacing type / Base type / Traffic (ADT.) / V. geom. (%) / Age (years) / State reh.			Asph. conc.				D. surf. treatm.			
			Gravel		C stone		Gravel		C stone	
			50-500	>1000	50-500	>1000	50-500	>1000	50-500	>1000
OVERLAYED	≥6	≥6	128	129						
		0-1.5%				158		109 009		
	0-2	≥6		125			035	032		
		0-1.5%		006			034	031		
AS CONSTRUCTED	≥12	≥6		119			123 172	110 008		
		0-1.5%		003 113		168	173	007		
	0-4	≥6	022	025	151	162	002	024 111	155	103 165
		0-1.5%	001 021	033 112	152	161 026	004 023	106	101	102

THE NUMBERS IN EACH CELL ARE THE SECTION NUMBERS.

Figure 4. Roughness data points and prediction of roughness over time for section 303



Details of section 303:
ADT = 320 vpd, Grade = 3.4%, rad of curv. = 180 m
quartzite

cells which were filled. In addition to the main factorial, which contains 44 sections, a further 22 sections were selected at intermediate levels of the factors to investigate curvature relationships in the regression.

High and "nil" levels of maintenance were studied on paved test sections. Each section was divided into two; on the one subsection only filling of potholes was permitted and it was called the "nil-maintenance" subsection. A high level of maintenance on the other subsection required cracks to be slurry sealed, bad base deformations excavated and replaced and potholes excavated to a regular format and filled with an asphaltic mix.

Measurement Techniques of Dependent Variables Studied

Roughness

Two instrumentation systems were used to measure roughness in Brazil - the Maysmeter and the G.M. Profilometer. The Maysmeter is a simple, low-cost instrument designed for installation into a passenger vehicle, capable of producing an acceptable measure of roughness while traversing a road section at a normal vehicle operating speed. The Profilometer is a sophisticated and expensive device that accurately measures the profile of the road surface over which it passes. The principal function of the Profilometer, was to provide a basis for calibrating the Maysmeters. The Profilometer contains a custom built computer which simulates the passage of a quarter of a car, consisting of a body mass, tire, shock absorber and spring system, and it summates the movements of the body relative to axle as it passes over the measured road profile. This output from the Quarter-Car-Simulator (QCS) is termed the quarter-car index (QI) and has units of counts/km. A very smooth, newly constructed asphaltic concrete would have a QI value of less than 30, whereas a very rough unpaved road would have a QI greater than 200.

Calibrating the Maysmeters is essential since numerical values of roughness generated by the Maysmeter are very sensitive to the vehicle, as well as to the condition of tires, shock absorbers and springs. A set of 20 paved road sections which cover a range of roughness, were used to calibrate the Maysmeter according to the methodology developed in Texas (6). This ensured that results obtained with any of the Maysmeters in use on the Project, at any time, were compatible. This compatible roughness output, through a correlation with QI, is designated by QI* in counts/km.

Roughness results with the Maysmeter were collected by traversing each pavement study section three times, in each direction. On paved roads the test speed was standardized at 80 km/h, whereas on unpaved roads the speed had to be selected consistent with the road conditions. The highest of three possible speeds, 80 or 50 or 20 km/h, that would not cause damage to the equipment, was selected. Since the Maysmeter-QI relationship is sensitive to speed, the results obtained at alternative speeds were standardized to 80 km/h in accordance to relationships presented in (7). Measurement frequency was about four to six months on paved roads, two to three weeks on unpaved roads used in the minimal maintenance program, and every two to three days on the high frequency maintenance sections.

Rut Depth

An AASHO type rut depth gauge with a base length of 1.22 m (4 ft), which is only supported at the two extremes, was used on paved roads. On unpaved roads, because of accentuated influence of localized depressions on the readings, a cross-bar was fitted to act as a base to the A-frame. The apparatus is graduated to read rut depth with an accuracy of 1 mm. On paved roads rut depths were measured at four to six month intervals. On the unpaved roads, in the minimal maintenance program, rut depth was measured at two to three week intervals, and every two to three days on the high-frequency maintenance sections.

Gravel Loss

Regravelling is the most expensive single maintenance operation on unpaved roads, and it may be compared to overlaying a paved road in importance. Prediction of gravel loss was thus an important objective of this study. The methodology adopted was similar to that developed in Kenya (4). A grid of points 1 m apart across the road's width, and 5 m apart along the length of the road, was levelled at three monthly intervals relative to fixed benchmarks. Consequently the change in height of the grid points relative to the benchmarks was calculated over time, which, when averaged, represents a change in level of the road surface, which is the thickness of gravel lost. Since the benchmarks were fixed in the subgrade, they accommodated settlement of the section.

Loose Material on Unpaved Roads

In the Kenya study (9) the presence of loose material on unpaved roads was found to have an influence on fuel consumption. Additionally, loose material could be a predictor of gravel loss. Loose material, in millimeters, was determined by dividing the volume of loose material collected by the area over which it is collected. A steel frame, 1 m by 0,25 m, was used to delineate the area. Looseness measurements were taken across the road per 1 m width at two transverse sections within each subsection. A wire brush was used to sweep the material together, which was then poured into a measuring cylinder. Moisture contents of the loose material at each transverse section was determined in the laboratory. These measurements were taken about every two to four weeks.

Cracking and Patching

A technique (8) was developed to survey the complete section and to plot the defects and cracks on a map of the section. In this way progression of the deterioration can be followed. An AASHO type classification of the cracks was adopted as shown in Table 1, but the classification was increased to include class four cracks which represent cracks in an advanced state of deterioration. Condition surveys, which consisted of delineating and mapping each area of each crack class were carried out at about four to six month intervals. The results of the condition survey were also used to trigger maintenance.

Table 1. Definition of classes of cracks

Class of crack	Description
Class 1	Very fine cracks with a width of less than 1 mm
Class 2	Cracks with a width of 1 mm to 3 mm
Class 3	Cracks with a width greater than 3 mm
Class 4	Any width of crack which exhibits ravelling (deterioration) of the edges

Measurement Techniques of Covariates Studied

Besides the dependent variables, which are those variables for which prediction expressions are desired, and the factors of the design matrix, which are independent variables with readily determinable values, there are covariates, which are independent variables whose values are not readily controlled, and which could have an influence on the dependent variables. The covariate data collected on unpaved roads included material characteristics, number of days since last blading, blading period number since start of observations and season. In the study area there are two distinct seasons, the dry season and the wet or rainy season. Generally the dry season is from beginning April to the end of September when hardly any rain falls, and the wet season extends from beginning October to the end of April when almost all of the annual rainfall of about 1600 mm falls. On paved roads the covariates were cumulative equivalent axles, pavement structural number and pavement deflection.

Cumulative Equivalent Axles

Traffic counts as well as axle weights were required to develop cumulative equivalent axles. Historical traffic classification counts were obtained at each section. Where unavailable, this data was collected during the Project's duration. Exponential curves, using least square techniques, were fitted to the historical vehicle classification counts to facilitate later computations.

Axle weights were measured with a weigh-in-motion (WIM) system and portable scales. The AASHO axle equivalency factors for a structural number of 2.0, were used for single and tandem axles. Results presented in (10) were used for the equivalency factors of triple axles, i.e., three axles in a group. Thus on each section the average number of equivalent axles per vehicle for each vehicle class was calculated in both directions. In the calculation of the average number of equivalent axles per vehicle, it was assumed that axle weights for each vehicle class on the section had not changed since the road was constructed. Verification of replicate results collected two years apart showed that the axle weight distribution was constant, except when mining or heavy industries were newly located in the region served by the road. Combining the traffic flow relationship for a specific vehicles class with the average number of equivalent axles per vehicle permitted the calculation of cumulative equivalent axles to correspond to the time when a roughness measurement or condition survey was carried out.

Pavement Structural Number

At each section three test pits were opened. Layer thicknesses, in-situ CBR and in-situ density were measured and samples taken for the standard laboratory tests on soil samples. The resilient modulii were measured by means of the indirect tensile test on the asphaltic concrete samples (11). Structural coefficients of each layer were related to strength measurements through relationships presented in the literature (4 and 12) and which were adapted to local conditions. The structural coefficients (a_i) per inch used are as follows:

$a_1 = 0.10$ for surface treatment

$a_1 = 0.18$ for asphaltic concretes

$a_1 = 0.46(1-e^{-0,000084M_{R30}})$ for asphaltic concretes with a thickness greater than 3 cm

Where M_{R30} = the resilient modulus at 30°C in kgf/cm^2

$a_2 = (29.14CBR-0.1977CBR^2+0.00045CBR^3)x10^{-4}$ for base courses

$a_3 = 0.01+0.065 \log_{10}CBR$ for sub-base layers or selected subgrade with an in-situ CBR greater than 40.

The structural number SN was calculated by summing the products of structural coefficient (a_i) and layer thickness (t_i) e.g.

$$SN = \Sigma a_i t_i$$

The structural number was corrected to allow for the structural support of the subgrade as follows:

$$SN^1 = SN+3.51 \log_{10}CBR-0.85(\log_{10}CBR)^2-1.43$$

Deflection

Deflections, which could be used as a surrogate of strength, or as a measure of variability of construction of the section, were obtained with the Benkelman Beam and with a Dynaflect. The measurements were usually made at 10 points in each wheelpath in each direction on each subsection. The Benkelman Beam rebound deflection was measured under a standard 40 kN dual wheel load using 2:1 ratio beams. Measurements with both instruments were obtained every six to nine months.

Presentation and Discussion of Results

Table 2 shows the mean, standard deviation and range for the independent and dependent variables used in the unpaved and paved road analysis. The statistics for gravel loss on unpaved roads are not shown, since these are a rate, and cracking ranges from uncracked to the complete area cracked. Since large changes in the cracked condition occur summary statistics have little meaning.

Analysis of Unpaved Road Results

Roughness on Unpaved Roads. Roughness data from 30 unpaved sections which had lateritic and quartzitic gravel wearing courses were analyzed. Two regression equations were developed. The first predicts roughness as a function of time within a blading period given the roughness after blading. The second predicts the roughness after blading.

The following equation was developed for predicting roughness (QI*) given the roughness after blading.

$$\log_e(QI^*) = \log_e F + D(0.00461 + 0.00477T + 0.00094G + 0.0000052ADT + 0.9832/R - 0.005777S - 0.0000055T.ADT + 0.003792T.S - 0.0000424T.F - 0.1871G/R - 0.00000535G.F - 0.0081F/R) \quad (1)$$

Where

D = number of days since last blading
F = roughness after blading (the first observed roughness after blading was used to develop equation 1)
T = type of gravel wearing course:
laterite: T = 0
quartzite: T = 1
G = absolute value of grade, in percent
ADT = average daily traffic in both directions
R = radius of curve, in m
S = season, dry season S = 0; wet season S = 1

The mean square error of the model is 0.031.

The mean roughness measurements per observation date of section 303 over a one year period are shown in Figure 4 together with the roughness prediction obtained from equation 1. In each case the first observed roughness after blading was used as input for F.

Roughness measurements were seldom taken immediately before or after blading. Therefore, equation 1 was used to predict the roughness immediately before and after blading from the first observation after blading and the last observation before blading respectively. The predicted values were used to develop the following model for roughness after blading.

$$F = 31.0 + 18.7T - 1.84G + 0.0392ADT + 14.3S + 554.7G/R + 2330.6S/R + 0.2726L \qquad (2)$$

Where

L = roughness before blading

The mean residual and the mean square error of equation 2 vary for different levels of L. For L less than or equal to 140 the mean residual, $\mu\varepsilon$, equals -11.7 and the mean square error, σ_ε^2, equals 601.3. For L greater than 140, $\mu\varepsilon$ equals 9.9 and σ_ε^2 equals 1971.6.

A series of roughness curves over time for four typical combinations of significant factors are shown in Figure 5. The roughness after blading at the start of the exercise was assumed to be 80. Equation 1 is then used to generate the roughness values until the first blading. The value for F used in the next blading period was then calculated in the following way:

$$F = F' + (\mu_\varepsilon \pm Z\sigma_\varepsilon) \qquad (3)$$

Where

F' = the first roughness predicted from equation 2

μ_ε = mean residual from equation 2 $= \begin{cases} -11.7 & \text{if } L \le 140 \\ 9.9 & \text{otherwise} \end{cases}$

Z = normal (0,1) value

σ_ε = standard error of equation 2 $= \begin{cases} 24.5 & \text{if } L \le 140 \\ 44.4 & \text{otherwise} \end{cases}$

This procedure was then repeated for each blading period. A distribution was used for the calculation of F since its value enters into equation 1 in a non-linear manner.

Equation 1 predicts that the increase in roughness with time within a blading period during the wet season is less than during the dry season. Under certain combinations of the significant factors during the wet season the road may in fact become smoother with time. This is caused by drivers avoiding puddles and thus following a path through the section which meanders, but avoids depressions. Roughness measurements were taken in the most prominent wheelpaths where the roughness was lower than the route through the depressions. Road width and traffic volume could influence this relationship, but even on the most heavily trafficked sections the road was sufficiently wide to permit meandering.

Rut Depth on Unpaved Roads. For the development of the models for the prediction of rut depth on unpaved road sections 30 test sections with laterite or quartzite wearing course gravel were used.

The data were analyzed in two stages. In the first stage the time effects on the change in rut depth, in millimeters, were considered and the following equation was developed:

$$\Delta\log_e(\text{rut depth}) = D\,(0.00481 + 0.00001ADT - 0.6663/R - 0.02496S - 0.00001ADT.T + 0.002749T.L + 0.01289S.T - 4.9024S/R + 0.004371S.G) \qquad (4)$$

Where

D = number of days since last blading
ADT= average daily traffic in both directions
R = radius of curve, in m
G = absolute value of grade, in percent
S = season; dry season: S=0; wet season: S=1
T = type of gravel wearing course; laterite: T=0; quartzite: T=1
L = lane; downhill lane: L=0; uphill lane: L=1

The mean square error of the equation is 0.125.

Equation 4 was then used to calculate the rut depth values at time zero for each observation, in the following way:

$$\log_e(\text{rut depth for } D=0) = \log_e(\text{rut depth at } D) - \Delta\log_e(\text{rut depth at } D)$$

The following equation was calculated for the mean rut depth, in millimeters, for D=0:

$$\text{Mean}\log_e(\text{rut depth for } D=0) = 1.447 + 0.726T + 0.00149ADT + 114.3/R - 0.1198WP + 1.021S - 106.3T/R - 0.5920T.S - 0.2093ADT/R + 0.00081ADT.S - 0.0893S.G \qquad (5)$$

Where

WP = wheelpath; external wheelpath: WP=0; internal wheelpath: WP=1

The mean square error of the equation is 0.244.

Equations 4 and 5 must be used together to predict the rut depth at any time since last blading. Figure 6 shows the data points and the rut depth calculated from equations 4 and 5 for section 303, which was selected because it had typical ranges of days since blading in both seasons.

Table 3 shows the results which were generated from equations 4 and 5 for the entire range of the significant factors. The rut depth of the internal wheelpath is not shown, but it is 8.8 percent less than that of the external wheelpath. The results for the wet season, presented in Table 3, are of a similar order of magnitude as the results obtained in Kenya (4), whereas the results in the dry season are lower than found in Kenya.

In the dry season ruts develop very slowly, and from the magnitude of the rut depth shown in Table 3, rut depth will probably not act as a trigger for maintenance. In the wet season substantial ruts develop, and thus could trigger maintenance activities. Under certain conditions the rut depth in the wet season decreases. This is probably due to the fact that drivers try to avoid water ponds and then tend to move their vehicles to drier ground. Consequently the wheeltrack position changes over time. Since rut depths were measured in the most prominent wheeltracks diminishing rut depths over time were recorded.

Gravel Loss. The prediction model for gravel

Table 2. Summary statistics related to variables studied

UNPAVED ROADS

Independent Variables	Mean	Std. Deviation	Maximum	Minimum
Average daily traffic (vpd)	236	167	608	18
Vertical alignment (%)	2.7	2.7	8.2	0.0
Horizontal curvature			Tangent	R=180m
Number of days between bladings	95	55	342	7
Cumulative traffic between bladings	24010	22120	143600	720
Dependent Variables				
Roughness (QI* counts/km)	130	63	554	35
Rut depth (mm)	18	11	55	2

PAVED ROADS

Independent Variables	Mean	Std. Deviation	Maximum	Minimum
Age at Jan. 1979				
Sections as constructed	7.9	4.4	20.5	2.5
Overlaid	5.3	4.1	12.5	0.5
Vertical alignment (%)	3.3	2.5	7.6	0
Cumulative Equiv. 80 kN axles (Jan. 1979)	1.36×10^6	3.27×10^6	20.4×10^6	0.003×10^6
Corrected structural number	5.0	0.9	7.5	3.4
Benkelman beam deflection (mm)	0.70	0.30	2.10	0.28
Dependent Variables				
Roughness (QI* counts/km)	37	15	99	15
Rut depth (mm)	3	1.6	11	0

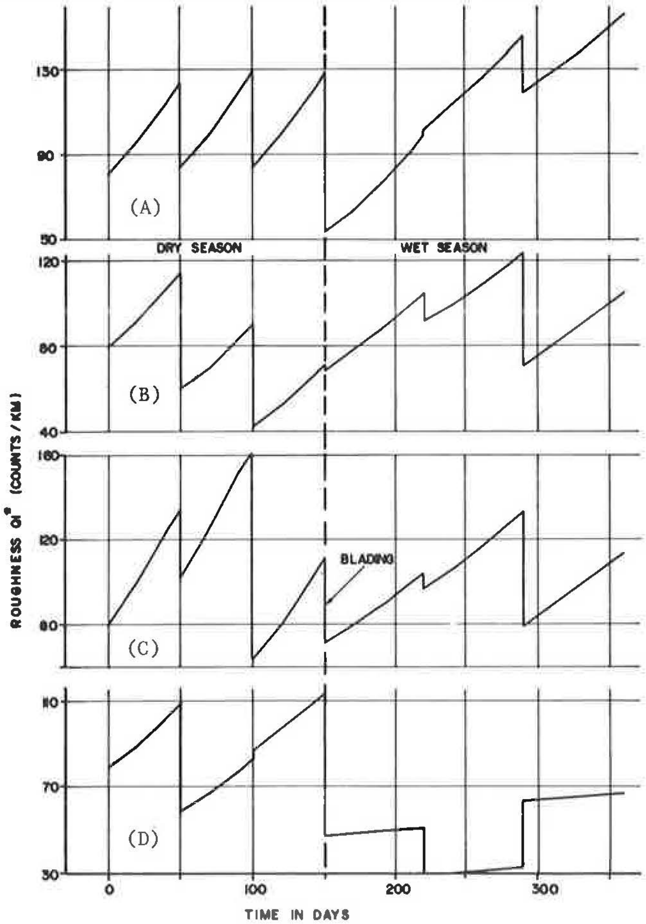
Table 3. Generated values of rut depth (in mm) in the external wheelpath on unpaved roads

Wearing course				LATERITE						QUARTZITE					
Average daily traffic (vpd)				20			600			20			600		
Season	Vertical align	Lane	Horizontal curvature \ Cumulative traffic	0	2000	4000	0	30000	60000	0	2000	4000	0	30000	60000
DRY SEASON	6%	DOWN HILL	Tangent	4	7	12	10	18	31	9	15	24	22	26	32
			250 m	7	9	11	10	15	23	9	11	14	13	14	16
		UP HILL	Tangent	4	7	12	10	18	31	9	19	41	22	30	43
			250 m	7	9	11	10	15	23	9	15	24	13	17	20
	1%	DOWN HILL	Tangent	4	7	12	10	18	31	9	15	24	22	26	32
			250 m	7	9	11	10	15	23	9	11	14	13	14	16
		UP HILL	Tangent	4	7	12	10	18	31	9	19	41	22	30	43
			250 m	7	9	11	10	15	23	9	15	24	13	17	20
WET SEASON	6%	DOWN HILL	Tangent	7	14		28	51		8	55		32	79	
			250 m	11	2		26	16		8	6		20	16	
		UP HILL	Tangent	7	14		28	51		8	72		32	90	
			250 m	11	2		26	16		8			20	19	
	1%	DOWN HILL	Tangent	11	2		43	27		13	10		49	41	
			250 m	18	0		41	8		13	1		31	8	
		UP HILL	Tangent	11	2		43	27		13	13		49	47	
			250 m	18	0		41	8		13	1		31	10	

Table 4. Generated values of change in gravel level in millimeters

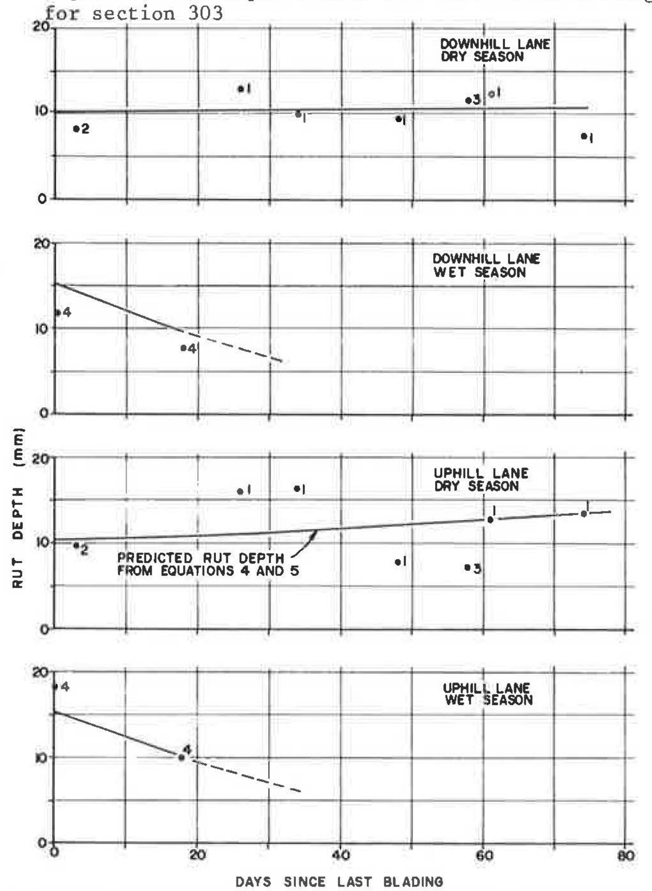
Average daily traffic (vpd)			20				600			
Wearing course	Vertical alignment	Horizontal curvature \ Number of bladings	No blading 250 days	2 bladings	6 bladings	12 bladings	No blading 250 days	2 bladings	6 bladings	12 bladings
LATERITE	6%	Tangent	-0.4	-6.0	-17.9	-35.7	-0.4	-16.1	-48.4	-96.8
		250 m Curve	-0.4	-7.7	-23.0	-46.0	-0.4	-17.8	-53.5	-107.0
	1%	Tangent	-0.4	-1.3	-3.9	-7.7	-0.4	-11.5	-34.4	-68.7
		250 m Curve	-0.4	-3.0	-9.0	-18.0	-0.4	-13.2	-39.5	-79.0
QUARTZITE	6%	Tangent	-0.4	-6.0	-17.9	-35.7	-0.4	-16.1	-48.4	-96.8
		250 m Curve	-0.4	-14.2	-42.6	-85.2	-0.4	-24.4	-73.1	-146.2
	1%	Tangent	-0.4	-1.3	-3.9	-7.7	-0.4	-11.5	-34.4	-68.7
		250 m Curve	-0.4	-9.5	-28.6	-57.2	-0.4	-19.7	-59.1	-118.2

Figure 5. Roughness curves over time generated from the roughness prediction model for unpaved roads



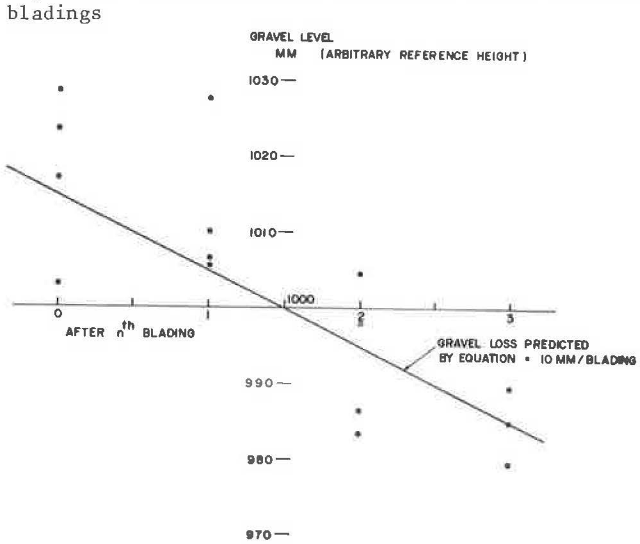
Note: blading every 50 days in dry season and every 70 days in wet season.
(A) quartzite, grade = 6%, ADT = 600 vpd, tangent
(B) quartzite, grade = 1%, ADT = 20 vpd, curve = 250 m
(C) laterite, grade = 6%, ADT = 600 vpd, tangent
(D) laterite, grade = 1%, ADT = 20 vpd, curve = 250 m

Figure 6. Rut depth versus time since last blading for section 303



Note: number next to data point refers to blading period number.
Details of section 303: ADT = 320 vpd, grade = 3.4%, rad of curv. 180 m, quartzite.

Figure 7. Gravel level as a function of the number of bladings



Legend: each data point is the mean gravel level of row 2 per subsection-lane.

Figure 8. Roughness versus age for paved test sections with SN' in the range of 4 to 5

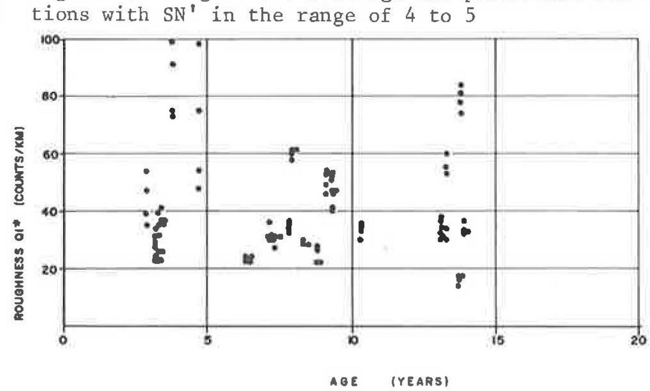
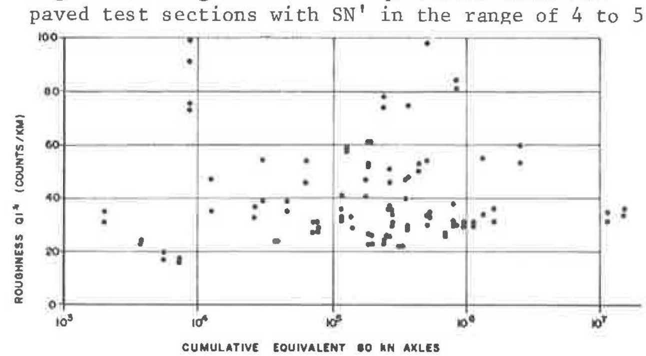


Figure 9. Roughness versus equivalent axles for paved test sections with SN' in the range of 4 to 5



loss was developed from data collected on 30 sections with laterite or quartzite wearing course gravel. The periods between bladings on these sections ranged from 14 to 250 days over an 18 month observation period. The equation can be applied to sections that receive maintenance in the form of blading, or that are left without blading.

The following is the prediction model for the change in gravel height, in millimeters:

$$\text{Change in gravel height} = B\,(-0.0046ADT - 213.8/R - 0.4670G - 816.6T/R - 0.0043ADT.R_2 - 0.0082ADT.R_3) + D\,(0.0580 - 0.0461R_2 - 0.1322R_3) \quad (6)$$

Where

B = number of bladings
ADT= average daily traffic in both directions
R = radius of curve in m
G = absolute value of grade in percent
T = wearing course material type, laterite: T=0 quartzite: T=1
D = time in days since last blading or start of observation period
$D = 0$ if the section has been bladed, i.e., if $B > 0$
$D > 0$ if the section has not been bladed, i.e., if $B = 0$
R_2= transverse location variable
R_2= 1 if location is 2 m from the road edge
R_2= 0 otherwise
R_3= transverse location variable
R_3= 1 if location is 3 m from the road edge
R_3= 0 otherwise

The mean square error of the model is 361.6.

Measurements taken in the external three rows of each subsection-lane, starting 1 m from the road edge, were used to analyze the influence of transverse location on gravel loss. $R_2 = R_3 = 0$ corresponds to the most external row, row one, located 1 m from the road edge. Row two is 2 m and row three is 3 m from the road edge.

Equation 6 can be used for the prediction of gravel loss for two different situations. If the road is bladed then B is set to the number of bladings and D is set to zero. The predicted change in gravel height represents the total gravel loss caused by blading, weathering and traffic. Although the model is a function of the number of bladings, it can be used in terms of a unit time by considering the number of bladings per unit time. If the road is not bladed then B is set to zero and the model becomes a function of time in days. Therefore, the first part of the equation should be applied in periods when the section is bladed and the second part used before blading has started or after it has terminated.

Equation 6 demonstrates that without blading there is a movement of gravel from the middle of the road towards the external row with time.

A plot showing the gravel loss model for row two for the combination of factors exhibited by section 257, together with the data points of section 257, is shown in Figure 7. Since gravel loss is a rate, the model was centred through the mean number of bladings and the mean arbitrary gravel level. This section received three bladings during the observation period, and the observations were taken between bladings.

The primary objective of predicting gravel loss is to program regravelling. The mean change in gravel height over a section was derived from equation 6 by combining the three rows.

$$\text{Mean change in gravel height} = B\,(0.0088ADT - 213.8/R - 0.4670G - 816.6T/R) - 0.00143D \quad (7)$$

Equation 7 predicts the mean change in gravel height after B bladings, or if no bladings are applied, after D days. This equation was used to generate changes in gravel height for 2,6 and 12 bladings, and over a 250 day period when no maintenance was applied, as shown in Table 4 for the ranges of significant factors.

The mean gravel loss is dependent on the number of bladings rather than time or cumulative traffic as is generally reported in the literature. Consequently a direct comparison to published results is not possible. The results in Table 4, up to six bladings, are of a similar order of magnitude as found in Kenya (4), assuming that the bladings occurred within a one year period. Lund (13) supports the finding that maintenance is correlated with gravel loss. His observations were that the quality of maintenance probably plays an important role on gravel loss. This was an aspect which was not studied in the present Project, and the relationships presented are deemed to consider average maintenance standards as were generally applied in the study region.

Loose Material. A preliminary analysis of the data collected shows that the thickness of loose material within 2 m from each road edge is considerably larger than the thickness over the rest of the road width. The results obtained at different cross-sections locations within a section are not significantly different, and consequently the test procedure was standardized to take measurements at two cross-sections per subsection. The time effect on the thickness of loose material was found to be strong, and could be caused by the influence of moisture content of the loose material. Further work is continuing to develop predictive equations for loose material.

Analysis of Paved Road Results

The design of experiments for the paved road analysis was structured to permit two types of analysis, viz of the time effect and of traffic, geometric and pavement characteristics. In the analysis of time effects the change in the dependent variable, e.g. roughness, rut depth or cracking and patching, is studied for each section during the observation period. An analysis of traffic, geometric and pavement characteristics permits an evaluation of these factors shown in Figure 3, together with directional and maintenance effects, on the dependent variables.

It is important to note that the paved road analysis is still in the preliminary stages, but that there are interesting initial observations and also some problems.

Roughness on Paved Roads. Preliminary analyses of the time effects showed that:

1. The steeper the grade, the greater the increase in roughness with time. The change in roughness over time was not significantly different for the uphill and downhill directions on any grade.
2. The increase in roughness over time on a section constructed with a crushed stone base is

greater than for a section constructed with a gravel base. In some cases the material classified as crushed stone was of a poorer quality than would be expected with crushed stone, and this could influence the finding.

3. Changes in roughness with time on the two maintenance subsections were not significantly different.

In the analysis of the main factors the mean roughness over time on the two subsections of each section were not significantly different. Certain problems have become apparent through the cross section analysis. For a corrected structural number of 4 to 5 the range of roughness is almost independent of age, as may be seen from Figure 8. In some cases an old road has the same roughness as a new road. This situation remains similar when age is substituted by equivalent axles as is shown in Figure 9 because of the high correlation between age and cumulative equivalent axles.

Sections which have carried large numbers of equivalent axles are often smoother than sections which have carried a low number of equivalent axle repetitions, because thicker pavements were designed for heavily trafficked roads. This situation presents problems in separating the structural number and cumulative equivalent axles effects.

Analysis techniques which are expected to overcome these problems associated with cross section analysis are presently being applied to develop performance models.

Rut Depth on Paved Roads. Initial analyses of the time effects indicated that:

1. The increase in rut depth with time on old pavements is faster than on newly constructed or newly overlaid sections.
2. On sections less than four years old, the increase in rut depth is greater for sections with a gravel base than for sections with a crushed stone base.
3. On sections less than four years old the increase in rut depth is greater for sections with an asphaltic concrete surfacing than a surface treatment.

In the analysis of the main factors similar confounding effects were found as elaborated above for roughness.

Cracking and Patching. Preliminary evaluation of the percentage area of the test section which exhibits class 1 to 4 cracking shows that the total area which is cracked is considerably larger for as-constructed roads with an asphaltic concrete surfacing than for sections with a surface treatment or sections which were overlaid. Further analysis of this dependent variable is continuing.

Conclusions

After a two year study period preliminary results of the performance of unpaved and paved study sections on in-service roads in the central and southeast regions of Brazil were presented.

The development of roughness on unpaved study sections over time within a blading period was related to the average daily traffic, vertical geometry, horizontal alignment, wet or dry season, lateritic or quartzitic wearing course gravel and the roughness after blading (Equation 1). A further model (Equation 2) was developed which predicts the roughness after blading as a function of roughness before blading, vertical geometry, horizontal alignment and wet or dry season. Generation of road roughness over time can then be a accomplished by using equations 1 and 2 with a normal distribution of the residuals of equation 2.

The prediction equation for rut depth on unpaved roads also consists of two models; one predicts the development of rut depth over time and the other predicts the rut depth after blading. In addition to the significant factors found for roughness, lane and wheelpath effects were found to be significant.

The change in gravel height was found to depend on the number of bladings, average daily traffic, horizontal alignment and material type. The presented model for roughness shows that it is necessary to blade the road to maintain a desired roughness level, and the gravel loss model indicates that by blading, gravel is lost. This finding requires that optimization techniques be used to program maintenance.

The paved road analysis was still very preliminary and some preliminary observations, as well as problems related to analysis of the main factors were presented. Several analysis techniques are being evaluated to overcome the problems related to cross section analysis.

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THE EFFECT OF ROAD DESIGN AND MAINTENANCE ON VEHICLE OPERATING COSTS-FIELD RESEARCH IN BRAZIL

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The design and methodology of the survey of vehicle operating costs on rural and inter-urban roads in a large area of central, western and southern Brazil is described. Data were collected from 117 operators for 1326 vehicles by a team of 20 researchers. Road roughness and geometric characteristics were measured on 423 routes nearly 40000 km in length using two specially instrumented cars. The method of analysis is explained and some preliminary relationships are presented and discussed. Special attention is being given to depreciation and vehicle maintenance costs. An example is used to show that on typical unpaved roads in Brazil increases in fuel consumption are small in relation to the overall increase in costs, when compared to paved roads. Analysis is in its early stages and will continue until November 1979.

Since October 1975 research has been underway in Brazil to determine the relationships between the cost of highway construction, maintenance and utilization, primarily for low volume roads. The project is sponsored by the government of Brazil and the United Nations Development Program, and is managed by the Ministry of Transport through its affiliate GEIPOT (Brazilian Transport Planning Corporation). The project is based at the headquarters of GEIPOT in Brasilia the Federal Capital, and will continue until November 1979.

This paper describes the vehicle operating cost survey and gives the results of some preliminary analyses. It is emphasized that the results are from early analyses of selected data. Field data collection has not yet been completed and final analysis of all data has not begun. The results should be considered with this in mind.

Research Objectives

The objective is to develop and improve relationships between construction, maintenance, and road user costs and combine them in a predictive model which will assist in minimizing the total cost of Brazil's highway network. The model calculates the construction cost of a road and predicts its condition over time for given traffic levels. Having predicted road condition the model estimates costs of road maintenance and vehicle operation.

Survey Design and Scope

In planning the survey it was realized that successful analysis would depend on how effectively the design and data collection could cover the extremes of operating conditions in Brazil, in terms of surface roughness and vertical and horizontal geometry. The need was for a sample from all major vehicle types, stratified by the important decision variables - road geometry and surface type. A very simple pre-stratification was used (Figure 1) to aid selection of operators of buses, trucks and automobiles, for two main reasons. First, it was not possible to pre-stratify road characteristics with any great degree of precision, and secondly, since very few vehicles operated on only one route it was necessary to consider vehicles using several routes of a broadly homogeneous character. The search for suitable vehicle operators was therefore concentrated in regions as widely different as possible with respect to topography, and in areas with the most unpaved roads.

Apart from buses, for which published data were available, little was known about the vehicle operators and their itineraries in the various regions, often remote from Brasilia.

It was generally fairly easy to locate operators with good cost records for vehicles on paved roads. However suitable operators on unpaved roads were much harder to find. There were fewer fleets in areas of largely unpaved roads, and fleet sizes were generally smaller. The need for homogeneous routes limited the areas available. Thus, areas of rolling terrain were avoided as far as possible while flatter or hillier areas were thoroughly searched.

It was necessary to search much wider and farther than had originally been planned, since many operators, although using roads suitable for the research, lacked adequate vehicle cost records. Almost one thousand operators, from large fleets to owner-drivers, were contacted in the initial searches and a mail-shot questionnaire was sent to a further 1500. Data quality was an important consideration. It was sometimes found necessary to be less restriccurve's effect on a steep grade. In order to quantify these kinds of effects algorithms were developed (5) defining a function for each grade and curve, weighted by the effect of the previous geometric condition. The weights are based on speed changes under various conditions derived from experimental data. Separate calculations are made for each grade and curve from the raw data file, summed and divided by the route total km to give the index (V and H) values.

Typical Roughness and Geometry Values

The mean value of the roughness index (QI*) for all the paved roads in the survey is 39 QI* with a standard deviation of 8, and for all unpaved roads 140 QI* with a standard deviation of 33. For geometry, a fairly flat and straight road would have a vertical index (V) of 4.5 and horizontal index (H) of 0.04, while a mountainous road would have V of 8.5 and H of 0.45.

Preliminary Analyses

Fuel, utilization, and parts data were obtained directly from the vehicles in the fleets of the companies. The mean costs per company/cell were analyzed since the variance from company to company within a cell was significantly greater than the variance from vehicle to vehicle within companies. Labour hours and tire data were collected directly at the company level. For these two items the mean value per company was analyzed as the dependent variable.

The dependent variable for all cost items is expressed in units per 1000 km. In the case of parts and depreciation costs where the units are Brazilian cruzeiros, (Cr$), the data values are transformed to constant cruzeiros of January 1976 when one U.S. dollar was approximately Cr$ 10,00. (6)

Weighted regression analysis was used. For fuel, utilization, and parts the reciprocal of the sample variance of the mean per company/cell was used as the weight. For labor hours data the fleet size was used as the weight, and for tyres, the number of tyres analyzed in each fleet.

For all cost items, the effects of vehicle characteristics were tested. The correlation between the highway characteristics and the vehicle characteristics was eliminated by adjusting the mean value of the latter per company/cell by its mean per cell. Thus the coefficient of each of the highway characteristics reflects the influence of that variable given the vehicle characteristics present in each cell of the factorial. The effect of the vehicle characteristics is calculated within the cells of the factorial and therefore can be interpreted independently of the highway characteristics. An interpretation of the equation for utilization is given after the discussion of depreciation.

Depreciation and Interest

The main objective was to develop a method of calculating vehicle depreciation with respect to differing highway characteristics. Two potential approaches were examined, one emphasing market valuations, and the other, vehicle utilization. The market valuation approach was finally rejected, because while it was possible to collect data for many actual sales, the vehicle histories, in particular routes operated, were unknown. Of vehicles included in the main survey, less than 10% were sold during the data collection period, an insufficient number for analysis.

Vehicle utilization (km per month) was analyzed using the hypothesis that if differential rates of utilization could be predicted, then differential depreciation costs could be calculated from average sale prices for vehicles of a given age. Two components were therefore used.

1. Utilization rates by vehicle class
   Prediction equations were developed where monthly or annual km is a function of highway characteristics and vehicle characteristics.
2. Average market values by vehicle class
   Curves were drawn representing the average decline in value for each year over the entire vehicle life. The data source was dealers specializing in used vehicles in the São Paulo area. Values were checked for consistency with actual sales of vehicles from the main survey. In addition, data on vehicle survival rates was obtained from the national vehicle registration authority. These data are not central to the methodology outlined above but are useful for the Brazilian model when calculating average lifetime depreciation and interest by vehicle class.

The components above were combined to predict depreciation cost, per km. Between given ages in a vehicle's life, depreciation per km is equal to the change in market value divided by utilization during that period, such that:

$$D_{c,j} = \frac{NP_c \; 1-DC_{c,j}}{\sum_{i=1}^{j} AV_{c,i}} \qquad (1)$$

Where:

j = 1,.....k
k = maximum vehicle age in years
$D_{c,j}$ = Depreciation cost per km at age j
NP_c = New vehicle price less tires for class c
$DC_{c,j}$ = Value of class c vehicle at age j, as a proportion of new price
$AV_{c,i}$ = Annual utilization in year i for class c vehicle

Interest. Interest should be included in vehicle operation cost calculations, normally as equal to the opportunity cost of capital. Having chosen the rate to be applied, the same components used above can be used to calculate interest cost, such that:

$$IC_{c,j} = \frac{NP_c R_c \sum_{i=0}^{j} DC_{c,i}}{\sum_{i=1}^{j} AV_i} \qquad (2)$$

Where:

$IC_{c,j}$ = Interest cost per km at age j for vehicle class c
R_c = Annual interest rate for vehicle class c

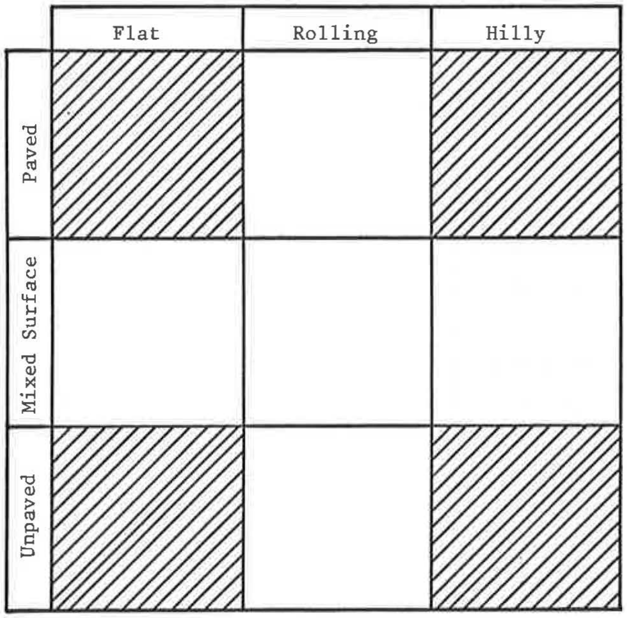
Preliminary Results

Utilization

The equation for utilization will be discussed to show how the influences of the vehicle characteristics can be interpreted for the other equations presented below.

The following generalized regression model spec-

Figure 1. Pre-stratification of vehicle operators.



Note. Searching was concentrated in the shaded cells.

tive on route characteristics in order to include a fleet operator with reliable and comprehensive cost records.

The geographic area of the survey now covers one million km^2, containing 30% and 40% of the total Brazilian paved and unpaved networks respectively. Data is collected regularly in 28 cities in the States of Minas Gerais, Goiás, Mato Grosso, Mato Grosso do Sul and Rio Grande do Sul, several locations being well over 1500 km from Brasilia.

The survey encompasses 653 buses, 442 trucks and 231 automobiles, from 62 fleets and 55 owner drivers, during an average data period of 16 months. The factorial dispersion of vehicles and routes is shown in Figure 2. All aspects of vehicle operating costs are being investigated, but special attention is given to depreciation, and spare parts and maintenance costs. Separate controlled experiments have been carried out in Brazil to investigate fuel consumption and vehicle speeds (1).

Methodology

Two separate but complementary tasks were involved-collection of vehicle cost data, and identification and measurement of the vehicle's routes. Coordination of these activities was the main rôle of survey management. The importance of the computer must be noted. More than 50 programs were specially written, and extensive use was made of a package for data retrieval and analysis.

Vehicle Cost Data

Information is collected on fuel, engine oil, workshop labor hours, spare parts and materials used, tyres, routes operated, and number of trips and kilometers on each route.

The basic approach is that frequent and regular visits are made to the vehicle operator's offices and workshop, so that the field researcher is as familiar as possible with the vehicle operations and company cost reporting procedures. Besides the benefits that such routine visits produce in terms of convenience to company managers, who give their time voluntarily, it is necessary to know something about the quality of data from such diverse sources. It is essential to know exactly how the data are assembled by the operator, and the use to which he puts the information.

The primary method of data collection is to make the maximum possible use of the vehicle operator's own records. When experience has been gained in interpretation of the material the method is considered efficient. At the outset, a detailed examination of all relevant company records is made by senior members of the research team in order to establish the reliability of the data. As noted above, it is then important that the field researcher makes regular visits to the company to become completely familiar with the costing system. Whenever possible, original records are photocopied for permanent reference. In the research office, data are compiled on a monthly basis and transferred to a keypunch form by the field researcher himself. In this way guidance on interpretation can be given, and closer supervision exercised by senior staff. Having been processed and edited the data are output to reports designed to facilitate screening and consistency tests. Errors are refered back to the field researcher for correction on numbered enquiry forms, while other less obvious inconsistencies are investigated by senior staff in direct field enquiries. If alterations to file data are necessary a separate computer report is generated so that new values can be checked.

Where the operator's own records are inadequate, as is frequently the case with owner drivers and some smaller fleets, the operator is given blank forms each month, and guidance on how to fill them out. The response rate was generally low, but dependent to a large extent on the effort given by the field researcher in setting up and maintaining such a system. In a number of cases contact with the research team has stimulated the operator to develop a more efficient cost control of his own.

Route Network

The routes extend throughout Goiás, Minas Gerais, Mato Grosso, Mato Grosso do Sul, and Rio Grande do Sul, with some sections into Espírito Santo, Rio de Janeiro, São Paulo and Santa Catarina.

So far, 423 routes have been measured totalling 66000 km, however since many routes have common links the actual road length is 39000 km, of which 18000 km is unpaved. Paved routes average about 220 km in length and unpaved 105 km. Traffic volumes average 2500 vehicles per day for paved routes and an estimated 200 v.p.d. for unpaved routes.

For each route the detailed itinerary was checked with the vehicle operator and transferred to a specially drawn map. Turning points on a route , or "nodes" were numbered and the location described in a computer file. For each route, the file then lists the complete sequence of nodes.

Roughness and Geometry Measurement. Two 2500 c.c. station wagons were used, crewed by a driver and an observer. Two passes of each link were normally made, measuring roughness in one direction and road geometry in the other.

For roughness, a Mays-Ride-Meter (2) was used, connected to a digital display giving counts every 322 meters (5 counts per mile) the count being noted by the observer on a keypunch form. Regular cali-

Figure 2. Vehicle and route final stratification.

Roughness	Horizontal	Vertical: Flat	Rolling	Hilly
Paved	1	80 vehicles 10 routes	349 vehicles 82 routes	53 vehicles 2 routes
	2	16 vehicles 13 routes	89 vehicles 67 routes	163 vehicles 37 routes
Mixed Surface	1	6 vehicles 7 routes	49 vehicles 21 routes	4 vehicles 5 routes
	2	4 vehicles 5 routes	31 vehicles 28 routes	9 vehicles 12 routes
Unpaved	1	138 vehicles 28 routes	147 vehicles 38 routes	58 vehicles 10 routes
	2	17 vehicles 5 routes	60 vehicles 27 routes	53 vehicles 26 routes

Note. Horizontal level 1 = low or average curvature,
level 2 = high curvature

bration was of paramount importance, because readings are sensitive to the condition of the vehicle's springs, shock absorbers, wheel alignment and tyres. The vehicles were calibrated before and after each field trip on 20 control sections against GMR Profilometer measurements. (3)

Horizontal curvature was measured with a standard aircraft type gyro compass. The start and end of each curve is called by the driver to the observer who notes the bearings from the gyro compass., and length of the curve in meters from the distance measuring instrument (DMI) (2).

Vertical profile was measured using an electronic linear accelerometer connected to a panel meter, scaled to plus or minus 12% grade. The instrument senses the gravitational acceleration resulting from tilting the axis of sensitivity off the level plane parallel to the ground. Drivers were trained to hold the vehicle speed constant while the reading is noted at approximately the mid-point of the grade. As with horizontal curvature, the driver calls the start and end of the grade and the observer notes the DMI readings. Transition grades are noted when the grade change is 1/2% or more and when the length of the grade is sufficient to record a stable reading from the panel meter. The system has been tested with known road profiles up to 15 km in length and although by no means exact, produces a satisfactory approximation to the known profile.

Roughness and Geometry Indices

Roughness. After each field trip the calibration coefficient obtained between the Maysmeter's measurements on the 20 control sections and those of the Profilometer is used to convert the raw data to a roughness index (QI*) scaled by the Profilometer's Quarter Car Simulator (3). A mean QI* value is then computed for each route. (4)

For unpaved roads in particular, roughness at any point on the road may vary considerably during the period of the survey. A replication measurement program is underway for selected unpaved and paved roads to determine the magnitude of this variation across the whole route. Preliminary analysis indicates that although individual points on the road vary due to bladings and the season, the overall effect on the mean roughness of the route is small and unlikely to affect the estimation of the roughness coefficients.

Geometry. It was hypothesized that the relationship between speed and geometry could be used to quantify, at any point out on the road, the effect of grades and curves on operating costs. For example, the effect of a steep positive grade on a vehicle could be greatly reduced by the momentum gained from an immediately preceeding negative grade. Also, that the effect of horizontal alignment was conditioned to a large extent by the vertical profile. For example, the slowing effect of a small radius curve on a level road is much greater than the same ifies the utilization prediction equations derived from preliminary analyses. The exponential model predicts for five vehicle classes - cars, light, medium and heavy trucks, and buses. 158 company/cell means are from a sample of 939 vehicles.

Model:

$$KMES=e^{9.0873-0.5801H+C1\ 0.3532+C4\ 0.2333-IRR.V\ 0.0004-AG\ 0.0558+V.DPM\ 0.0098} \qquad (3)$$

Where:

KMES = Average monthly utilization in kilometers
H = Horizontal geometry index
IRR = Roughness, QI*
V = Vertical geometry index
C1 = 1 if cars, 0 if other vehicles
C4 = 1 if medium trucks, 0 if other vehicles
AG = Mean age per company/cell minus mean age per cell
DPM = Mean days operational per month per company/cell minus mean days operational per cell

In the equation the effect of roughness can be examined for medium trucks by setting V equal to 8 and H equal to 0.15. For these combinations of V and H the mean days operational per month on paved routes with roughness 39 QI* was 13.3 and on unpaved routes with roughness 140 QI* the value was 18.2. The mean ages were 7.0 and 6.6 for the paved and unpaved routes respectively. Therefore, if these sample means for age and days operational are entered into the utilization equation the AG and DPM terms equal zero and the model becomes a function of the roughness, horizontal and vertical indices. On the paved route the predicted annual utilization is 108,000 km for 7 year old vehicles operating 13.3 days per month. For the unpaved route annual utilization is 77580 km for 6.6 year old vehicles operating 18.2 days per month. On paved routes, utilization is about 39% higher, even though vehicles are operated 5 days per month less.

It is possible however to estimate the roughness influence if 6.6 year old vehicles on paved routes are allowed to operate 18.2 days per month, i.e. as for unpaved routes. In this case, the predicted annual utilization is 162000 km or 109% higher. It is therefore suggested that the sample mean vehicle characteristics be used when applying all equations, since entering other values may lead to extrapolation beyond acceptable limits.

Other Results

Table 1 gives the regression equations for fuel consumption of cars, buses and medium and heavy trucks, parts costs and labor hours for buses, and tyres for buses.

Discussion of Preliminary Results

In the equations produced so far, road roughness is the most important influence. In the case of fuel for medium and heavy trucks, the influence of a steep vertical profile is substantial, increasing consumption by about 40% on paved roads and 33% on unpaved roads. For car fuel, high horizontal curvature increases consumption by about 9% on paved roads but the effect is reduced to 3% on the average unpaved road. No vertical influence was detected, probably because the range of V is narrower than for other vehicle classes. However even if such powerful effects of road geometry on fuel consumption are confirmed in further analyses, roughness will continue to be the dominant influence when considering total vehicle operating costs, especially economic costs. (i.e. less taxes and transfer payments). This is because fuel is a relatively minor item in terms of total operating costs. Table 2 shows economic cost differences for the typical 4 year old bus in rolling terrain (V=6.0, H=0.15) for a paved (38 QI*) road and an unpaved (137 QI*) road. March 1979 prices are used, when 1 dollar = Cr$ 23,00. The interest rate used is 10%. Table 3 gives supporting information. Fuel costs increase by 6% for the unpaved road, but other costs increase by 60%. For the paved road, vehicle maintenance plus depreciation cost almost twice as much as fuel, and for the unpaved road, three times as much.

Summary and Conclusions

Useful results have been derived from preliminary analyses, however final analysis of the complete data sets will continue for some time. The effects of geometric parameters on operating costs will be examined in more detail, and attention will also be given to vehicle characteristics, for example engine size, since it is apparent that to some extent vehicle operators choose or adapt vehicle specifications according to road surface type or geometric characteristics. For this reason, comparisons of different roads using the same vehicle are unwise. Analysis of diesel buses so far shows that road roughness has the strongest influence on total operating costs, but only a small influence on fuel consumption.

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Table 1. Preliminary regression equations

Car Fuel (gasoline) 98 vehicles

$$\mathrm{COMB}=85.34+.078\mathrm{IRR}+46.53\mathrm{H}-.201\mathrm{IRR}*\mathrm{H}+.0316\mathrm{ENGP} \quad \text{(4)}$$

Medium and Heavy Truck Fuel (diesel) 120 vehicles, 31 means

$$\mathrm{COMB}=78.35+.909\mathrm{IRR}+33.805\mathrm{V}-.0879\mathrm{IRR}*\mathrm{V}+9.206\mathrm{PB1}+.0001527\mathrm{IRR}*\mathrm{ENG1} \quad \text{(5)}$$

Bus Fuel (diesel) 582 vehicles, 66 means

$$\mathrm{COMB}=289.9+.165\mathrm{IRR}-.0062\mathrm{ENG1}+3.63\mathrm{AGE1} \quad \text{(6)}$$

Bus Parts (Cr$) 513 vehicles, 61 means

$$\mathrm{PSM}=30.23+2.568\mathrm{IRR}-.0039\mathrm{KM1} \quad \text{(7)}$$

Bus Workshop Labor Hours 603 vehicles, 17 companies

$$\mathrm{HORAS}=7.38\ .087\mathrm{IRR} \quad \text{(8)}$$

Bus Tires 3253 tires, 12 companies

$$\mathrm{ENT}=51.117-.197\mathrm{IRR}+11.712\mathrm{RC1} \quad \text{(9)}$$

Where:

COMB = Fuel in liters per 1000 km
PSM = Parts costs in Cr$ of January 1976, when 1 dollar = Cr$ 10,00
HORAS = Workshop man-hours per 1000 km
ENT = New cross ply tire life in 000's km
IRR = Roughness QI*
H = Horizontal Index
V = Vertical Index
ENGP = Adjusted engine size (cc) covariate, (cars)
ENG1 = Engine size (cc) covariate, (trucks and buses)
PB1 = Gross vehicle weight (tonnes) covariate, (trucks)
AGE1 = Age (years) covariate, (buses)
KM1 = Utilization (000's km/month) covariate, (buses)
RC1 = Recapping (No. recaps per new tire) covariate, (bus tires)

Table 2. Economic costs per 1000 km-4 year old bus

Item	Paved Road Cr$	Paved Road %	Unpaved Road Cr$	Unpaved Road %
Fuel	888	22.3	939	15.9
Tires	528	13.3	956	16.2
Parts	307	7.7	917	15.6
Workshop Labor	384	9.6	695	11.8
Driver	923	23.1	1171	19.9
Depreciation and Interest	953	23.9	1209	20.5
Total	3983	100	5889	100

Table 3. Bus costs - supporting data

Item	Paved	Unpaved	Financial Cost	Taxes
Fuel	296 lt/1000 km	313 lt/1000 km	Cr$ 4,80/liter	60%
Tires	43614 km	24111 km	Cr$ 4800(x 6 tires)	25%
Parts	Cr$ 384/1000 km	Cr$ 1146/1000 km	March 1979 prices	25%
Workshop Labor	10.67 hs/1000 km	19.30 hs/1000 km	Cr$ 40,00/hour	10%
Deprecia-tion	Cr$ 5917/month	Cr$ 5917/month	Cr$ 89000 in 4th year	25%
Interest	Cr$ 3000	Cr$ 3000	Cr$ 45000 in 4th year	25%
Driver	Cr$ 8640	Cr$ 8640	Cr$ 9500/month	10%
Km/month	9356	7377	-	-

RELATING VEHICLE OPERATING COSTS TO LOW VOLUME ROAD PARAMETERS IN BRAZIL

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Overview of a 12 million dollar Brazil Ministry of Transport and United Nations Development Program supported research study aimed at determining relationships between vehicle user costs, roadway design standards and maintenance policies for low volume roads. The relationships are based on an analysis of data generated in a survey of vehicle users operating on over 60,000 km of homogeneous paved and unpaved routes that has produced up to two year of user costs data on over 1200 vehicles, together with 450,000 observations based on radar speed measurements of vehicle behavior in Brazil plus fuel measurements from nine project test vehicles. Finally, data developed from 187 paved and unpaved roadway test sections is being used to develop equation to predict road performance. All of the relationships are being incorporated into a computer based planning model to be used for evaluating alternative highway transportation strategies.

A 12 million dollar research project aimed at improving highway transport planning is concluding in Brazil. This jointly sponsored Brazilian Government - United Nations Development Program (UNDP) project started in July 1975. It is being conducted by Empresa Brasileira de Planejamento de Transportes (GEIPOT) and Departamento Nacional de Estradas de Rodagem (DNER), as arms of the Brazilian Ministry of Transport, and by Texas Research and Development Foundation (TRDF), as the contractor for the World Bank, which is serving as the executing agency for the UNDP.

The study reflects the Brazilian Government's desire to improve planning tools used for allocating scarce highway resources, particularly analytical procedures used to predict the economic consequences of alternative highway design standards and maintenance policies. An important component of these economic analyses is the determination of vehicle operating costs. The relationship between these costs and roadway characteristics, particularly for low-standard roads, has not been well defined in the scientific literature. Further, the relationships that are reported in the literature do not necessarily apply to Brazil. The Brazilian Government expects this study to provide the basis for updating and developing improved relationships between roadway and design maintenance standards and the cost of vehicle operation; the study is also seen as the foundation for establishing objective economic criteria for policy decisions directed toward minimizing total highway transportation cost.

The final product of the Brazil study is to be a computer-based planning model that incorporates the relationships developed from the data collected over the past four-years study period. The models basic function is to determine, for a specified roadway design and maintenance policy, the total transportation costs for a given road project. Total transportation costs is the sum of all construction, maintenance and road user costs over the life of the project, which may be 20 years or more.

The model is based on the adaptation of an existing model entitled "The Highway Design and Maintenance Standards Model (HDM)", which is the result of a series of studies initiated by the World Bank in 1969. These studies had as their objective the development of an analytical model to predict the costs of different highway design and maintenance options for low-volume road projects. The Bank first contracted a Research Group at the Massachusetts Institute of Technology to develop a conceptual framework for interrelating construction, maintenance and vehicle operating costs. A framework schematic for the resulting model is shown in Figure 1.

Many of the relationships used in the first conceptual model could not be supported by sound empirical data, but this deficiency did indicate areas of needed research. Subsequent efforts have involved field studies that focus on developing primary information to be used to improve the empirical relationships between vehicle operating components and roadway characteristics. The first such effort in Kenya between 1971 and 1975, was a cooperative venture between the World Bank and the Transportation and Road Research Laboratories (1).

The Kenya Study (1) succeeded in expanding the understanding of the underlying engineering-economic relationships between vehicle speed, user cost and roadway condition. This new information is incorporated in the current HDM model which reflects the best features of the efforts by both the MIT and TRRL. Subsequent efforts are still in progress and include both the work carried out in Brazil and reported herein, and a substantial effort in India which has been underway for the past one and half years.

Research Approach

Figure 2 schematically illustrates the key components of the planning model being adopted in the study. The components being addressed are delineated by dashed lines and include predictions of maintenance quantities, roadway condition and vehicle consumption. The actual research is organized into three study areas, one on pavement performance and the other two on vehicle operations.

The pavement and maintenance studies have been structured to determine how different pavement designs perform for two extreme levels of maintenance in the Brazil environment. The objective is to predict the condition of a paved or unpaved roadway surface at any point in time for inclusion in the planning models. Knowing the condition of a road surface is important because the quality of the ride, reflected by the interaction between vehicles and the roadway surface, has an important impact on vehicle operating costs.

Two different study approaches were taken for developing improved vehicle operating costs-roadway characteristic relationships. One approach involved defining a series of controlled experiments that are designed to generate data to be used in formulating equations to predict vehicle speed and fuel consumption. In the second approach, a survey of road users was implemented, since no practical experimental procedure seemed appropriate for the other vehicle operation parameters, i.e., tire wear, maintenance, oil consumption, and depreciation.

Factorial experiments were defined and the levels set to insure that the inference space over which data would be collected covered the range of environmental and physical factors that prevail in Brazil. As an example, the design factorial for an experiment directed toward predicting speed patterns by vehicle class is illustrated in Figure 3. This shows that the three factors, surface type, roughness, and vertical profile, are to be investigated for their effect on vehicle speed. It shows that there are two levels of roughness, two levels of surface type, i.e., paved and unpaved, and three levels of vertical profile.

By establishing similar factorial experiments for all study areas it was possible to discern the interaction effects between the various factors being considered, which not only enhanced the analysis but reduced the actual amount of data needed.

Finally, each of the factors identified for study were defined in a manner that allows the study results in Brazil to be used elsewhere in the world. The Brazil results, when combined with previous work in Kenya, in the United States, and work currently underway in India, will permit broad application of the concepts to road problems all over the world.

Organization of the Project

The project was organized to support the three principal study efforts. This is illustrated in Figure 4 where the Surveys, Experiments and Pavement studies are highlighted and shown to be receiving the support of both the administrative and service groups. Ten of the 160 people assigned directly to the project are supported by the UNDP. The actual leadership is shared between the Brazilian and International personnel in a counterpart arrangement as also shown in Figure 4.

Geographic Scope

The original plan called for conducting the entire study in the State of Minas Gerais. This was the recommendation of the French Consultant ("Centre Experimental de Recherches et d'Études du Bâtiment et des Travaux Publics") in their preliminary planning report for this project to the Brazilian Ministry of Transport in 1973 (2).

GEIPOT is located in the Federal District so when the study was actually initiated in mid-1975 the original study area included Goiás and Minas Gerais and the D.F. area itself as shown in Figure 5. Later, to satisfy the inference space established in the experimental factorials, the States of Mato Grosso, São Paulo, Rio Grande do Sul and Bahia were added. Consequently, more than half of the paved and unpaved kilometrage in Brazil are included in the project study States.

Instrumentation and Equipment

One of the major project undertakings has been the acquisition, modification and fabrication of various instrumentation and equipment needed to carry out the substantial measurement program required for the majority of the project activities. An even more substantial challenge has involved the maintenance and repair needed to keep this equipment, valued at over $750,000, operational during the conduct of the study. Some of the more substantial equipment purchases included a Dynaflect, a Weigh-in-Motion System (WIM), a Dynamic Modulus Tester (MR), a GMR Profilometer, and an Analog-Digital (A/D Converter System).

The Dynaflect, together with a Benkelman Beam are being used to measure the deflections on 140 paved test sections three times per year. The WIM was rotated between six locations where it was operated continuously at each site for seven days, 24 hours per day to develop truck weight data. This data is being used to establish axle-load information to be used in analyzing pavement performance.

The Dynamic Modulus equipment is used in the project's soil laboratory to establish the strength characteristic of bituminous cores taken from the project paved test sections. The GMR Profilometer is used to standardize road roughness measures while the A/D System is being used to transform data to digital format for computer processing.

Modified or fabricated to meet project requirements were seven Mays-Ride-Meters, nine Volumeter Fuelmeters, three Camera Boxes systems, needed to develop incremental distance measurements, three survey vehicle measurement systems that were developed for rapidly collecting roadway vertical and horizontal geometry together with roughness, and finally a Traffic Flow Data Logger (TFDL). This latter equipment was designed to automatically generate spot speed, space mean speed and headway data for calibration of project traffic-flow models. All of the project equipment has been completely documented as to its design, calibration, operation and maintenance. This documentation is available to anyone on request.

Elements Requiring Major Attention

Using economic analyses for the highway project planning process has steadily increased over the years although mostly for high-volume highway arterials and for streets in urban areas. The underlying foundation of information needed to permit a effective analysis related to low-volume roads is very deficient, particularly in being able to explain the influence of varying roadway surface conditions on user costs. On unpaved access roads, both the vertical and horizontal alignment can be very severe. The influence of extremes in this geometry on vehicle operating costs need to be better defined.

A broad spectrum of cause and effect relationships between roadway characteristics and vehicle operating costs were addressed in the study. Special efforts were made to insure that reliable information would be developed between roadway surface conditions and user cost so that the following questions could be answered.

1. How do varying highway maintenance policies influence the performance of a given road pavement design?
2. What is the influence of roadway performance on road surface conditions and how does this surface condition affect vehicle
   a) speed,
   b) fuel consumption,
   c) tire wear,
   d) maintenance and repair,
   e) oil consumption?
3. How do roadway characteristics and surface conditions influence vehicle utilization and service life?

Road surface condition for practical reasons means road roughness because it is the roads riding quality that affects the vehicle and, therefore, vehicle operating costs. This is a critical variable to decision processes because poor roadway performance manifests itself in a rough ride to the roadway user. Poor riding quality is translated into increased vehicle operating costs, so decisions relating to surface design standards and levels of roadway maintenance eventually impact directly on vehicle operating cost. Figure 6 shows the 40 major variables being addressed in the study. Roughness is shown to be effected by 17 and to directly affect seven others or over 60% of the interactions being studied. There is also a recognized need in the world to have a standardized roughness measure that can be used in comparing roadway performance and the resulting roadway condition on vehicle operating costs in different countries. Consequently, diligent efforts were made during the study to standardize the roughness measures obtained.

The principal roughness measurement instrument has been the Mays-Ride-Meter. This equipment, modified to permit direct digital output, was installed in seven vehicles. These seven roughness measurements systems have been operating continually throughout the study period and between them have been driven over 70,000 km in the process of developing 50,000 km of roughness measurements on paved and unpaved roadways in Brazil.

The reliability of the measurements obtained with the Maysmeter has been rigorously controlled. Each system is checked at the beginning and end of a day's work to insure the measurements it is generating are under control. Anytime a change in the response of a Mays-Ride-Meter system is observed it is completely recalibrated. This involves running the system over 20 control sections covering a spectrum of roughness conditions. The runs are made over all 20 sections on five different days, and the average roughness numbers are used to define a calibration curve which is used to transform the Mays-Ride-Meter system measurements to a standard roughness measurement.

A GMR Profilometer served to define the unchanging roughness standard that was used throughout the study period. This was accomplished by making periodic measurements of each control section with the GMR Profilometer. These measurements were used to define the roughness of each control section to which each Mays-Ride-Meter system was calibrated. Use also was made of the GMR Profilometer in transforming the roughness measurements obtained in Kenya (1) and those currently being obtained in India to the scale being used in Brazil. This allowed a direct comparison to be made between the relationships involving roughness for the three different studies. Recommendations for a worldwide roughness standard are currently being reviewed and studied. Regardless of the form the final standard takes, it will be possible to transform the Brazil measures to this final standard using the permanent tape records obtained with the Profilometer.

Pavement and Maintenance Studies

The Pavement and Maintenance Studies had as their ultimate objective establishing equations to predict the performance of any roadway in terms of its roughness. Such equations in the hands of decision makers is an invaluable tool, one that permits them to examine alternative design standards, construction strategies, and maintenance policies and for each combination predict the resulting roadway riding quality at any point in time.

The equations developed to predict the performance of both paved and unpaved road are based on information generated from a series of test sections. The paved sections were selected to satisfy a 64-cell design factorial where two levels were designated for each of six factors as shown in Figure 7.

Eventually, 128 different paved test sections were established. Each section was split into two 320-meter segments where two extreme levels of maintenance were sought. For the low or nil level of maintenance only pothole patching was permitted and this for safety reasons. For the high level of maintenance sections, the predominant maintenance response was to slurry seal the surface at the first sign of cracking. In the case of severe distress, complete replacement in kind of base and surface were implemented.

There were 48 unpaved test sections and these were also split into two segments. The principal maintenance response was roadway grading and this was varied from two weeks to five months so that data on roadway performance for various grading frequencies could be established.

A substantial measurement program was established to develop data on the performance of each of the test sections. For the paved sections this involves a detailed surface condition survey every four months on each section. Further, periodic road roughness and deflection measurements are obtained on each section. The roughness measuring system was described earlier. The deflection measurements are being made with both a Dynaflect and with a Benkelman Beam. The initial material characterization test on each section were subcontracted to a soils consultant and consisted of measurements for field density, field CBR, field moisture, and layer thickness. From samples of each layer, laboratory testing has established material grading, Atterberg limits, and laboratory CBR and density.

Road User Costs Survey

Over 200 vehicle users having fleets of either passenger cars, buses or trucks varying from 1 to 1000 vehicles are participating in the survey of vehicle operating costs. The cost components being addressed in the survey include fuel, oil, tires, maintenance parts, labor and depreciation. The routes over which each vehicle operates are essentially homogeneous in character. Quantitative measures of the roadway characteristics have been obtained for each route using the survey vehicles described earlier; included are horizontal and vertical geometry measurements on each route together with continuous roughness measurements at .2-mile intervals.

This is the largest survey ever undertaken to develop relationships between roadway characteristics and vehicle operating costs. New and meaningful relationships have been established for all of vehicle operating cost elements. Depreciation is one of the most important because almost no empirical data exist which can be used to substantiate a real-world-based procedure for defining differential depreciation rates on roads having different characteristics.

The rate at which the capital investment in vehicles is depreciated is normally tied closely to the life time mileage of a vehicle which in turn is a function of the vehicle's years of service and annual mileage. In general, road improvements are expected to reduce the physical wear on vehicles and increase speed. Both of these effects are assumed to increase the vehicle's life time mileage by first extending the service life and second by increasing the annual mileage, although not proportionately, because increased speed also accelerates the wear on a vehicle. The problem is to establish if and how a vehicle's service life and annual mileage is actually related to operation on roads of widely ranging character, i.e., different geometry and surface conditions.

Road User and Traffic Experiments

Time and fuel savings are two of the established benefits that accrue to highway users through road improvements. Considerable published information exists on both the prediction of vehicle fuel consumption and speed as a function of roadway characteristics. However, the available information is not necessarily applicable to Brazilian vehicles and conditions. Further, this information does not adequately address the effect of roadway condition on speed and fuel. Nevertheless, the ability to design experiments to develop detailed relationships between roadway characteristics and vehicle speeds and fuel consumption is well documented.

Brazil, like many other countries throughout the world, is concerned about the increasing levels of oil consumption, particularly for highway travel. Therefore, understanding how decisions related to highway design standards and maintenance policies influence vehicle consumptions takes an added importance. Given the desire to identify the parameters influencing fuel consumption, particularly road surface condition, and the demonstrated ability to design experiments that can produce the required detailed relationships, the decision was made to develop a computer-based model (TAFA) that would simulate the speed and compute the fuel consumption for a vehicle as it traversed a section of road with given vertical and horizontal alignment and road surface conditions.

Thirty different individual experiments were identified, designed and implemented to develop the data required to establish the relationships needed to create and calibrate TAFA. The speed experiments involved monitoring the behavior of traffic at 176 control locations established along the roadway. The sections were selected to cover as wide an inference space as possible so that the data being analyzed included a wide spectrum of grades and roughness on both paved and unpaved roadway sections. At each location, the road characteristic and the environment during the period of observations were identified through measurements of:

1. Road characteristics including:
   a) grade
   b) horizontal alignment
   c) pavement width
   d) surface type
   e) surface roughness
   f) surface rutting
   g) gravel looseness (unpaved)
   h) surface material moisture (unpaved)
2. Environmental characteristics including:
   a) rainfall
   b) dust level
   c) altitude
   d) wind velocity and direction
   e) cloud cover
   f) traffic volume and composition

Nine vehicles were purchased and instrumented for the fuel studies. They included a VW1300, two VWKombis, one gasoline-powered truck, four diesel-powered trucks, and a diesel bus. This emphasis on diesel vehicles was supported by the current trend showing increased use of these more economical fuel consumption vehicles on the road. The same parameters identified for speed measures applied to the 28 sections selected for the fuel experiments.

Over 450,000 observations were obtained through the speed and fuel experiments. These data were subjected to rigid checking before being incorporated into computer files. Further screening and review of computer-generated reports of the field data files normally resulted in the identification of some data discrepancies. Where the discrepancies could not be satisfactorily resolved, the experiments were repeated and the original data discarded. The final step in data processing was to combine various files identifying test section parameters with the speed or fuel observations and structure appropriate analysis files.

The final data analysis files were considered to be both accurate and reliable and were subjected to very rigorous analysis. The final relationships developed include every variable and variable interaction that could be identified as significant in explaining any of the variation in either of the observed dependent variables, speed and fuel consumption. Also, wherever possible, the equations were generalized, i.e., if vehicle classes could be combined into a single equation, they were combined.

Information Dissemination

Because of the wealth of new information and the nucleus of new knowledge generated in Brazil, the project team has tried to completely document every significant facet of the project. Three levels of reporting were initiated. These include Formal Reports, Project Working Documents and Project Technical Memos.

The 12 formal project reports expected from the project include the following:

1. Inception Report (3)
2. Midterm Report (4)

3. Instrumentation Report
4. User Survey Design and Methodology
5. User Survey Analysis and Results
6. User Experiments Design and Procedure
7. Project Data File Documentation
8. User Experiments Analysis and Results
9. Pavement and Maintenance Studies Design and Procedures
10. Pavement and Maintenance Studies Analysis and Results
11. Executive Summary
12. Documentation and Users Guide for Planning Model (MOBAIR)

The Project Working Documents form a series containing information related to the research that is too detailed to be included in the Formal Reports. They include detailed tabulations of data, descriptions of each of the Experiments, documentation and user guides for project-generated programs and operational and maintenance manuals for various instrumentation systems.

The Technical Memos were devised to capture significant facets of the project as they were completed. These include the formation of procedures to collect data, defining approaches to the analysis of information, interim results of isolated and completed segments of the study, and detailed descriptions of some of the instrumentation developed for the study. The Technical Memos have been consolidated annually into a Working Document so they can be more easily referenced and disseminated.

The majority of the data being generated in the study is being handled by computer-based data processing systems. Over 100 files have been created to store project data, and each of these files is being completely described in a project formal report entitled Project Data File Documentation.

The ICR Model

The final product of this research project is an operational Highway Investment Model that incorporates the relationships developed during the study. The program structure for this model already existed in the Highway Design and Maintenance Standards Model (HDM) developed from the combined MIT/TRRL/WORLD BANK effort. A version of the HDM was adopted for the project in mid-1978 and a series of modifications implemented to transform the HDM for use in Brazil. The modified HDM was designated ICR and a number of different ICR versions have been produced to handle, in a stepwise manner, each of the modifications. These include the conversion of all output to Portuguese, and the establishment of routines to handle new variables which are needed for changes in the subroutines for predicting pavement performance and vehicle operation.

The detailed speed and fuel consumption information that can be generated through using the Time and Fuel Algorithm (TAFA) is being handled at both a micro and macro levels, and therefore two different ICR versions are being produced. An entirely new subroutine for computing depreciation has been developed and major changes were made in the subroutine used to determine vehicle oil consumption, tire wear, and maintenance and repair costs. The final model will be called Modelo Brasileiro para Avaliação de Investimentos Rodoviários (MOBAIR-1) and represents the final version produced by this project. Further improvements will unquestionably continue to be made in the future as additional data is collected and data files are further expanded. In subsequent analyses the individual equations will be improved further and the new relationships will be incorporated into MOBAIR-2 and later versions of the model.

MOBAIR-1 is expected to find a number of specific applications in Brazil. It can be applied to any proposal for the improvement of an existing highway facility or the construction of a new facility. The application can be to a specific project, a number of projects making up a long route, a system of highways or a complete geographic area varying from a local municipality to the entire country. Another prevalent use will be in selecting between alternative route locations both at the feasibility and pre-feasibility level of study or to make comparisons between reconstruction and/or construction on existing alignment or right-of-way and that of a new location.

Considerable interest has been expressed for using the model to set design guidelines for both vertical and horizontal geometry. This is an area where the micro subroutine version of TAFA can have application. In addition, some other uses that can be made of the model include establishing warrants for truck lanes on steep grades; making decisions related to stage construction options, establishing policies on permissible vehicle weights and dimension or vehicle speed limits, and of course in the important area of establishing highway maintenance levels.

A Brazil Highway Maintenance Study has been underway concurrently with this reported study and the individual maintenance activities included in the model are based on the productivity and labor, equipment and material requirements established in the maintenance study. Consequently, the MOBAIR Model can be used to examine alternative maintenance strategies. The resulting predictions of maintenance work can be used as a direct input into the budget procedures established for managing highway maintenance in Brazil.

Summary

The Brazil study has expanded the empirical foundation on which many vehicle-operating cost relationships are based. These new relationships between vehicle-operating costs and roadway characteristics, traffic and environmental conditions are being incorporated into a highway planning model that will be known as MOBAIR. This model is expected to be implemented widely in Brazil. Further the model and the underlying relationships together with the data used in their development are available for use throughout the world.

References

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3. Inception Report, Research Concepts and Procedures. Research on the Interrelationships between Costs of Highway Construction, Maintenance and Utilization, (UNDP Project BRA/74/012)-GEIPOT, UNDP, IBRD, Brasilia, Brazil, Apr. 1976.
4. Report II, Midterm Report, Preliminary Results and Analysis. Research on the Interrelationships between Costs of Highway Construction, Maintenance and Utilization, (UNDP Project BRA/74/012)-GEIPOT, IBRD, Brasilia, Brazil, Aug. 1977.

Figure 1. Flow chart for highway cost model developed in the MIT study (Ref 1)

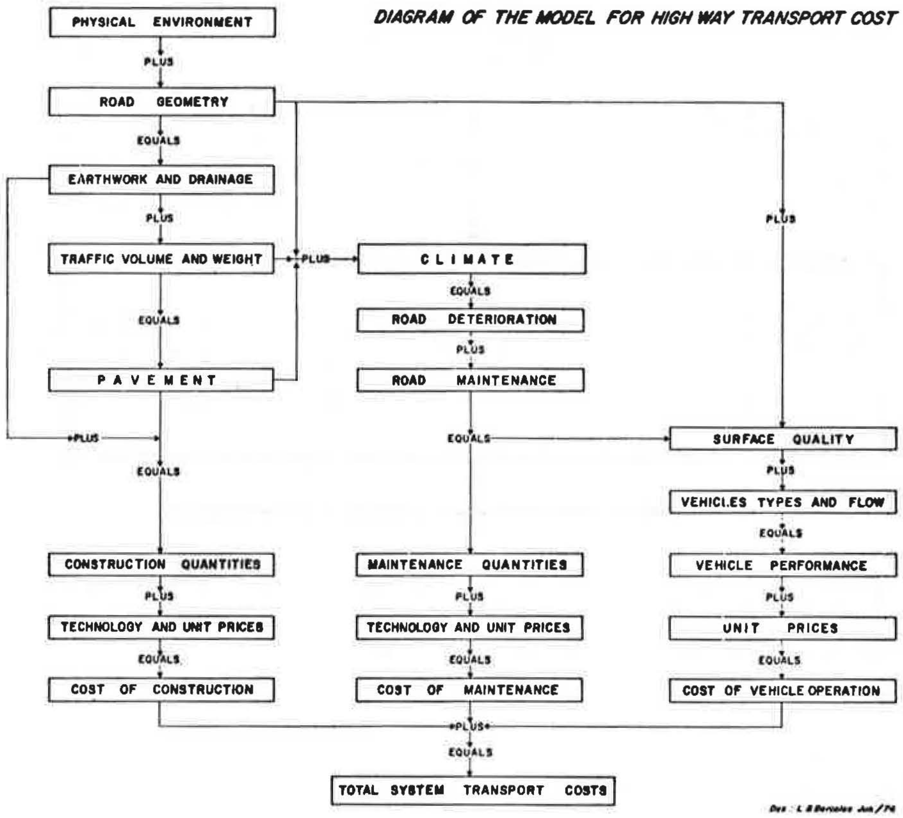

Figure 2. Conceptual framework of highway planning model showing module interfacing

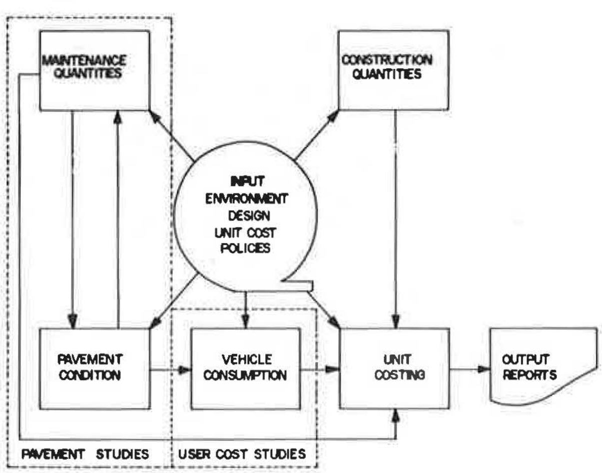

Figure 3. Sampling frame for traffic on positive grades, experiment TB-1

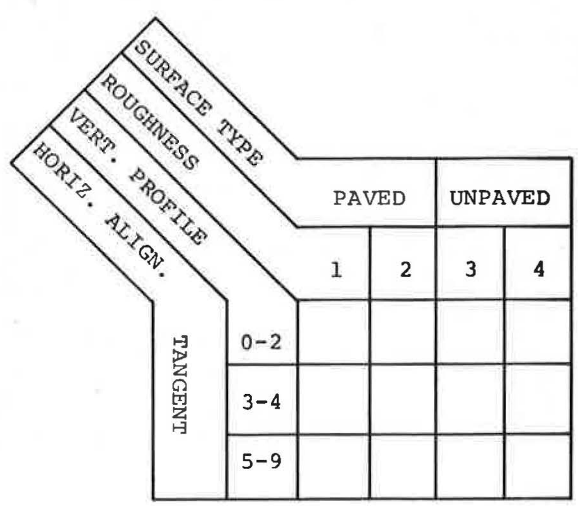

Figure 4. Functional organization chart of project

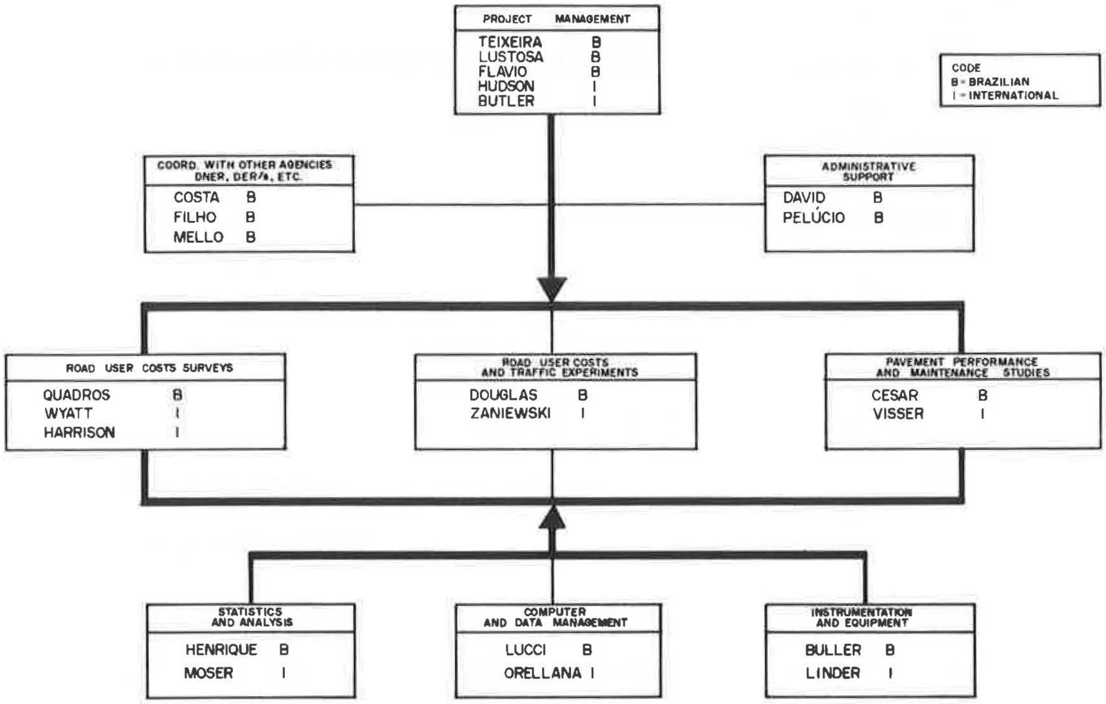

Figure 5. Summary of the variables that have been identified for study in the project

PREDICTION

FACTORS	Annual Vehicle Use	Vehicle Speed	Depreciation	Maintenance Repair	Tire Wear	Oil Consumption	Grease Consumption	Fuel Consumption	Rut Depth	Pavement Cracking	Surface Looseness	Gravel Loss	Road Roughness
Vehicle Speed	•		•					•					
Traffic Volume	•	•						•	•	•	•	•	•
Traffic Composition	•	•						•					
Accummulated Axle Loads									•	•	•	•	•
Vehicle Class	•	•		•	•	•	•	•					
Annual Vehicle Use			•	•	•								
Vehicle Age	•		•	•	•	•	•						
Vehicle Load	•	•		•	•	•	•	•					
Vehicle Gear		•						•					
Vehicle Activity	•		•	•	•	•	•	•					
Rain Fall	•							•	•	•	•	•	•
Vertical Geometry	•	•		•	•	•	•	•	•	•	•	•	•
Horizontal Geometry	•	•		•	•	•	•	•	•	•	•	•	•
Road Surface Type	•	•		•	•	•	•	•	•	•	•	•	•
Roadway Age									•	•	•	•	•
Rehabilitation Age									•	•		•	•
Material Characteristics									•	•	•	•	•
Pavement Structure No.									•	•		•	•
Pavement Deflection									•	•		•	•
Pavement Cracking												•	•
Road Rut Depth												•	•
Road Roughness	•	•		•	•	•	•	•					
Surface Looseness		•									•	•	•
Regraveling Rate											•	•	•
Blading Frequency										•	•	•	•
Maintenance Level										•	•		•
Roadway Dust		•											

Figure 6. Map of Brazil showing the study's final geographic coverage by state

Figure 7. Sampling matrix for the paved road experiment

SURFACE TYPE			ASPH CONC				D SURF TREATM			
BASE TYPE			GRAVEL		C STONE		GRAVEL		C STONE	
TRAFFIC (ADT)			50-500	>1000	50-500	>1000	50-500	>1000	50-500	>1000
STATE REH.	AGE	V. GEOM. (%)								
OVERLAYED	≥ 8	> 6								
		0-1.5%								
	0-2	> 6								
		0-1.5%								
AS CONSTRUCTED	≥ 12	> 6								
		0-1.5%								
	0-4	> 6								
		0-1.5%								

FUEL CONSUMPTION RELATED TO VEHICLE TYPE AND ROAD CONDITIONS

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Pedro José de Morais, Brazilian Road Research Institute
Russ L. Kaesehagen, Western Australia Main Roads Department

An investigation of the effect of road condition and vehicle type on fuel consumption was conducted in Brazil. A test fleet of nine vehicles ranging from a Volkswagen to an articulated truck was used to perform nine experiments. Roadway characteristics studied in the experiments include horizontal curvature, grade, surface type, and roughness. Vehicle operating modes studied were constant speed, acceleration, deceleration, and deceleration due to gravity. In addition, an experiment was performed to determine the effect of adding 20% alcohol to gasoline. Regression equations for predicting fuel consumption as a function of speed, and vehicle and roadway characteristics were developed for constant speed operation. It was found that curves of 72 m in radius did not cause a significant effect on fuel consumption. The inclusion of 20% alcohol in gasoline was found to result in approximately a 10% increase in fuel consumption.

The Government of Brazil and the United Nations Development Program are sponsoring a research project on the Interrelationships between Highway Construction, Maintenance and User Costs. Within this research, extensive measurements were made to establish relationships between fuel consumption and roadway characteristics. Experimental data were collected with a nine vehicle test fleet performing virtually every phase of vehicle operation over test sections with a select combination of roadway characteristics. A summary of the experimental program is presented herein followed by a detailed description of the methodology, equipment, and results of experiments on constant speed, road curvature, and using 20% alcohol in gasoline.

Experimental Program

For these experiments the inference space, or the region for which the results apply, is established by the range of the independent variables and by the vehicles which are represented in the study. One or two vehicles were tested for each vehicle type and applications of the models must be made with this knowledge. Experiments were conducted to study fuel consumption in each phase of vehicle operation: constant speed, acceleration, deceleration and momentum. Additional experiments were performed to determine fuel consumption as the vehicles traversed horizontal curves and sag curves. The effect of vehicle tuning and using an alcohol-gasoline mixture was investigated by partially repeating the constant speed experiment. The latter experiment was performed and was included because a mixture containing 20% alcohol, by volume, is sold in Brazil. Finally, a calibration experiment was performed for validating the computer program for predicting fuel consumption (1). Thus, the experimental program consisted of nine individual experiments as summarized in Table 1.

Roadway Parameters Studied

Roadway characteristics were used as independent variables in each of the experiments. After considering an extensive list of roadway parameters, the four thought to have the most significant effect on fuel consumption were selected: surface type, curvature, roughness, and grade. Two levels of surface type were tested, bituminous surfaces (asphalt concrete, and surface treatments), and unpaved roads. Lateritic gravels were studied for the unpaved roads due to their predominance in the study area. Except for the curvature and calibration experiments, sections with high speed alignment were used. A smooth and rough level of roughness was included for each of the surface types. Roughness measurements were made with a Maysmeter (2). Due to the importance of grade, up to seven levels were used in the experiments. The combinations of factors and levels selected for each of the experiments are shown in Table 1. All of the vehicles operated on all the sections except in the constant speed experiment, the light trucks and the bus did not operate on the 13% section and in the curvature experiment the light diesel truck was not included.

Measurements in the momentum experiment were started at the base of the grade and terminated when the vehicle reached steady state speed. Thus the length of the measurement area was variable. In the calibration experiment 10 km long test sections were used. For all other experiments sections 2 km long were sought to provide 500 meter transitions and 1000 meters for measurements. A minimum of 100 meters for the transitions and 600 meters for the measurements were accepted. The properties of the

Table 1. Summary of experimental program.

Experiment	Description	Levels of Roadway Factors												
		Surface Typed		Curvature		Roughness		Grade (%)						
		Bituminous	Gravel	Tangent	Curved	Smooth	Rough	0	2	4	6	7[a]	8[b]	13[b]
Constant Speed	Constant speed operation on positive and negative grades	x	x	x		x	x	x	x	x	x	x	x	x
Acceleration	Vehicles accelerate from zero to maximum speed on positive and negative grades	x	x	x		x	x	x	x	x	x	x	x	x
Deceleration	Vehicles decelerate from maximum speed to zero on positive and negative grades	x	x	x		x		x			x			
Momentum	Vehicles decelerate from high entry speed to crawl speed on positive grades using full power	x		x		x				x		x		
Curvature	Partially repeat a constant speed experiment on one section with extreme curvature		x		x	x	x						x	
Sag Curves	Vehicles use full power to traverse sag curves on two sections	x		x		x								
Untuned	Partially repeat constant speed experiment with vehicles deregulated	x	x	x	covariate			x			x			
Alcohol	Partially repeat constant speed experiment with gasoline vehicles using a mixture of 20% alcohol and 80% gasoline	x	x	x	covariate			x		x	x			
Calibration	Operate test vehicles over longer sections using normal driving procedures	x	x		x	x		high/low						

a) paved only

b) unpaved only

test sections used in the constant speed, curvature and alcohol experiment are given in Table 2.

Other Independent Variables Measured

In addition to the specific roadway characteristics studied, several variables were measured that describe the environment and roadway conditions. Environmental variables included temperature, wind speed and direction, and rainfall. Roadway covariates measured were rut depth of all sections, and looseness, moisture content, and spacing of corrugations of gravel test sections.

Equipment

Vehicle Fleet Characteristics

The test vehicles were selected to be representative of those operating on rural roads in Brazil. The fleet included seven different vehicle types and two replicate vehicles to obtain a measure of replicate vehicle variance. The test fleet consists of one car, two utilities, a light gasoline truck, a light diesel truck, two medium diesel trucks, an articulated diesel truck and a rural diesel bus. The properties of these vehicles are shown in Table 3. All vehicles were purchased new and maintained in good mechanical condition.

All trucks were fitted with flat beds for ease in loading. Concrete blocks weighing 665 kg were fabricated with an exposed loop of reinforcing steel for use as loads. One of the trucks was fitted with a hydraulic crane for loading and unloading the trucks. Concrete cylinders of 13.4 kg were used for loading the other vehicles.

Fuel measurements were made with the vehicles empty, and fully loaded to the manufacture's specifications. The medium diesel trucks with a full load was an exception. It weighed more when full than the rated gross vehicle weight because in calculating the loads to apply to these trucks, the weight of the trucks bed was not included in the tare weight for the vehicle as given by the manufacture. In addition, as time permitted, test were made at half load and with a 10% overload.

In order to maintain comparable weights, two of the vehicles carried some load in the "empty" condition. One medium diesel always carried a load approximately equal to the weight of the hydraulic crane mounted in the other medium diesel. In the case of the light trucks, the gasoline truck always carried a load equal to the difference weight of the two motors.

All fuel used was obtained from the normal stock of local suppliers. The standard gasoline prior to August 1978 in Brazil was 73 octane. After this date 20% alcohol, by volume, was added to the gasoline in the study area. Therefore, an experiment to determine the effect of 20% alcohol in gasoline was performed.

Fuel Meters

The fuel meters used were based on the designs

Table 2. Test section properties.

Section No.	Surface[1] Type	Grade[2] (%)	Roughness (QI)	Curves	
				Number	Radius (meters)
503	DST	- 6.1	30-39	1	1011
507	DST	± 6.0	25-31	1	700
508	AC	± 4.0	55-59		
510	AC	+ 5.8	24-33	1	1146
511	DST	- 2.0	23-39		
512	DST	+ 1.9	23-39		
513	GV	± 2.0	42-88		
514	DST	± 0.2	25-30	1	2929
516	GV	± 8.0	43-100	1	650
517	GV	± 8.0	26-100	2	72-101
519	AC	+ 3.8	23-26		
520	GV	± 4.0	57-98, 151-230		
521	GV	± 0.1	38-97	1	702
522	DST	± 7.0	24-31	4	400-550
523	GV	± 6.0	48-99, 146-154	1	1011
528	AC	± 4.0	25-32		
531	DST	± 1.8	107-147	1	603
532	DST	± 2.3	103-158	2	510-702
533	DST	± 3.6	88-125	1	674
535	DST	± 0.3	140-229	2	603-996
536	DST	± 5.8	80-129		
537	GV	± 1.0	76-137		
538	GV	± 6.1	141-293	4	287-350
555	GV	±13.0	141-155	7	84-500

1. AC = asphalt concrete
   DST= double surface treatment
   GV = gravel

2. The sign indicates the direction used during the test. On two sections; tests were run in only the positive direction; two sections were used only in the negative direction; on all other sections tests were run in both directions.

by Abaynayaka (3) and Sawhill and Firey (4). To ensure that the meters were zeroed at the start of a run, a syphon was incorporated into the design. Since the accuracy of this type of fuel meter is a function of its size, tests were performed to determine the minimum size fuel meter which could be used with each vehicle. Thus, the size of the fuel meters varied from 250 ml to two liters.

Due to the size differential, two designs were adopted. The smaller fuel meter consisted of a 250 ml graduated cylinder with appropriate tubes and valves for installation. The larger fuel meters had a two cylinder reservoir. A tube scaled in 10 ml increments was put between the cylinders so the fuel could be accurately read on grades and cross slopes. Fuel readings were interpolated to the closest 5 ml with the large fuel meters and to the closest 2 ml with the 250 ml meters.

Other Equipment

Measurements of looseness and rut depth were performed in accordance with the procedures described by Visser (2). Wind speed was measured with an Airguide Model 918 Windial and the direction was estimated with respect to the axis of the test section.

Each vehicle was equipped with a Numetrics electronic distance measuring instrument (DMI) and either two normal stopwatches or a split second hand stopwatch.

Methodology

A common methodology was used for the constant speed, curvature, and alcohol experiments to simplify crew training and minimize equipment development. The procedure required the observer to zero the fuel meter, stopwatch, and the DMI while the test vehicle was stopped prior to the start of the transition. The driver then accelerated the vehicle to the desired speed by the start of the transition section and then maintained constant speed. As the vehicle passed the start marker of the transition section, the observer switched on the DMI. When the reading on the DMI was equal to the length of the transition section, the observer simultaneously started the stopwatch and the fuel meter. When the DMI showed that the distance travelled was equal to the length of the transition section plus one half the length of the measurement section, one of the stopwatches, or split second hand on that type of watch was stopped. When the vehicle reached the end of the section, the stopwatch was stopped and the fuel meter switched off. The vehicle was then stopped and the measurements were recorded on the data forms. These forms were designed for direct keypunching to eliminate transcribing errors.

Test runs were made in 10 kph increments for all possible speed gear and loading combinations. At least three repeat runs were made for each combination of speed gear and loading condition. In some cases maximum possible speeds could not be realized for safety reasons.

Analysis and Results

Data Processing

Following field data collection, the data forms were manually checked in the office for completeness, keypunched, verified and then edited for logic and limits using a computer. The original data forms were retained in the event of errors being established during the data processing and analysis phases. Due to the magnitude of the experiments, all data were stored on magnetic computer tapes. Space mean speeds for each half and whole run were computed. Fuel consumption was computed in milliliters per second and a summary computer print out of the speed and fuel consumption produced. This output was reviewed for inconsistencies and replicate runs were performed to resolve questionable data.

Tests for Significance

Replicate vehicle and replicate run variances were calculated for the utility and medium truck types. The replicate run variance was one tenth of the replicate vehicle variance for the utilities and one sixth for the medium trucks. Thus, the mean fuel consumption in milliliters per second across replicate runs was used as the dependent variable for all vehicle types (5).

The replicate vehicle variance for the utilities was significantly smaller than for the medium trucks. Since these variances were not homogeneous separate regression equations were developed for each vehicle type in the constant speed experiment.

The replicate vehicle variance was used to test the significance of the independent variables in the analysis of variance for the utility and medium truck types. The pooled variance of the four factor and higher interactions was used to test the independent variables for the other vehicle types. In the case of the utility and medium truck types, using pooled variance would have resulted in conservative tests since the pooled variance was smaller than the replicate vehicle variance. It is therefore expected that the tests for the significant factors for the

Table 3. Properties of the test vehicles.

Type	Car	Utility	Bus	Light Truck	Light Truck	Medium Truck	Medium Truck	Articulated Truck	
Make	Volkswagen	Volkswagen	Mercedes Benz	Ford	Ford	Mercedes Benz	Mercedes Benz	Scania	Rodoviaria
Model	1300	Kombi	0-362 Bus	F-400	F-4000	1113	1113 with Monk	110-39	trailer (3 axles)
BODY (m)									
ground clearance	0.152	0.200	0.273	0.200	0.200	0.279	0.279	0.300	
total height	1.500	1.912	2.945	1.890	1.890	2.454	2.454	2.013	
width	1.540	1.746	2.500	2.030	2.030	2.350	2.350	2.403	
distance between axles	2.400	2.400	5.500	4.030	4.030	4.200	4.200	3.800	8.750
front overhang	0.760	1.130	2.310	0.750	0.750	1.100	1.100	1.480	0.90
rear overhang	0.910	0.867	2.800	2.200	2.200	3.850	4.600	-	2.600
total length	4.070	4.397	10.660	6.980	6.980	9.150	9.900	16.630*	12.250
WEIGHT (kg)									
tare	780	1,195	7,500	2,227	2,444	8,065	9,735	5,583	14,000
GVW	1,160	2,155	11,500	6,060	6,060	18,500	18,500	45,000*	
load level 1	920	1,335	7,640	2,567	2,584	9,935	9,875	19,723*	
load level 2	1,050	1,615	8,650	4,147	4,124	14,190	14,265	33,323*	
load level 3	1,200	1,885	9,890	5,952	5,909	20,175	20,250	46,923*	
load level 4	-	-	-	6,410	6,427	23,378	23,383	48,983*	
MOTOR									
fuel	GAS	GAS	DIESEL	GAS	DIESEL	DIESEL	DIESEL	DIESEL	
type	Horiz.Opp.	Horiz.Opp.	in line	V	in line	in line	in line	in line	
cylinders	4	4	6	8	4	6	6	6	
bore (mm)	77	83	97	92	105	97	97	127	
stroke (mm)	69	69	128	84	120	128	128	145	
displacement (cc)	1,285	1,493	5,675	4,457	4,163	5,675	5,675	11,000	
compression	6.6	6.6	17.0	7.3	17.8	17.0	17.0	16.0	
torque/ RPM (m.kgf)	9.1/ 2600	10.3/ 2600	37.0/ 2000	33.5/ 2200	29.2/ 1600	37.0/ 2000	37.0/ 2000	79/ 1200	
horse power/ RPM (SAE)	48/ 4600	60/ 4600	147/ 2800	169/ 4400	102/ 3000	147/ 2800	147/ 2800	285/ 2200	
DRIVE TRAIN									
Gear 1	3.80	3.80	8.02	6.40	5.90	8.02	8.02	13.51	
2	2.06	2.06	4.77	3.09	2.85	4.77	4.77	10.07	
3	1.32	1.32	2.75	1.69	1.56	2.75	2.75	7.55	
4	0.89	0.89	1.66	1.00	1.00	1.66	1.66	5.66	
5						1.00	1.00	4.24	
6								3.19	
7								2.38	
8								1.78	
9								1.34	
10								1.00	
Differencial	4.375	4.375	4.875	5.140	4.630	4.875	4.875	4.710	
Tire diameter (m)	0.650	0.654	1.016	0.808	0.808	1.016	1.016	1.080	1.080

* Total for tractor-trailer unit.

other vehicle types are also conservative.

Constant Speed Experiment

The significant effects identified in the analysis of variance were entered into a model of the form:

$$FUEL = a_0 e^{(a_1 x_1 + a_2 x_2 + \ldots)} + \varepsilon \qquad (1)$$

The non-linear regression program in the SAS library (6) was used to estimate the coefficients. The equations, their mean square error, $S^2{}_e$, and the R^2 value are presented in Table 4. The equation for the articulated truck varies from the form of equation 1 in that a_0 is divided by the absolute value of negative grades.

Figures 1 to 5 illustrate the form of the equations and the magnitude of the main effects. Figure 1 shows the predicted fuel consumption for each vehicle type. There is a general trend of increasing fuel consumption with vehicle weight. This can be clearly seen in comparing the fuel consumption of the bus and medium trucks. Even though these vehicles have identical drive-trains, the bus comsumes much less fu l than the trucks at half load, due mainly to the 5,500 kg difference in the weights used in this comparison.

One exception to the trend of increasing fuel consumption with weight, is the comparison of gasoline and diesel trucks. The light gasoline truck actually consumes more fuel than the medium weight diesel trucks at speeds less than 45 kph. The fuel consumption of the gasoline truck is always greater than the equivalent diesel truck as shown in Figures 1 and 2. Figure 2 demonstrates that the gasoline truck empty consumes 50 to 100 percent more than the diesel vehicle consumes full. Figure 2 also shows the effect of gears on fuel consumption for the empty gasoline truck.

The effect of load on fuel consumption for the articulated diesel truck is shown in Figure 3. The significance of the load-speed interaction is clearly illustrated as the curve for the loaded vehicle has a greater slope than the empty vehicle. In fact, at 10 kph the fuel consumption is about the same for all three load conditions, but at 70 kph, the vehicle consumes approximately 50% more when loaded.

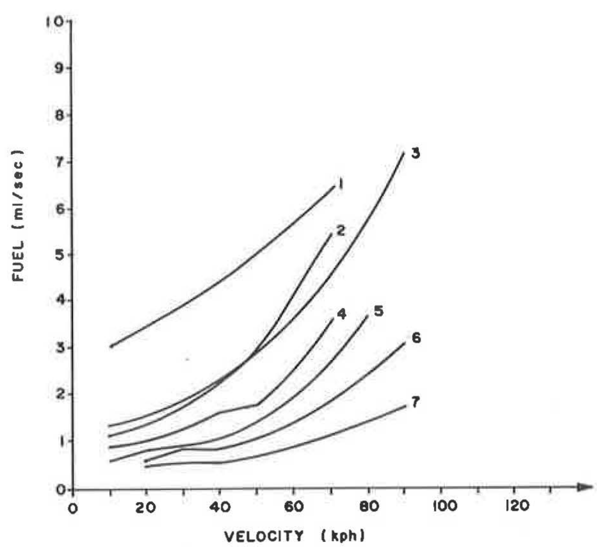
The effect of surface type was tested indepen-

Table 4. Fuel consumption equations (ml/seg).

Vehicle	S_e^2	R^2	
Car	.037	.91	$FUEL=.142e^{.02287S+.000855(S)GR+.03782\ P(GR+3)+.2695(5-GEAR)+.0001024(QI)(GR+14)}$
Utility	.060	.93	$FUEL=.197e^{.02579S+.001062(S)GR+.02932\ P\ (GR+3)+.2485(5-GEAR)+.0000785(QI)(GR+14)}$
Light Gas Truck	.41	.94	$FUEL=.906e^{(.0127+.00063P+.00699(5-GEAR)+.0000215(QI))S+.01234GR(P)GEAR}$
Light Diesel Truck	.14	.90	$FUEL=.1826e^{.0325S+.00208(GR)S+.0254P(GR+1)+.2333(5-GEAR)+.001405QI}$
Medium Truck	.19	.93	$FUEL=.583e^{(.02356+.000491(P)(GR+1))S+(.00594P+.01224GR)(6-GEAR)+.00057QI}$
Heavy Truck	.41	.96	$FUEL = (2.76/\sqrt{1+G})e^{(.00404+.0002169(P)(GR+1)+.0000282QI)\ S}$
Bus	.17	.92	$FUEL=.195e^{.0359S+.0044(GR)S+.0075(P)(GR+1)+.2781(6-GEAR)+.0002088(P)QI}$

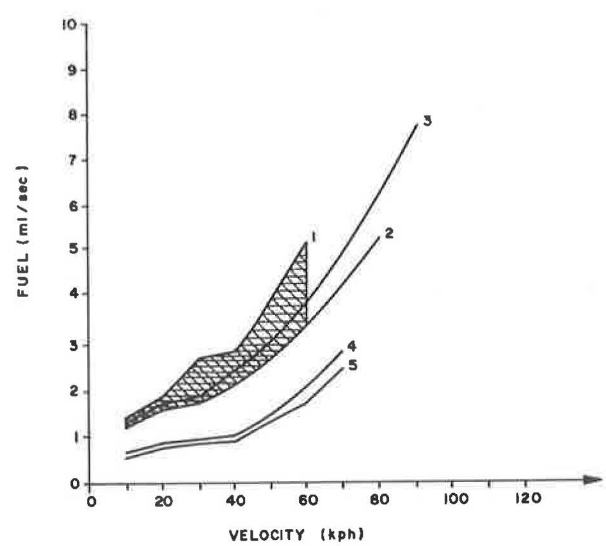
S = Speed (kph)
GR = Grade (%)
G = |GR| for negative grades, 0 otherwise
P = Weight of vehicle (metric tons)
QI = roughness

Figure 1. Fuel prediction for each vehicle type (load level 2, see Table 3, highest gear at each speed, zero grade, roughness 30 QI).



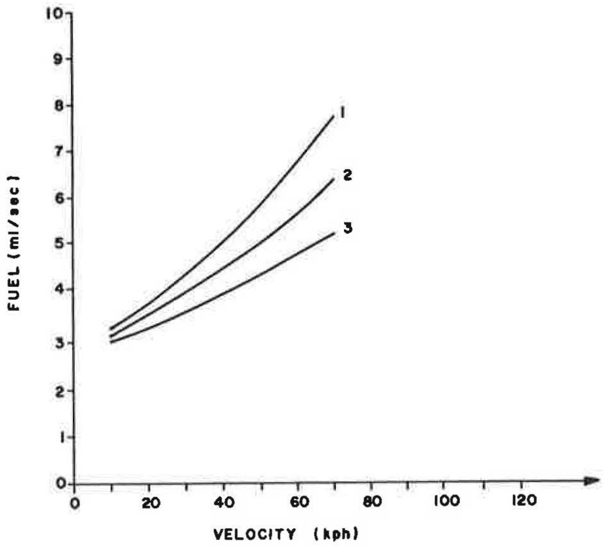
1. Articulated truck
2. Medium truck
3. Light gasoline truck
4. Bus
5. Light diesel truck
6. Utility vehicle
7. Car

Figure 2. Fuel prediction for light gasoline and diesel trucks (zero grade, roughness 30 QI). Shaded area shows the effect of gears.



1. Light gasoline truck, load level 1, lowest gear for each speed
2. Light gasoline truck, load level 1, highest gear for each speed
3. Light gasoline truck, load level 3, highest gear for each speed
4. Light diesel truck, load level 3, highest gear for each speed
5. Light diesel truck, load level 1, highest gear for each speed

Figure 3. Fuel predictions for articulated truck (zero grade, roughness 30 QI).



1. Load level 3
2. Load level 2
3. Load level 1

dent of roughness for a range of QI from 90-110 where QI values of rough paved roads overlap those of smooth gravel roads. The data from the medium diesel trucks were used to test the surface type effect and it was found to be not significant for α=0.1. Because QI values of less than 90 and greater than 110 usually represent paved and gravel roads respectively, and the surface type was nonsignificant from 90 to 110, the roughness variable explains the combined effect of surface type and roughness. Roughness was found to be significant for all vehicle types. Figure 4 shows the influence of roughness on fuel consumption for the utility vehicles.

Finally, Figure 5 shows the effect of grade on fuel consumption for the medium weight trucks. As expected, grades have a major influence on fuel consumption, and the grade-speed interactions is very significant. On steep negative grades, the fuel consumption is almost independent of speed, but for the 6% positive grade, increasing speed from 10 to 20 kph results in an increase in fuel consumption of more than 80%.

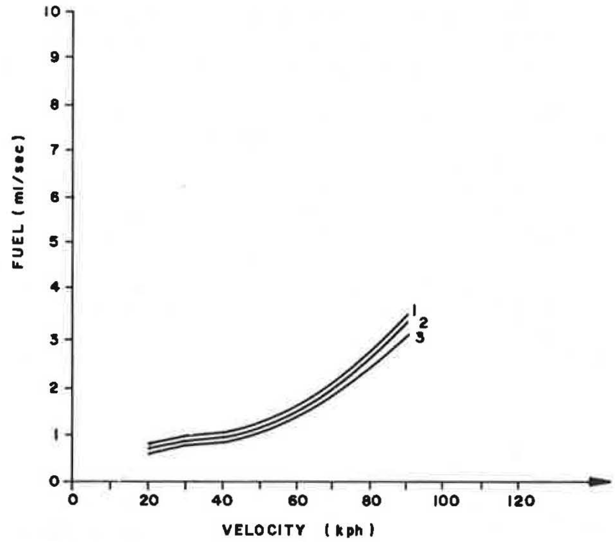
Horizontal Curvature Experiment

Two gravel sections with 8% grade were used to test the effect of horizontal curvature. Measurements made on a tangent section were compared to those from a section with a 72 m and a 101 m radius curve. For all vehicle types the effect of curvature was found to be nonsignificant at the α=0.1 level. A maximum increase of 3% on the negative curved section was observed with the utility vehicles.

Effect of Alcohol on Fuel Consumption

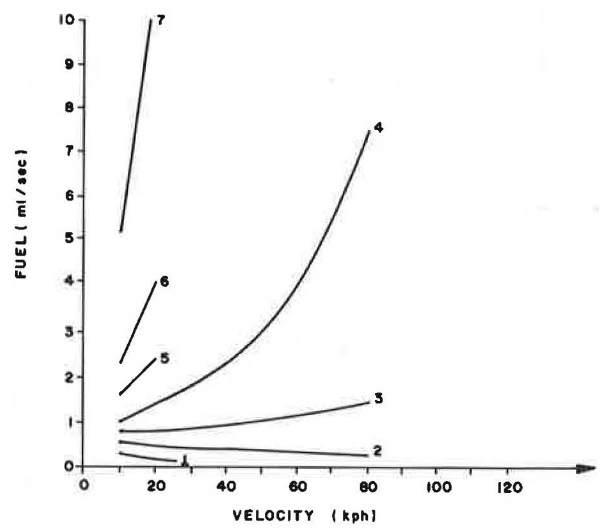
The data set analyzed for this experiment consisted of fuel consumption measurements of the four gasoline vehicles in the fleet operating on paved and unpaved test sections of 0,4, and 6% grade in the positive and negative direction. Data for the light gasoline truck on the 0% unpaved section was not collected because of

Figure 4. Fuel predictions for utility vehicles at three roughness levels (load level 2, highest gear for each speed, zero grade).



1. 150 QI
2. 100 QI
3. 30 QI

Figure 5. Fuel predictions for medium diesel trucks at various grade levels (load level 2, highest gear for each speed, roughness 30 QI).



1. - 13%
2. - 6%
3. - 3%
4. 0%
5. + 3%
6. + 6%
7. + 13%

weather conditions. Data were collected with 20% alcohol in the gasoline starting in August of 1978 when this fuel mixture became available to the public. These data were compared to data collected earlier during the constant speed experiment. Thus, results from this experiment may be confounded by vehicle age.

For all three gasoline vehicle types the mean consumption of fuel with 20% alcohol was significantly greater at α=.01 than the consumption of pure gasoline. The amount of increase in all cases dependend on the speed-gear combination used and therefore is probably dependent on the revolutions per minute of the vehicle motor. For the utility vehicle the increase was significantly different in the positive and negative directions.

The average increase for the car on paved sections was 6% for the combinations of speeds and gears tested. This value varied from 1% for 30 kph in third gear to 14% for 40 kph in second gear. The mean increase for the utility vehicle was 11% for the speed-gear combinations tested. A maximum value of 14% for 40 kph in third gear and a minimum of 2% for 20 kph in second gear were found. On positive grades the mean increase was 12% and on negative grades 7%. The light gasoline truck had a mean increase of 4%. The differences between the consumption of the two types of fuel was constant for the two surface types.

Summary and Conclusions

A major experimental program investigating the relationship between fuel consumption and roadway characteristics was described. The data collection effort is complete, and three experiments have been analyzed. These experiments demonstrate that at constant speed, vehicle type, weight, gear, grade, roughness, and mixing alcohol in gasoline, all have a significant effect on fuel consumption. Horizontal curvature was not found to be significant in a limited experiment. Surface type did not produce significantly different fuels for a fixed roughness and since roughness and surface type were highly correlated the continuous scale of roughness was used to explain the overall effect.

Regression equations are presented for calculating fuel consumption during constant speed operation. These equations, one for each vehicle type, show that fuel consumption is exponentially related to speed, weight, gear, grade, and roughness.

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PREDICTING TRAVEL TIME AND FUEL CONSUMPTION FOR VEHICLES ON LOW-VOLUME ROADS

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A major experimental investigation of the effect of roadway characteristics and environmental conditions on traffic behavior was conducted in Brazil. Empirical relationships were developed for predicting free speeds on positive and negative grades. Free speed was found to be related to vehicle class, grade, surface type, and road roughness. These relationships have been incorporated into a computer program, TAFA, for predicting travel time, free speed, and fuel consumption on low-volume rural roads. Example applications of the model show its usefulness in evaluating operational and road maintenance strategies which are of interest to highway engineers and economic analysts.

The Brazilian Government and the United Nations Development Program are sponsoring a research project on the Interrelationships between Highway Construction, Maintenance, and User Costs. In one study area of the project, vehicle fuel consumption and traffic behavior experiments were performed. Regression equations were developed and incorporated into a computer model for predicting travel time and fuel consumption of vehicles traversing rural roads. The activities undertaken in this study are shown in Figure 1. The traffic behavior experiments and time and fuel algorithm are addressed herein. The fuel experiments are described in detail in another paper (1).

Traffic Behavior Experiments

Table 1 summarizes the 11 experiments designed to investigate the relationships between speed and specific environmental and roadway conditions. Sections were sought which were homogeneous with respect to grade, surface type and roughness. Equal emphasis was given to each of these factors. Figure 2 shows the design matrix for the acceleration and free speed on positive and negative grades experiments. This factorial was expanded for the horizontal curve experiment to include four radius levels: 20-100 m, 101-200 m, 201-400 m, and greater than 600 m.

In the free speed calibration experiment smooth paved and unpaved sections with varying grades were sought. The two grade levels used were less than 2% and greater than 5%.

The data collected in the remaining experiments were restricted to a few sections limited to studying a specific aspect of driver behavior. For example, in the night experiment, observations were made on five sections.

General Methodology

Observations of the vehicle population were made at fixed stations on the sections. Speeds were measured to the nearest kilometer per hour with radar speed meters. Vehicles were classified by the scheme shown in Figure 3 and observers estimated loads as empty, half full, full, or undetermined for all buses, utilities, and trucks. In addition, during the deceleration, dust, and calibration experiments the license number was recorded and time was measured to the nearest one tenth of a second with digital stopwatches.

A test fleet was used in the acceleration experiment since acceleration rates of the vehicle population could not be observed without influencing traffic behavior. This fleet, described in (1), included an economy car, two utility vehicles, a light gasoline truck, a light diesel truck, two medium diesel trucks, an articulated diesel truck and a rural bus. Test runs were made from zero to maximum speed using full power and timing gear changes to obtain maximum acceleration. Time and distance were photographically recorded at fixed time intervals.

The grade, curvature, and cross slopes of each test section were determined by the project survey crew and roughness measurements were made with a Maysmeter (2). Rainfall, temperature, and wind speed and direction were recorded hourly on all sections. Daily measurements of looseness, moisture content of the loose material, and spacing and depth of corrugations were made on gravel sections.

Free Speed on Positive and Negative Grades

Data Collection. Initially nine mirror boxes were used at intervals of 167 m to define stations of observation for three radar teams. In November, 1976, a policy of strict enforcement of a nationwide 80 kph speed limit was instituted. After this date, exposed radar units affected speeds. Subsequently, observations were made with hidden radar units, without the aid of mirror boxes, and the distance between stations was increased to 500 m.

Figure 1. Activities for completing study and model of traffic behavior and fuel consumption.

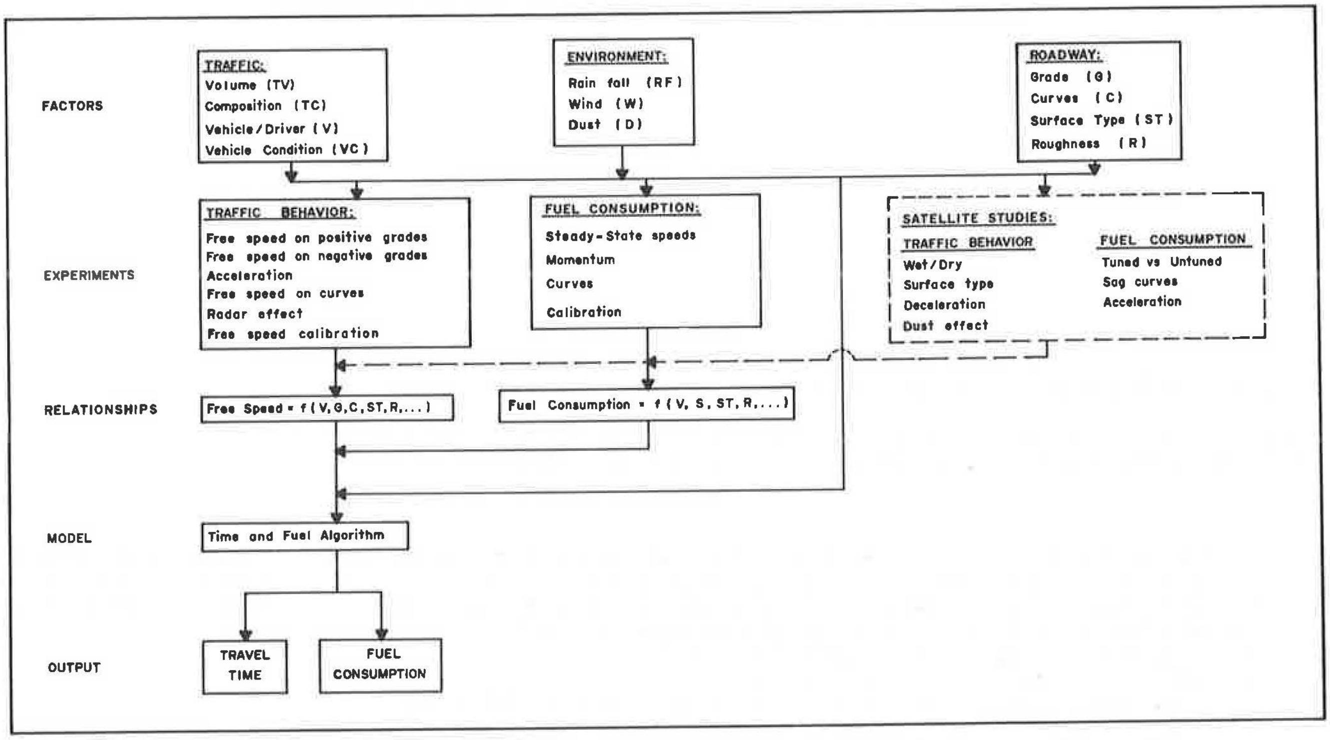


Table 1. Summary of the traffic behavior experiments

Experiment	Description
Free Speeds on Positive Grades	Determine the speed patern of vehicles climbing positive grades as a function of grade, position on the grade, surface type, and roughness. Observations made at 500m intervals on test sections 1 to 2 km long.
Free Speeds on Negative Grades	Same as above except observing traffic descending grades.
Free Speeds on Curves	Determine the speed distribution of vehicles traversing horizontal curves as a function of curve radius, grade, position on grade,surface type, and roughness. Spot speeds measured at the midpoints of curves
Acceleration	Determine the acceleration rate of vehicles climbing and descending grades as a function of vehicle type, load, grade, surface type, and roughness. A test fleet of vehicles was used to represent the vehicle population.
Free Speed Calibration	Collect independent data to verify the computer model. Spot speed and travel time collected on five sections, 3 to 5 km long.
Radar Effect	Determine if speeds are being affected by experiment methodology.
Wet/Dry	Measure the effect of rain on speeds. Curve test sections were used.
Deceleration	Determine the distribution of deceleration rates used by vehicles entering curves. Spot speed and time measured as vehicle starts to change speed, uses brakes, and enters curve. Observations made on paved roads with grades of 0, 2, and 3.6%.
Surface Types	Determine if the type of gravel surface affects speed. Observations made at the midpoint of long level tangent sections.
Dust	Collect data on the effect of dust on vehicle speeds and headways. Spot speed and travel time measured at three points on the test section. Dust measured by a photoelectric device.
Night	Determine the effect of darkness on speeds. Observations made on level tangent paved sections.

Figure 2. Design for acceleration and free speeds on positive and negative grades experiments.

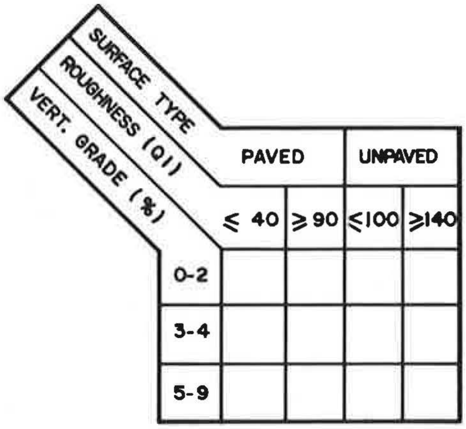
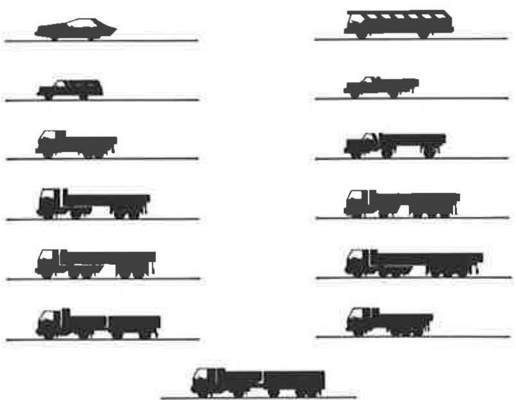


Figure 3. Vehicle classification scheme for traffic behavior experiments.



Test sections 2 km long were sought to provide sufficient length to observe the speed pattern on the grade. A minimum length of 1 km was accepted for some sections with grades greater than 4%.

Analysis of Free Speeds on Grades. The data for the trucks and buses obtained before the enforcement of the speed limit were not used in the analysis as they were significantly different from the speeds observed with the hidden radar. Over 100,000 observations remained for use in the analysis. Even with this quantity of data, it was necessary to group the vehicle classes shown in Figure 3 into cars, buses, utilities empty, utilities with load, trucks empty, and trucks with load for the analysis.

The mean speed per section, class, and station was the dependent variable analyzed. Unweighted analyses of variance, ANOVAS, were used to identified the important main effects and interactions. Examination of the data showed that no ANOVAS could be run to test surface type interactions. Thus, in the model, separate coefficients are estimated for the paved and unpaved sections.

The unweighted ANOVAS were only approximate tests since the number of observations per mean ranged from 1 to 1269. In the regression analysis, the data were weighted by the inverse variance of the mean. In all cases, the unweighted ANOVAS and the weighted regression analysis showed the same main effects and interactions to be significant.

The velocity at the last station was assumed to be the constant speed which would be maintained on a grade of infinite length. A regression equation was developed for this constant speed for both the positive and negative grade experiments. Data from the other stations were used to produce a speed change equation for each of the experiments.

Constant Speed on Positive Grades. The ANOVAS for the constant speed on positive grades showed that the class, grade, and roughness main effects, and the class-roughness interactions were significant for all sections. The class-grade interaction was also significant for the paved sections. The weighted regression analysis on 150 means produced equation 1 with $S_e = 4.1$.

$$S_p = ST_1(75.4 + 21.6V_1 + 13.0V_2 + 10.2V_3 + \sqrt{G}\,(-21.9 + 13.6V_1 + 15.2V_3 + 16.6V_4 + 12.5V_5) - R\,(0.0747 + 0.0675V_1)) + ST_2\,(95.9 + 38.1V_1 + 22.5V_3 + 18.1V_4 + 13.3V_5 - 25.4\sqrt{G} - R\,(0.108 + 0.072V_1 - 0.060V_2)) \qquad (1)$$

S_p is the estimated crawl speed in kilometers per hour and the other variables are defined in Table 2.

The equation shows that all classes react the same to roughness on paved roads except for cars for which the effect is almost doubled. The effect of grade on crawl speed for paved roads is shown in Figure 4. For all classes, as grade increases the crawl speed decreases, and the effect is greatest for trucks with load and buses. In comparing the two parts of the equation across surface type, one finds that at zero grade, the predicted speed is higher for unpaved roads than paved. The reason for this illogical conclusion is that the lowest grade analyzed for a gravel road was 2%, and thus predictions for unpaved grades less than 2% are extrapolations.

Speed Changes on Positive Grades. The ANOVAS for speed changes on positive grades showed the grade main effect was significant for all sections. The grade-class interaction was also significant for paved roads. The weighted regression on 622 means produced equation 2 with an $S_e = 4.05$.

$$\Delta S_p = GL\,(ST_1(-0.00534 + 0.00272V_1 + 0.00311V_3 + 0.00250V_4 + 0.00218V_5) - 0.000794ST_2) \qquad (2)$$

ΔS_p is the change in speed in kilometers per hour and all other variables are defined in Table 2. This equation is shown in Figure 5 for paved roads with a 6% grade and an arbitrary initial speed of 80 kph. On paved roads, trucks with load and buses decelerate at a higher rate than other classes. The deceleration rate on unpaved roads is constant for all classes and lower than on paved roads.

Constant Speed on Negative Grades. The ANOVAS for constant speed on negative grades indicated that class and roughness main effects were significant on all sections and that the grade main effect and grade-class interactions were also significant on paved sections. A weighted regression on 121

Table 2. Definition of variables in the free speed equations.

Variable	Value	Conditions or Units
ST_1	1	Paved roads
	0	Unpaved roads
ST_2	1	Unpaved roads
	0	Paved roads
V_1	1	Cars
	0	All other vehicle classes
V_2	1	Buses
	0	All other classes
V_3	1	Utilities empty
	0	All other classes
V_4	1	Utilities with load
	0	All other classes
V_5	1	Trucks empty
	0	All other classes
R	Roughness	QI (see Ref. (2))
G	Grade	Percent (100 rise/run)
L	Distance	Meters

Figure 4. Constant speeds on positive paved grades as a function of grade for a roughness of 30QI.

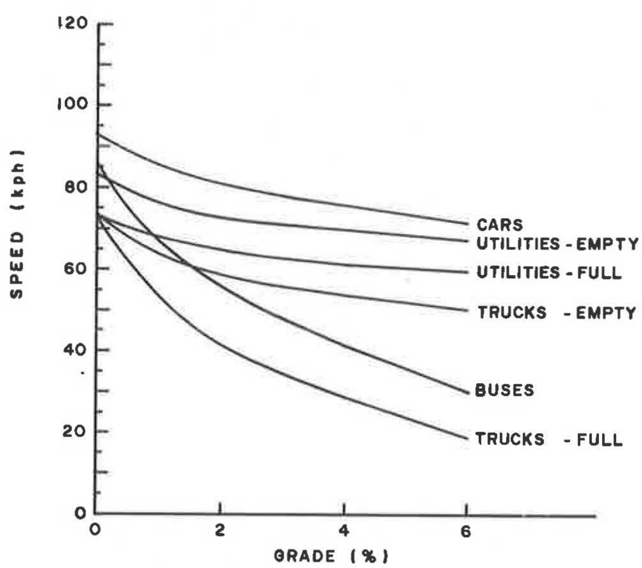
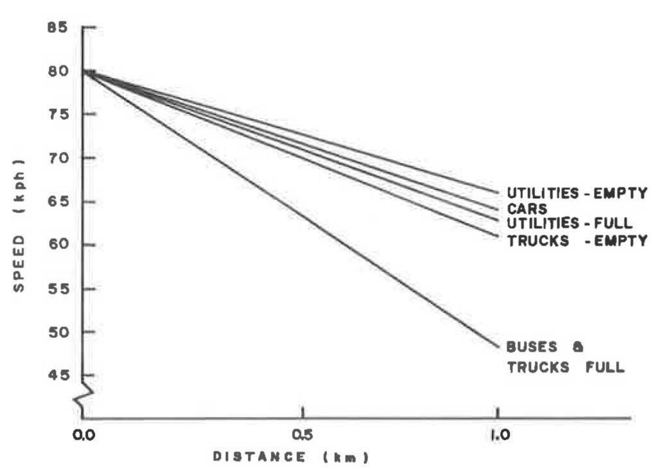


Figure 5. Speed change on positive grades.



mean speeds produced equation 3 with $S_e = 4.7$.

$$S_n = ST_1(76.2 + 17.4V_1 + 13.6V_2 + 5.6V_3 + 7.7V_4 - G\ (3.19 - 3.18V_1 - 3.71V_2 - 1.82V_3 - 2.86V_4) - 0.22R) + ST_2\ (79.8 + 7.8V_1 + 5.5V_3 - 0.25R) \qquad (3)$$

S_n is the desired speed on negative grades in kilometers per hour and the other variables are defined in Table 2. The speed for each class as a function of grade is shown in Figure 6 for roughnesses of 30 and 80 QI on paved and unpaved roads respectively. Grade has no effect on unpaved roads and the effect is minimal for cars on paved roads. Buses showed a tendency to go slightly slower on steep grades than on flat sections. Trucks, both empty and with load, show the greatest tendency to go fast on paved grades; they change from the slowest vehicles on flat grades to the fastest on 6% grades.

Speed Change on Negative Grades. The ANOVAS for speed change on negative grades showed that the grade effect and the grade-class interaction were significant for paved sections and indicated that vehicles do not change speed when descending unpaved grades. Equation 4 was produced from a weighted regression of 381 mean speeds with $S_e = 2.02$.

$$\Delta S_n = GLST_1\ (-0.00198 + 0.00120V_1 + 0.00166V_2 + 0.00134V_3 + 0.00157V_4) \qquad (4)$$

ΔS_n is the speed change of vehicles on negative paved grades and the other variables are defined in Table 2. Figure 7 shows that trucks use the greatest acceleration rate on paved negative grades, while buses and utilities with loads use the lowest acceleration rate.

Figure 6. Constant speeds on negative grades as a function of grade. Roughness equals 30 QI for paved and 80 QI for unpaved.

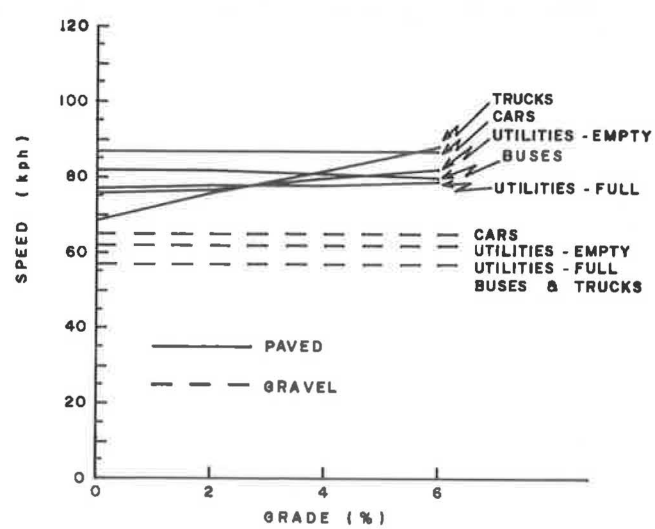
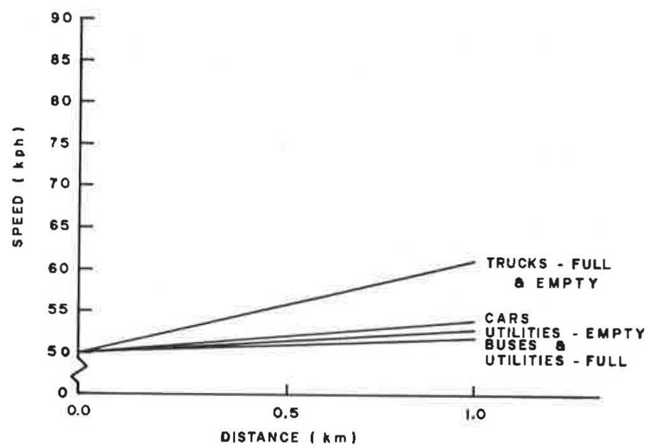


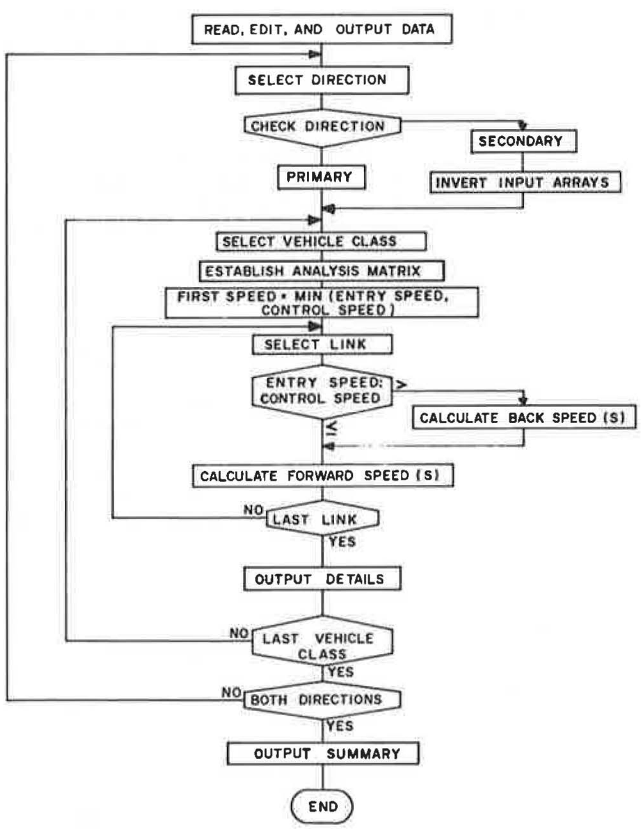
Figure 7. Speed change on paved negative grades.



Time and Fuel Algorithm

The relationships derived from the experimental program are incorporated in a computer model, TAFA, for predicting speeds, travel time, and fuel consumption for any low-volume rural road. The roadway is described to the model as a series of links which are homogeneous with respect to grade, horizontal curvature, surface type and roughness. Each vehicle class is sequentially processed along the road by the logic shown in Figure 8.

Figure 8. Logic used in the TAFA computer model.



Input to TAFA

The roadway characteristics are described in five arrays: grade, surface type, roughness, horizontal curves, and speed controls. The first three arrays are specified continuously along the road by giving the distances to points of change and the value of each characteristic as defined in Table 2. In the horizontal curve and speed control arrays the start and end distances are specified for each curve or zone along with the radius or control speed, respectively. The user can input the mean speed for a zone where exogenous influences control driver behavior.

The model can analyze six vehicle classes: cars, buses, utilities empty, utilities with load, trucks empty, and trucks with load. For each truck class the distribution of light gasoline, and light, medium, and heavy diesel truck types must be specified. Up to four power-to-weight ratios can be included for each class and type.

The Simulation Process

As shown in Figure 8, the first steps in the simulation process are to select the direction for analysis, and a vehicle class. The grade, horizontal curve, speed control, surface type, and roughness arrays are then combined to establish a matrix of homogeneous links. In preparing this matrix, a default speed control is assigned to each link where none is input. For links with curves, the minimum of the control speed for the link and the speed calculated for the curve is assigned to the speed control array.

The vehicle is entered onto the roadway with a speed equal to the minimum of the specified entry speed and the control speed for the first link. The desired speed for the link is calculated using the constant speed equation from either the positive or negative grade traffic behavior experiment. Due to a discontinuity in these equations the desired speed used for positive grades is the minimum of the speeds calculated with the positive grade equation and the negative grade equation evaluated at zero grade. The vehicle is then either accelerated or decelerated toward the desired speed. If the vehicle reaches the desired speed or control speed before the end of the link, then it is held at a constant speed for the rest of the link. The travel time and fuel consumption are then calculated.

The fuel consumption rate for the vehicle class is calculated as a weighted mean of the rate for each power-to-weight ratio. The weighted rate is then multiplied by travel time for the link to obtain the fuel consumption.

The vehicle is sequentially processed through each link, using the exit speed from one link as the entry speed to the next. However, if the entry speed to a link is greater than its control speed, the program back tracks along the road to find the point where deceleration must start in order to meet the speed constraint.

The logic is repeated until all of the links in one direction have been analyzed for the vehicle class. The speed, travel time, and fuel consumption for each link are printed if requested by the user. The next vehicle class is then simulated. When all classes have been processed the logic is repeated for the opposite direction if requested. Finally, speed, travel time, and fuel consumption for each vehicle class and direction are printed for the roadway.

Status of TAFA and Potential Uses

The logic for computing speed, travel time, and fuel consumption is complete and the model may be used for analysis if certain restrictions are understood by the user. Empirical relationships for free speed on positive and negative grades have been incorporated into the model, but assumed functions are used for other situations. For example, the rates given in the Transportation and Traffic Engineering Handbook (3) are used when a vehicle accelerates on a positive grade. The empirical relationships for fuel consumption at constant speed are used for all fuel calculations, but the estimates are modified when a change in speed is simulated.

To demonstrate some potential uses of the model, several example simulations have been run. The same roadway is analyzed in each example to demonstrate how operating and road maintenance policies affect travel time and fuel consumption.

The vehicle classes included in this analysis were cars, buses, and trucks with load. The trucks were divided into four types. The power-to-weight ratios and their distribution are given in Table 3.

The distribution of grades of the road used in the these examples is given in Table 4. The maximum grade on the road is 6% and the minimum radius of a curve is 100 m. The road has 44 km of gravel surface and 120 km of surface treatment.

Roughness of 30 and 80 QI were used for the paved and unpaved sections, respectively. Analyses were performed first assuming no speed control zones and then with speed controls of 80 and 50 kph on the paved and unpaved sections, respectively. The unpaved section of the road was then reanalyzed for roughnesses of 140 and 200 QI without speed control. The results of these five runs are shown in Table 5.

As would be expected, the inclusion of speed control zones had the greatest impact on cars. Their average speed dropped by 7% and this resulted in a fuel savings of 11%. The bus and truck classes showed only a slight reduction in overall speed. Fuel economy of the bus improved by 5.2%. The light gasoline truck economy increased by 4.0%, and the diesel truck economy increased by 6.7%.

Table 3. Vehicle characteristics used in the example problems.

Class	Type	Percent of class	Power to Weight Ratios (bhp/ton)					
			P/W	%*	P/W	%*	P/W	%*
Cars		100	41.4	70	35.0	30		
Buses		100	12.8	80	20.0	20		
Trucks	Lgt.gas.	20	26.4	70	50.3	30		
w/load	Lgt.dsl.	20	15.9	22	25.0	60	38.1	18
	Med.dsl.	50	6.3	50	15.1	50		
	Hvy.dsl.	10	6.1	90	17.0	5	22.0	5

* Percent of vehicles within the class-type combination with the specified P/W.

Table 4. Distribution of grades on the roadway used in the examples.

Grade Range							
	Max	+5	+3	0	-3	5	7
	Min	+7	+5	+3	0	-3	5
Percent of Route length		4	8	34	45	7	2

Table 5. Example problem results.

Situation	Cars Speed kph	Cars Gas km/l	Buses Speed kph	Buses Diesel km/l	Trucks w/load Speed kph	Trucks w/load Gas km/l	Trucks w/load Diesel km/l
Entire road No control speed	72	11.9	56	3.8	48	2.4	1.5
Entire road W/control speed	63	13.2	53	4.0	46	2.5	1.6
Unpaved section, 80QI	66	12.6	56	3.8	56	2.6	1.2
Unpaved section, 140QI	52	13.1	44	3.8	44	2.8	1.7
Unpaved section, 200QI	38	9.3	30	4.7	30	2.6	1.9

Road roughness showed a major influence on speeds as shown in Table 5. On the unpaved sections, equal speeds were predicted for the buses and trucks while the cars were estimated to have higher speeds. The fuel economy of the cars showed an increase as roughness changed from 80 to 140 QI, then decreased when the roughness went to 200 QI. The buses experienced the same fuel economy at 80 and 140 QI, but a substantial increase at the high roughness level. The light gasoline truck's fuel economy did not substantially change whereas the fuel economy of the diesel trucks kept improving as the vehicles slowed down due to roughness.

Even though the TAFA model is yet to be calibrated, these example problems do demonstrate the capability of the model to analyze the effect of different operating and road maintenance policies. Others areas where the model will be useful include the evaluation of geometric standards, and strategies for paving roads, economic analysis for route selection, etc.

Summary and Recommendations

An experimental investigation of the effect of roadway and environmental conditions on traffic behavior was presented along with empirical relationships for predicting free speeds on positive and negative grades. In these relationships, free speed is a function of vehicle class, grade, surface type, and roughness. These relationships have been incorporated into a computer program for predicting speed, travel time and fuel consumption on rural roads.

Further work is needed in this area to complete the analysis of all the experiments described, and incorporate the relationships into the computer model. Finally, the model needs to be validated using data which are already available for Brazil. Application of the model outside of Brazil will require the execution of calibration experiments to establish the validity of the model.

References

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