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PROCEEDINGS OF PAVING IN COLD AREAS MINI-WORKSHOP CANADA/JAPAN SCIENCE AND TECHNOLOGY CONSULTATIONS October, 1984

Pavement Division
Road Department
Chemistry Division
Geology and Chemistry Department
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Ministry of Construction

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by
Pavement Division
Chemistry Division

Synopsis:
This Technical Memorandum of PWRI is the proceedings of the second Mini-Workshop of "Paving in Cold Areas", which is one of the research subjects of Canada/Japan Science and Technology Consultations. These proceedings consist of 34 papers, 16 Canadian ones and 18 Japanese ones. The topics discussed at the Mini-Workshop are stated below.
Pavement Management
-Measurement of quality of service (the what and how)
-Automated data collection and analysis (the why and when)
-Criteria for quality of service for various classes of highway
-Timing and methods of preventive maintenance
-Predicting life cycles for various types or methods of pavement research
-Budget planning
Prevention and Cure of Cracking in New Pavement and Reflection Cracking
-Applications and effectiveness of asphalt additives such as polymers
-Various new techniques and materials
-Mix design and pavement structure design
-Theoretical analysis of reflection cracking
-Measurement of crack movement
-Timing and materials, methods and equipment for maintaining reflection cracking

Keywords:
Canada/Japan Science and Technology Consultations, Paving in Cold Areas, Reflection Cracking, Wearing, Maintenance and Repair, Fatigue Breaking, Cracking, Asphalt Pavement, Pavement Life, Evaluation of Performance, Rutting, Thermal Stress, Overlay, Concrete Pavement, Elastic Force, Improved Asphalt, Studded Tire, Freezing and Thawing Test

Foreword

The first Mini-Workshop of "Paving in Cold Areas", Canada/ Japan Science and Technology Consultations, was held in Vancouver, Canada on October 18-22, 1982. The second Mini-Workshop was held at the Public Works Research Institute, the Ministry of Construction, Tsukuba Science City, Japan on October 15-19, 1984, two years after the successful first Mini-Workshop. This Technical Memorandum of PWRI is the proceedings of the second Mini-Workshop and consists of 34 papers, 16 Canadian ones and 18 Japanese ones.

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A Comparative Analysis of the Canadian and Japanese Highway Environments

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ABSTRACT

In considering pavement design, it is essential that a number of factors be borne in mind depending on the pavement design procedure being used. Among these are: traffic loading (i.e. traffic volumes, axle load distributions), environmental conditions (precipitation, temperature ranges, freeze-thaw cycles, etc.), subgrade soil (type, moisture content, properties), available or proposed pavement structures (materials, costs, etc.), and other external constraints or defined design parameters (economic, minimum service life to first overlay, etc.).

The PICA program to date has very successfully concentrated on technical information exchanges relating to a broad spectrum of questions and subjects related to the unique problems of paving in cold areas. This paper outlines a systematic analysis of Canadian and Japanese highway environments with a view to pinpointing particular areas of similarity and difference.

The work is structured under the major comparative areas of climatic conditions, network size, economic environment and the design context. Comparisons are developed for specific parameters under each of these major subject areas. These are used as a basis for an analysis and discussion aimed at highlighting both hard and soft technology areas where future Canada/Japan efforts might best be pursued.

The paper is intended as a general disucssion which will focus on the parameters governing the paving technologies used in both countries. It is meant as a complement to the very specific subject area papers being presented in the PICA 2 workshop by focusing attention on areas where the greatest potential possibilities lie for mutual technology transfer in the future.

A COMPARATIVE ANALYSIS OF CANADIAN AND JAPANESE HIGHWAY ENVIRONMENTS

by

J. Robinson, T. Iijima, P. Eggleton

1.0 INTRODUCTION

Canada and Japan convened the first science and technology consultations in 1974 after it was felt by a number of people in both countries that there were real potential benefits to cooperation in specific science and technology areas.

The consultations took place every two years and covered a variety of areas of interest. At the fourth set of such consultations held in Tokyo in 1980, it was agreed that one topic where information exchange and collaborative activity would be of mutual benefit was paving in cold areas. As a first step in this activity, national contacts were established in each of the countries and reciprocal visits were made during 1980 and 1981. In October of 1982 a mini workshop was held in Vancouver under the ageis of the Canada/Japan Science and Technology Consultations. The PICA Workshop had three main thrusts. The first and dominant one involved an in-depth exchange of technical information. Flowing from this, the second element aimed at identifying needs and opportunities for further research and development in this area of concern. Finally, the third major aim was towards defining specific directions for further cooperative efforts between Canada and Japan on the problem of paving in cold areas in the future.

As a result of the presentations and round table discussions, four major subject areas were identified as topics of common interest and possible cooperative research:

1. additives
2. pavement management
3. low temperature effects
4. construction and design

In addition, the workshop participants agreed that a high priority should be assigned to problems in two major theme areas:

1. reflection cracking, causes, treatment and cure
2. pavement management criteria, the measurement of quality of service and appropriate criteria for use in decision making in pavement management

Following the PICA I workshop and the designation of more specific interest areas, discussions between Canada and Japan have resulted in PICA II, the second workshop in the series. In this workshop, particular attention has been given to concentrating discussion in the two major theme areas in hopes that an effective and productive exchange of information will take place. It is also hoped that in this instance specific cooperative activities and projects might be identified for the future.

Efforts to date have focused on the exchange of general technological information relating to paving in cold areas. In order to help determine what areas are most suitable for continued exchange in efforts between Canada and Japan, it is educational and useful to compare the environments in which highways are being constructed in both countries. Such a comparison should help to focus attention on particular areas which may be most suitable for future cooperative efforts.

The purpose of this paper is to carry out an examination of the comparative highway environments in each of our countries in the context of the problem of paving in cold areas. This comparison, which forms the body of this paper, looks at a number of the key factors affecting pavement design in our countries, including:

1. climatic conditions
2. existing road networks
3. the economic environment
4. the design context

The paper attempts to pinpoint particular areas of similarity and difference. The work is structured under the major comparative areas outlined

above and this analysis is used as a basis for discussion aimed at highlighting both hard and soft technology areas where future Canada/Japan efforts might best be pursued.

2.0 CLIMATIC CONDITIONS

2.1 General

Canada and Japan are countries of marked contrasts and similarities. Japan, having a land area of some 370 000 km^2, is oriented primarily north-south and ranges from latitude 28°N to LAT 46°N. Canada, on the other hand, has a total area of almost 10 000 000 km^2, the second largest country in the world. In width, it spans six time zones and, in the north-south direction, ranges from about LAT 45°N to the north geographic pole. In spite of this huge land mass, over two-thirds of all Canadian live within 160 km of the southern border and, as a result, it is here also that the greatest density of road and highway development is found.

2.2 Temperature

In comparing the climatic conditions in the two countries, it becomes evident that the southern part of Canada and the northern portion of Japan are quite similar. Table 1 shows a comparison of a number of key temperature indicators in the two countries.

As can be seen from the table, in some cases, directly comparable measures were not available in both countries. The similarities between Hokkaido and the southern parts of Canada, however, are evident. February temperatures on the northern island compare well with those found in south-central Ontario and the Atlantic provinces. In addition, conditions in the interior of Hokkaido are similar to those found in the province of Quebec at the same time of year.

Unfortunately, other data shown in the table, while indicative of typical conditions in the two countries, cannot be directly compared. In many parts of Canada, however, it seems that winter temperature conditions are

more severe and enduring than in Japan. However, as noted above, the southern Ontario and Atlantic regions do appear quite comparable to those found in Hokkaido.

2.3 Precipitation

Overall precipitation levels (see Table 2) are considerably higher in Japan than in Canada with the country's average being over 1 000 mm and with some areas experiencing over 3 000 mm of rainfall. The latter case occurs primarily in the southern parts of Japan and along the Japan Sea coast. Annual precipitation levels in Hokkaido (rain + rain equivalent snow) range between 1 000 and 1 500 mm annually.

The Hokkaido levels compare well with those found in Canada. Total annual precipitation (rain + rain equivalent snow) in the latter case vary between 350 to 525 mm in the prairies, through 800 to 1 200 mm in central Canada, up to 1 500 mm on the east coast.

Average annual snowfalls in Canada (see Table 3) vary quite widely across the country with Vancouver getting only around 60 cm of snow annually, the prairies and central Canada about 110 to 130 cm and the east coast over 300 cm.

Table 3, in addition to showing Canadian snowfall statistics, also offers a comparison of the average number of days annually, on which snow falls. In Japan, these values range from 10 to 50 in the southern and central parts of the country to around 100 on Hokkaido Island. Canadian figures are very similar. The lowest values are found in the west (i.e. Vancouver with only 10 days) with the rest of the country varying between 41 to 88 days. While this statistic is not necessarily a reflection of annual snowfall depths, it does serve as an indicator of frequency of winter maintenance activity required.

3.0 EXISTING ROAD NETWORKS

3.1 General

A direct comparison of the existing road networks in Canada and Japan is difficult, simply because of the different statistics and classifications used in each country. Summary tables of each country's networks are presented in Tables 4 and 5. Discussion of the tables is included in this section.

Roads in Japan are classified administratively into national, prefectural, municipal roads and expressways. The expressway system is administered by a special organization, the Japan Highway Public Corporation. All principal cities in the country are connected by the national highway network. These national highways are divided according to their importance into designated and undesignated routes. The former are administered directly by the Ministry of Construction and the latter are left to the prefectures to administer under budgets afforded to them by the Ministry.

In Canada, a corresponding administrative classification divides the highway system into federal, provincial and municipal roads. Constitutionally, the power to administer the highway system rests with the provinces, and the federal portion of the network is limited in size and generally restricted to National Parks and other federally controlled facilities. Municipalities usually administer their own roads within the context of a global municipal budget which is approved by the provincial government. Within the administrative classifications, each jurisdiction uses a functional classification of roads divided into freeways, arterials, collectors and local streets.

3.2 Comparison and Discussion

Table 6 offers a summary comparison of road networks in Canada and Japan. It is interesting to note that Japan's highway system is almost 20% longer than Canada's, measuring just over 1 100 000 km versus 928 000 km for the latter.

If the federal and provincial systems are grouped together and compared to the national, prefectural and expressway systems in Japan, it can be seen that about 29% of Canada's roads fall into this grouping (for this paper called class 1 roads), whereas only about 16% of Japanese facilities do so. The remaining 71% (in Canada) and 84% (in Japan) of roads are the responsibility of municipalities to administer (class 2 roads). This has obvious consequences for municipal budgets which in most countries tend to have the least amount of flexibility built into them. Effective management and administration of road funds at this level is therefore an extremely important issue.

Table 7 presents a comparison of paved versus unpaved roads in the two countries. No distinction is made regarding the type of pavement. Unfortunately, in Canada, detailed statistics are not available at the municipal level and, thus, figures are not presented for the Canadian case in this category. Of the Class 1 roads, over 141 000 km or 53% of the network is paved in Canada. In Japan, the percentage is much higher and almost 87% of this class is paved. Within Canada, there is a wide variance in this figure by region, ranging from 6% in the Northwest Territories to over 93% in Ontario.

Of Japan's Class 2 roads, 416 547 km are paved, and this represents almost 44% of the municipal roads in that country. In all, in Japan, over 50% of all roads are paved.

4.0 ECONOMIC ENVIRONMENT

The economic environment in most industrialized countries has deteriorated somewhat in the last decade. Rising energy costs and increasing interest rates have coupled to force most governments to reexamine national spending priorities. In many cases, as a result of these examinations, the relative share of government expenditures devoted to the roads sector has decreased steadily even if it has increased in absolute terms.

While it is generally acknowledged that the age of rapid growth in capital investment in our highway systems is now over, engineers are faced with

devoting more significant levels of effort to maintaining the extensive infrastructure that has been put in place.

The economic climate will dictate to a great extent what can or cannot be done with existing road networks. Great care will have to be exercised in the management of maintenance and in the development of economical rehabilitation and maintenance techniques in order to get the most out of every dollar invested. An examination of the economic environment of the roadway system in Canada and Japan is helpful in pinpointing areas of common concern.

4.1 Motor Vehicles in Service

Since 1945, motor vehicle registrations have grown steadily in both Canada and Japan. Table 8 illustrates this fact.

In 1970, for instance, some 8 million motor vehicles were registered in Canada with about 81% of these being passenger cars and 19% being other vehicles (i.e. trucks, buses, etc...). By 1982, total vehicle registrations in this country had grown to almost 14 million and the automobile proportion had dropped slightly to 78% of the total with trucks, etc. increasing to around 22% of all registered motor vehicles.

The 1970 total for Japan was almost double that of Canada, totalling close to 16 million vehicles. Interestingly, at that time, passenger cars occupied a minority position in that country, representing just over 45% of the total vehicle registrations with commercial vehicles holding the remaining 55%. In 1980, both the total and proportion had changed significantly, demonstrating a substantial leap in private car ownership in the intervening decade. Total vehicle registrations had more than doubled to over 36 million with car registrations increasing almost threefold. At this time, car registrations represented 63% of the total and other vehicles the remaining 37%.

If the "number of registered motor vehicles per kilometre of highway" is indicative to some degree of the use of the system, a comparison of this

index between the two countries may give some idea of the relative intensity of roadway use in Canada and Japan.

In Canada, the national average for this number is 15 motor vehicles per km (mv/km), ranging from a low of 3.8 in Saskatchewan to a high of 30.7 in the province of Quebec. Japan, on the other hand, has a national average of 32 mv/km. No data was available on regional variations for this figure within the country. While the figures mean little in absolute terms, the fact that Japan's national average is almost double that of Canada (32 vs 15 mv/km) would appear to indicate a much higher overall average intensity of use in Japan. Under such conditions, one would expect to see a shorter design life used in that country, and, as will be discussed later in this paper, this is the case.

From a pavement design standpoint, the presence of heavy commercial vehicles within the traffic stream has a critical influence on the life of a road. Comparisons of percentages of commercial vehicles in specific locations are relatively meaningless on a national level. The importance of the roadway system as an element in moving freight within the country, however, is probably a good indicator of the presence of heavy vehicles that allows for comparisons between the two countries.

Table 9 summarizes freight modal splits in the two countries in 1980. It should be noted that the Canadian percentage represents only the for-hire trucking industry. No statistics are available on the share carried by private motor truck carriers.

In percentage terms, Japan's trucking industry seems to have almost double the market share that Canada's does. This is undoubtedly not the case, however, since, as noted above, the Canadian figures do not include the share carried by private carriers. Despite this fact, there is little doubt that the Japanese trucking industry does have a more important role to play in its country than the Canadian industry does in Canada.

If volumes of freight carried are compared for 1980, Japanese truckers moved some 178 billion ton-km of freight that year. The for-hire trucking industry in Canada accounted for about 42 billion ton-km of freight move-

ment in the same year. If high-range estimates of the volume of private carrier movements in Canada are included, the Canadian total truck-freight movements jump to about 80 billion ton-km. Even at this figure, the Japanese trucking industry is still moving about 2.25 times as much freight (in ton-km) as the Canadian one, over a road system that is only about 20% longer than Canada's. The intensity of heavy-truck use in Japan would seem to be considerably higher than in Canada, with obvious consequences on expected design-life.

An historical analysis of trends within the two countries, which is not detailed in this paper, shows that over the past decade in Japan, there has been a steady increase in the share of freight traffic carried by road, at the expense of the rail market share. In Canada, the market shares seem relatively more stable since the early 1970s with trucks having gained only about 6% in market share (based on tons carried) since that time. Recent research initiatives in Canada, aimed at examining current weights and dimensions regulations for the trucking industry, may result in some new long-term shifts in market share trends, but this question has not yet been investigated in my detail. There is no doubt, however, that these research efforts will result in an important gain in the understanding of the impacts of heavy vehicles on flexible pavements.

4.2 Government Expenditures

Trends in government spending have, and will continue to influence the development and maintenance of our highway systems. Pressures for government dollars from other program areas have forced highway administrators to better justify proposed expenditures and to ensure that investments are made in areas which show the best potential return. This is appropriate and has led to the development of management systems that permit this type of evaluation to be carried out. There is little doubt that even more emphasis will have to be put on the most effective management of resources if we are to continue to operate efficiently in today's constrained economic environment. A brief comparison of expenditure trends in Canada and Japan is instructive in this respect.

Japanese expenditure trends on highways since the mid-seventies are illustrated in Figure 1. These figures include expenditures on the construction and maintenance of bridges. Growth rates for capital and maintenance expenditures increased at annual rates of 20 to 30% until about 1978. At that time, the rate of increases slowed down and, since 1981, has been at zero or slightly negative. In 1981, expenditures had reached over the 3 000 billion yen (15.8 billion dollars) mark.

Canadian expenditures on highways between 1945 and 1979 have totalled some 11 400 billion yen (60 billion dollars). In 1979 the total yearly expenditures by all government levels had reached about 1 000 billion yen (5.5 billion dollars). The context of these highway expenditures in the overall government spending paterns of the past 20 years is more worrisome in Canada. Figure 2 illustrates these and shows the declining importance of the transportation and communications share of overall budgets. Where in 1955, almost 13% of all government expenditures were directed to transportation and communications, by 1980 this figure declined to 8.4%. This serves to illustrate the increasingly fierce competition between various government program areas for shares of the tax dollar.

Whereas the above percentages cited dollars spent on all transportation modes, a direct parallel can be drawn between Canada and Japan if highway expenditures as a percentage of government spending are examined. This comparison is illustrated in Table 10. The same information is summarized more graphically in Figure 3. Data from the United States is also included for comparison purposes. The similarity in trends in expenditure patterns over the past 15 to 20 years in the three countries is startling. All have seen a steady decline in the importance of the share of highway expenditures as a percentage of total budgets. Given the maturing state of the countries' networks, this may be appropriate. However, bearing in mind recent developments in the U.S.A regarding the condition of their infrastructure and the problems which now must be faced in rebuilding it, the similarities and possible consequences cannot be ignored.

Efficient maintenance management systems will have to be developed in the future as road networks in both Canada and Japan mature and approach design-life limits. In addition, the technology of both maintenance and

rehabilitation will need to be further improved to render such work as cost-effective as possible.

5.0 THE DESIGN CONTEXT

Differences in design procedures are not as much a part of the design context as a product of it. As such, they are not discussed extensively in this paper, but are simply outlined in sketch form in the conclusion of this section. It is interesting to note that a great many flexible pavement design procedures find at least a part of their basis in the landmark AASHO road test carried out in the U.S. in the 1950s. In most instances, particular techniques have been adapted and refined over the years, bearing in mind local conditions, materials, expertise and other factors. Thus, a variety of approaches exist, including more recently developed theory-based methods using calculated stresses and strains. This section of the paper concentrates on a discussion of relative wheel loads and related concepts used in Canada and Japan under current regulations.

5.1 Axle Loads and Gross Vehicle Weights

The loading regime to which a road is subject obviously is one of the most important factors affecting its design and ultimate longevity. All industrialized countries of the world regulate the trucking industry from a permissible load standpoint. Although approaches differ from country to country, generally, regulations include provisions for three key elements:

- maximum gross vehicle weight
- maximum permissible axle (or wheel) loads
- configuration and dimensions (i.e. lengths, widths, axle spacings, etc.)

For the first two of these, a greatly simplified comparison of the Canadian and Japanese cases is presented in Table 11. As can be seen, maximum permissible axle loads are very similar in both the single and tandem axle cases. The gross vehicle weight (GVW) regulations, as currently written, show a substantial difference. It appears as though Canada's maximum

(which only occurs in some provinces) is almost 50% again as high as Japan's. In Japan, however, a program of special permits does allow a good degree of well-controlled flexibility in respect of GVW regulations, which sees the regulatory maximum exceeded under government permission.

It must also be emphasized that Table 10 greatly simplifies the Canadian regulatory context which varies quite widely from province to province in that country. The regulations regarding configurations and permissible GVWs and axle loads are particularly complex.

The Canadian case is also currently the subject of an extensive in-depth research program which will probably see some changes in regulations in four or five years. The question of axle spacings and relative pavement damage is one which is raising particular interest.

5.2 Design Concepts and Techniques

Pavement design concepts and techniques used in Canada and Japan have been well documented in the literature as well as being thoroughly described in Japan's case in the "Manual for Design and Construction of Asphalt Pavement" produced by the Japan Road Association and for Canada in the "Pavement Management Guide" produced by the Roads and Transportation Association of Canada. Some key points are summarized below.

In Japan, for thickness calculations, the design life of Asphalt Pavements is set at 10 years and the design principle assumes a certain cumulative number of 5-ton wheel loads (10-ton equivalent axle loads - EAL) during that lifetime. The determination of EAL is carried out using either a table look-up procedure (see Table 12) or a more complex "formula" approach where particularly large volumes of heavy vehicles or heavy wheel loads are expected to occur.

The table look-up procedure uses a five-group volume classification system which was developed from a series of observations carried out on Japanese highways. The one-way daily traffic volume of heavy vehicles in five years is estimated to determine the volume classification into which the roadway falls.

In addition to the road classification, the California Bearing Ratio (CBR) of the supporting subgrade is also taken into account. For given CBR values (evaluated using JIS A 1211 - similar to ASTM D 1883), target values of TA (pavement thickness which would be required if the entire depth of the pavement were to be constructed using hot asphalt mixes used for binder and surface courses) and H (target value of the total thickness) are obtained (see Table 13). An iterative process is followed and the thicknesses of the various binder and surface courses, base and sub-base layers, are determined using relative strength coefficients and the formula:

$$TA = a.T1 + a2T2.....+ anTn$$

where a, a2....., an: Coefficients of relative strength
and T, T2....., Tn: Thickness of individual layers in cm.

There are, of course, variations on this process and special approaches used in certain special conditions. These are not discussed here. Figure 4 illustrates, in flow chart form, Japan's thickness design procedure for flexible pavements.

Figure 5 shows typical pavement cross sections used on a number of Japanese expressways.

Canadian thickness design methods can be broken down into three categories:

1) experience-based methods;
2) empirical methods;
3) theory-based methods.

Table 14, extracted from RTAC's Pavement Management Guide, summarizes the characteristics of these methods.

The Canadian methods generally consider, as an integral part of the methodology, problems of frost heave, differential frost heave, spring strength loss and other climate effects. Supplementary measures are also

often taken in respect of these specific problems, but these are not discussed in this paper.

All of the above methods have some elements in common. A design life is chosen (usually 15-20 years) and thicknesses of the constituent layers of the pavement selected in consideration of the anticipated traffic on this system over that period and in consideration of the materials to be used in the constituent layers. Expected traffic on the system is based on a procedure whereby all axle loadings are related to a common base, the 80 kn (18 000 lb) equivalent single axle load (ESAL).

Flow charts illustrating an empirical approach and the Ontario method of thickness design for flexible pavements are shown in Figures 5(a) and 5(b) respectively.

Some examples of typical pavement thicknesses used in various provinces are shown in Table 15.

6.0 DISCUSSION AND CONCLUSIONS

6.1 Discussion

The foregoing has attempted to outline some key comparisons between Canadian and Japanese highway environments. It has done so in a broader context than just paving in cold areas and has deliberately avoided discussion of current design practices in the two countries in any detail. The latter is a subject area that will be discussed in other papers at the PICA workshop.

Each of the environmental parameters discussed (i.e. climatic, economics, etc.) gives rise to a set of technical issues.

Table 16 provides a synthesis of these generalized technical issues and attempts to identify their relative importance in the overall context of paving in cold areas. A "transferability index" (High, Medium or Low) is

included for each issue. The index is subjective and is founded on similarities in, and/or differences, between the two countries.

In addition, an "intensity index" is listed for both Canada and Japan in an attempt to denote the relative importance of these issues within the two countries. Each issue is identified as design, maintenance or management related or some combination thereof. Finally, a score is developed for each category based on the transferability and intensity indexes by assigning points as follows: H=3, M=2, L=1; and adding the three values to give the total score.

The exercise is simplistic and somewhat subjective but does serve to highlight what seem to be the priority areas for more detailed attention. The issues related to snow and ice control, while important in their own contexts in each country, do not appear to offer a great deal of promise for cooperative future efforts. This is primarily because the approaches used, particularly for ice control in Canada and Japan, are substantially different. In Canada, ice problems are addressed primarily through the use of salt as a de-icing agent on highways. While effective, major social issues relating to pollution and corrosion of vehicles have opened this technique to an intense scrutiny.

In Japan, on the other hand, the safety problems presented by road icing are addressed through the extensive use of studded tires. Salt is prohibitively expensive and is not widely used for both that reason and the possible environmental repercussions. The use of studded tires by over 90% of the vehicles has created a serious pavement wear problem which has no current parallel in Canada.

So, while both countries have problems in addressing the safety-related problem of road icing, the approaches used to address it are different and offer little possibility for mutually beneficial cooperative research projects in the future.

The remaining issues in the Table can be thought of as falling into two main groups: those that are directly related to cold area paving problems and practice, and those which are related to paved roads in general.

The former, which include temperature-related design problems (i.e. low temperature cracking, freezing soils, etc.), are generic scientific questions. To solve these requires research into the basic nature and behaviour of road construction materials under certain conditions. Information from these types of research will be transferable between countries, where the materials and conditions being explored are similar. In many cases, however, while the knowledge about materials behaviour can be transferred, solutions to problems presented by that behaviour may differ depending on the cost and availability of materials, etc. This applies equally to design-related problems and repair-related questions. Proposals for co-operative international research and/or exchanges should be judged keeping this fact in mind. Comparisons of Canadian and Japanese climatic conditions show close similarities. Thus, an opportunity exists for continuing joint efforts and information sharing on temperature-related design problems. The efforts should be tightly focused, however, with a view to increasing the base of knowledge available to both parties from which applied solutions can be developed.

In the larger context of road paving, there is little doubt that two issues are currently of great concern, not only for Canada and Japan, but for other countries as well. These are:

1) the need for more effective management systems;
2) the question of the impact on pavement life of heavy vehicle loads.

The comparisons presented previously in this paper have highlighted the size of the paved road networks in Canada and Japan. As these extensive systems mature, the need for more and more investment in maintenance, restoration and rehabilitation increases.

The budget analyses presented in this paper indicate that both Canadian and Japanese highway engineers are beginning to feel the economic pinch. Budgets are decreasing in real terms while needs for maintenance dollars are only beginning to increase substantially as the network ages. In addition, traffic volumes continue to increase and, in both countries, the

trucking industry plays a major role as a goods mover. These latter trends have a serious impact on the design life of highways.

As a result of the above, road administrators must become more efficient with the allocation of their budget dollar. In the highway field this means more effective management tools. For the paving area in particular, accessible and useful pavement management systems are essential. Such systems require significant amounts of data, however, and their usefulness is directly proportional to the availability of that information.

In great part, these data are obtained manually (i.e. spot measurements, physical observation, etc.), a fact which limits the amount of material that can be collected and also affects the consistency of data quality. The implementation of a true PMS requires extensive sampling which, for many agencies, is simply not possible with the resources they have available. Automated data collection devices for PM systems greatly facilitate both the collection and analysis of data and, through that, the practicality of pavement management systems.

While much work has been done in this area, a great deal remains to be accomplished. In essence, data for PMS can be classified into two main types:

1) structural strength information;
2) surface conditions data.

On the pavement strength side, the Benklemann Beam test, Falling Weight Deflectomer, Dynaflect, etc. are all pieces of equipment aimed at providing a measure of the structural strength of an existing pavement. While most existing strength-testing technologies are capable only of spot samples, which can be taken with varying degrees of speed and efficiency, some machines have been developed to take continuous measurements or repeated spot samples while in motion. The best known of these is the French "Déflectograph". This, however, is still a low-speed device.

Research is currently underway in Canada on a high-speed pavement strength-testing vehicle which could take continuous measurements at close

to the operating speed of traffic. The benefits of such a device, in terms of efficiency of measurement, minimization of delay to traffic and decreased safety risk, are self evident. This high-risk, high-benefit research area would appear to be a prime candidate for cooperative efforts in the future.

In addition to the structural problem, surface conditions are also a major concern in PM systems. Surface conditions include such factors as road roughness, skid resistance, rutting, pavement reflectivity and various forms of pavement distress.

Once again, much automated equipment for measuring certain aspects of surface conditions already exists. However, norms for procedures, applicability, analysis and measurement vary widely and many elements, particularly the evaluation of pavement distress, must still be done using large amounts of human input. Great scope exists here for research efforts to rationalize and perfect not only the measurement tools themselves, but the standardized measurement statistics to be used in the associated PM systems.

The question of the damage caused to pavement by heavy vehicles is one which has aroused a lot of interest in the United States, Canada, Australia, Japan and many European countries. In cold climates in particular, the variability of subgrade bearing capacity during the year is still a major problem. Many questions have been posed in respect of these issues; however, few satisfactory answers have been received.

6.2 Conclusions

This discussion has not attempted to identify specific projects which might be candidates for future activity. Rather, it has looked at a number of contextual parameters and tried to identify general areas of similarity where mutual cooperative activities might be of some benefit. While the analysis is admittedly simplistic, a number of useful conclusions can be drawn from this exercise:

1) The highway environments in general are quite similar in particular parts of the two countries. This is true from a climatic point of view and an economic one. While design techniques, solutions, etc. are different, the contexts themselves are not.

2) Problems relating to cold weather paving technology do offer a good opportunity for possible future activities. To be of maximum benefit, however, a more focused effort is required, centering on very specific problem areas. To be of maximum mutual benefit, the results of such work must be highly transferable and should probably center on the basic nature and behaviour of road construction materials under specific conditions. Where applied research is undertaken, the context of that work must be similar in both countries.

3) A broader area of problems has emerged relating to paving in general rather than just in cold regions. This seems to be an area worth pursuing in the future.

4) In particular, given today's economic conditions, Pavement Management Systems (PMS) seem to be a vital tool for the road administrator. General areas where research relevant to this topic should be pursued include:

- automated data collection systems for both structural and condition rating problems;
- development of standardized measurement statistics for PMS;
- development of standardized equipment for collection of data required to develop/calculate standardized measurement statistics;
- refinement and development of PMS incorporating the above.

5) Research information on heavy vehicle pavement impacts is required. Many countries in the world are presently researching this area. A pro-active program of information exchange in this area is essential.

Cooperative international projects are at best difficult and often impossible to arrange. Such cooperative efforts, especially in formative stages, must be rather general in nature, centering on overall information exchange rather than very specific focused efforts. This has been the way to date in the PICA series of workshops. It is mutually beneficial and worth continuing.

As discussions take place, however, certain specific issues come to light, and it is in the elaboration of these issues, and subsequent work on them, that significant gains can be made by both parties.

The OECD Road Transport Research Steering Committee, through the projects which it carries out, provides a good model for a second step in cooperative activities. By defining a specific area requiring attention, countries create a committee of experts to work on very well-defined problem areas. Carrying out their work over 18 months and holding three to four meetings, the committees produce high-quality and useful results. Such a structure, applied to a specific problem area of concern to Canada and Japan, and consisting of a small group of experts from each country, might be a logical new addition to our ongoing general information exchange program. It is worth considering.

The highway infrastructure in both Japan and Canada plays a vital role in ensuring that our countries remain economically and socially healthy. The pressures for cutbacks in the future will be severe. The problems will be numerous. Ours is the challenge to respond to those needs in a responsible and technically sound manner. The sharing of information, experiences and problems between our countries will help ensure that we do so effectively.

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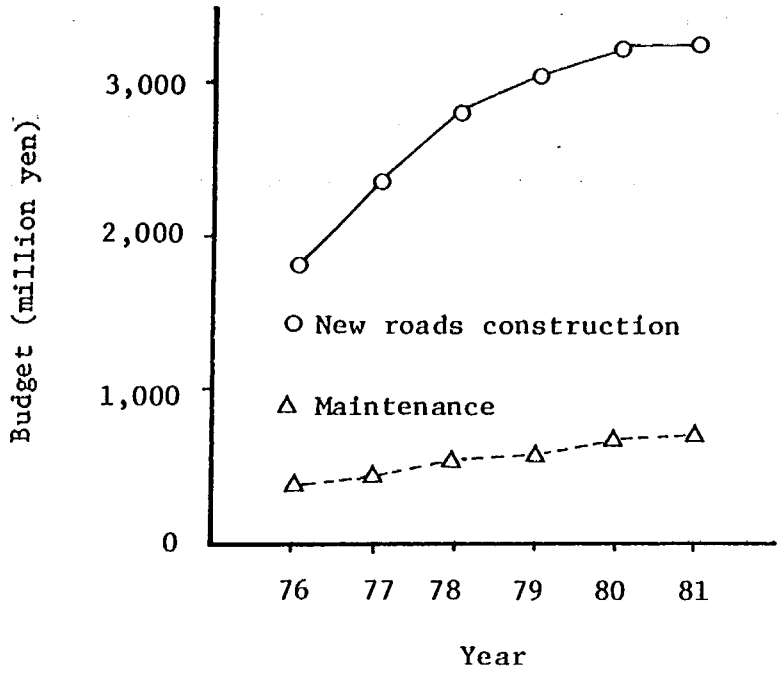
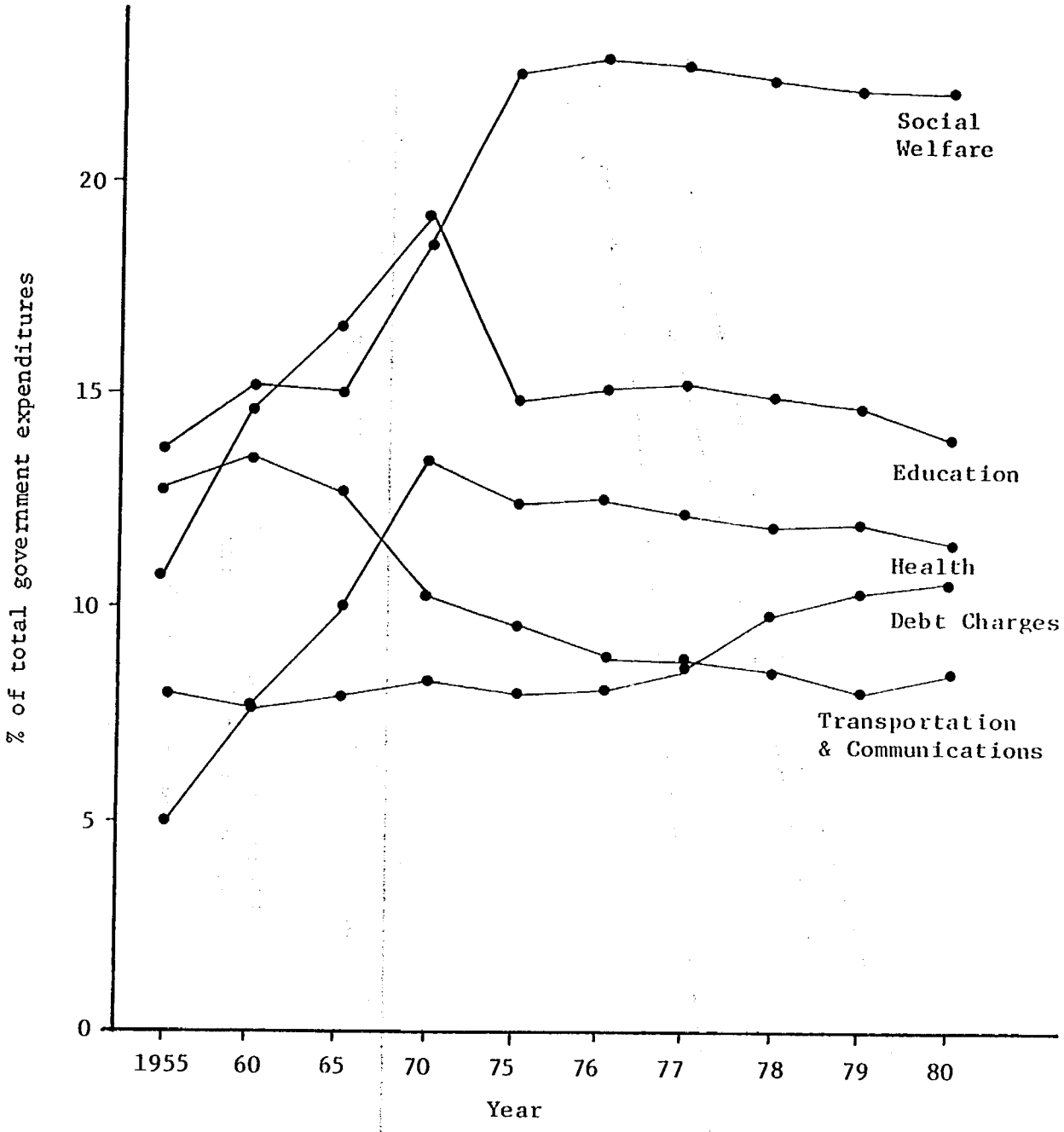


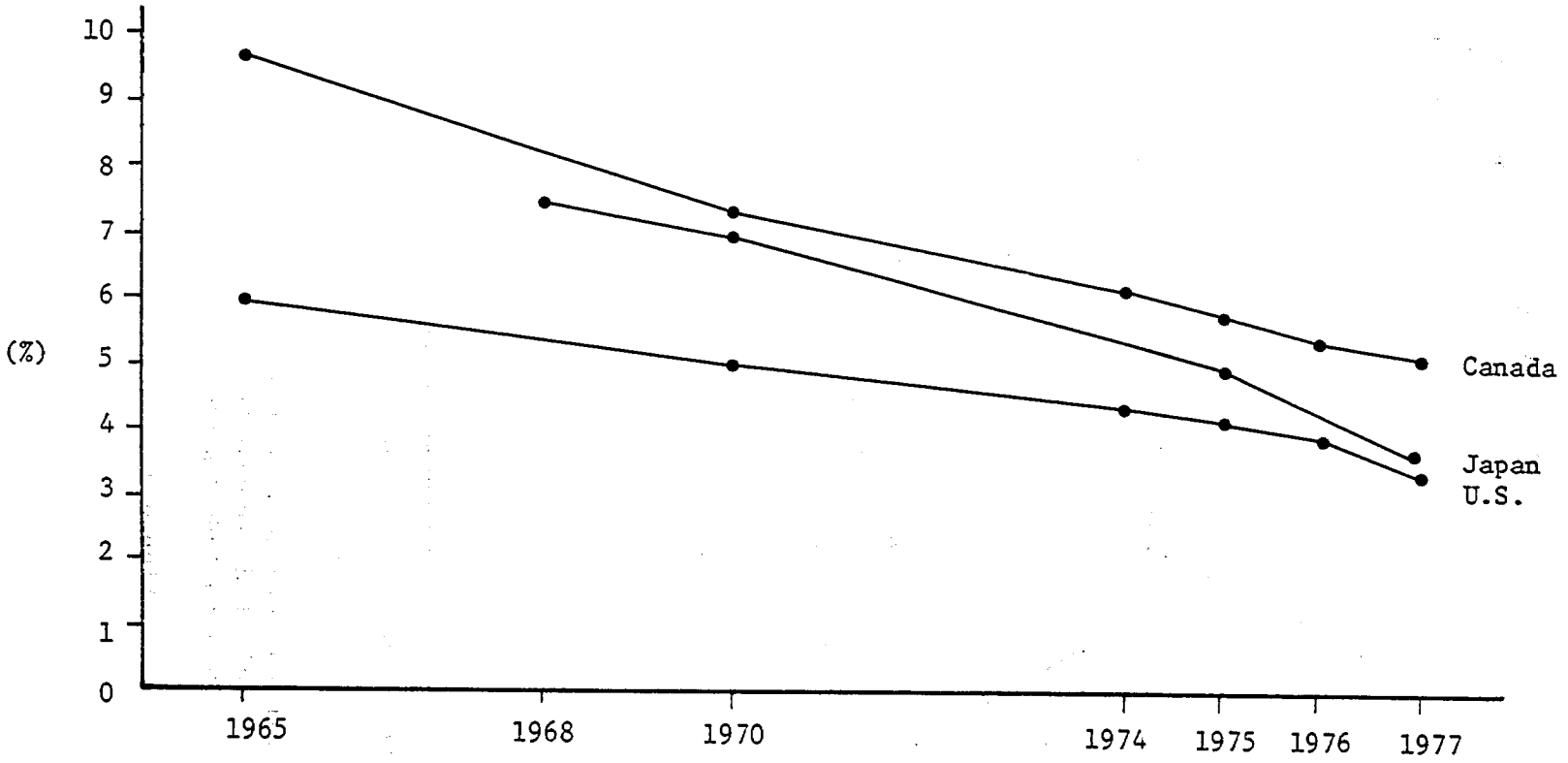
Japanese Highway
Expenditure Trends

Figure 1



Government Expenditure Trends by
Selected Program Area
1955 to 1980 Canada

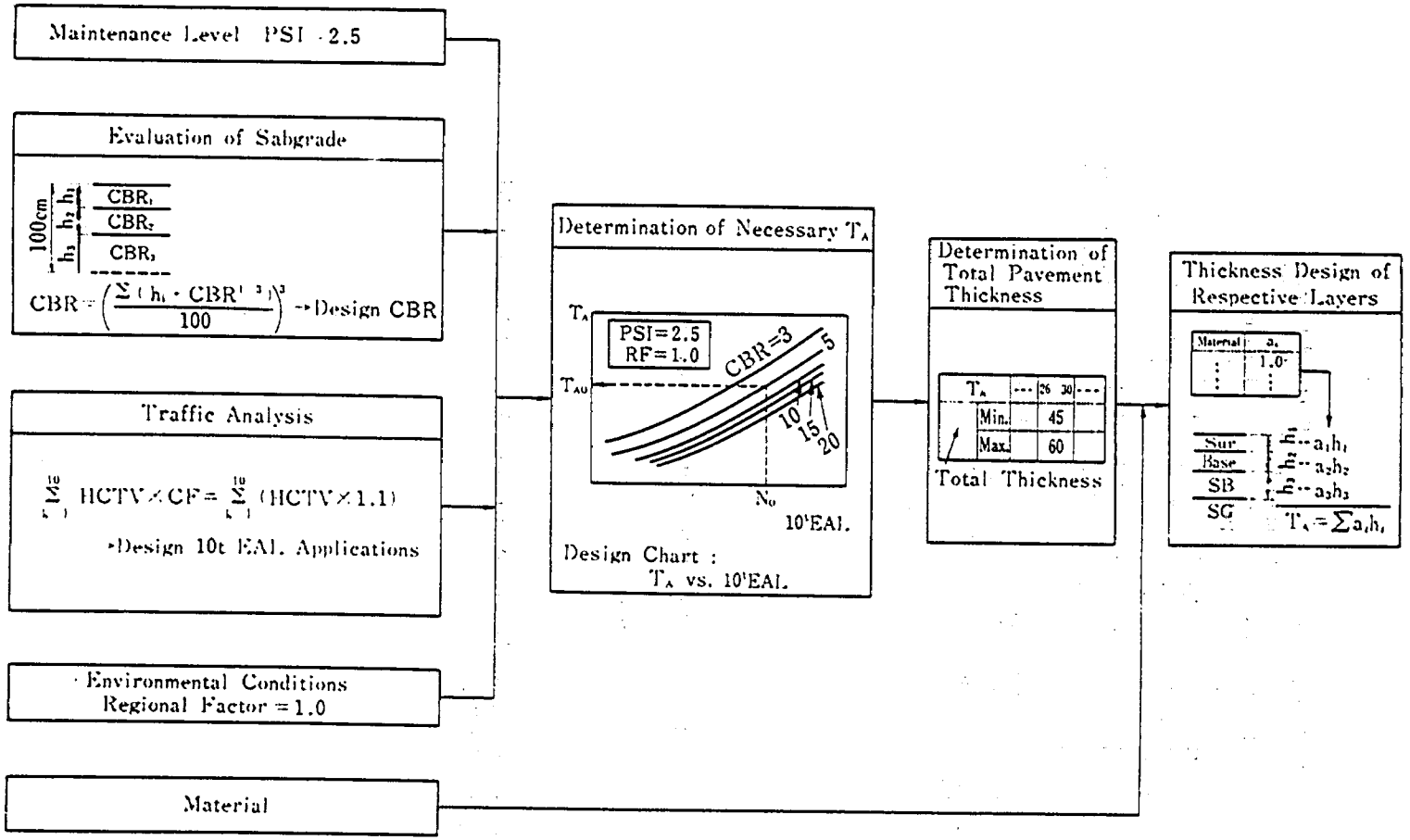
Figure 2



Highway Expenditures as % of
all Government Expenditures

Canada - Japan - USA

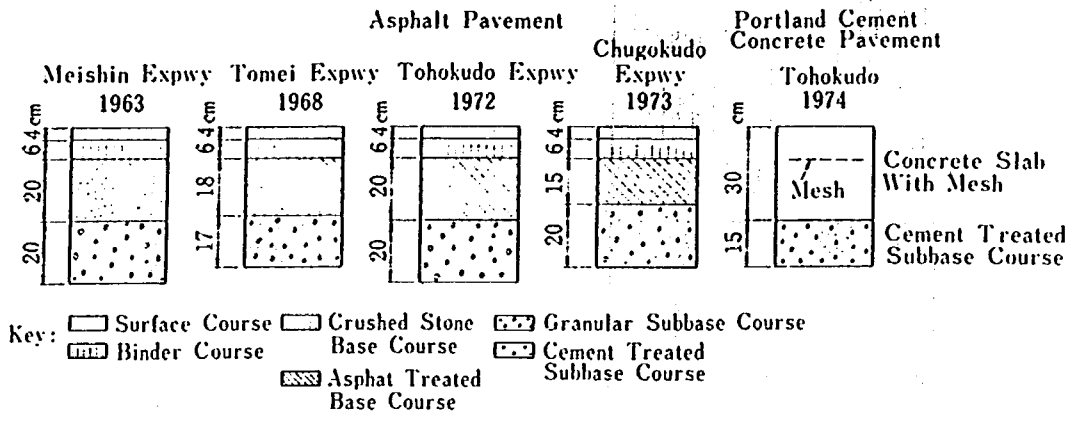
Figure 3



Structural Design Procedure for Asphalt Pavements: Japan

Figure 4

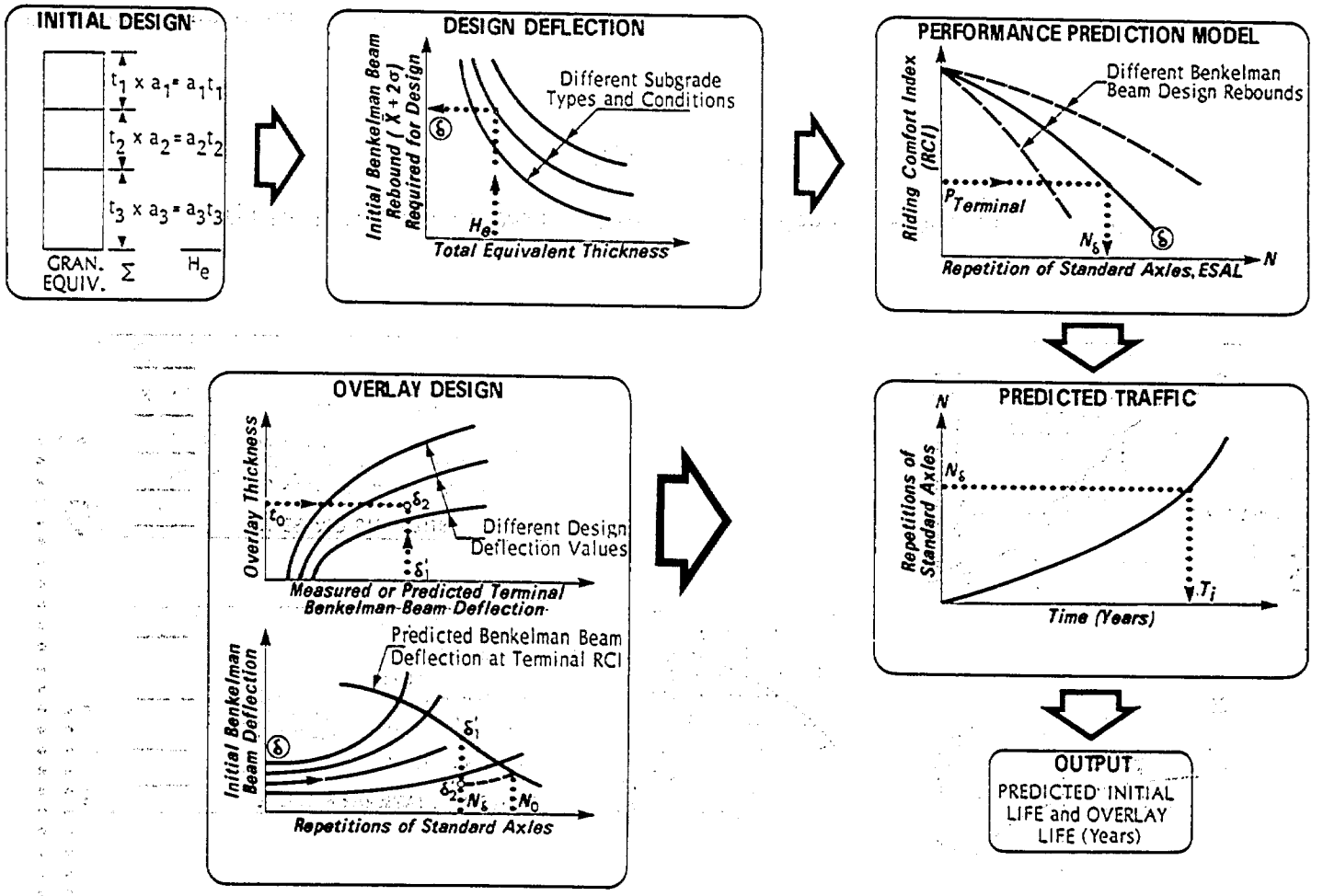
Source: National Expressway Practices in Japan. Vol: 4, Pavement. Nihon Doro Kodan, 1980



Typical Pavement Cross Sections - Japan

Figure 5

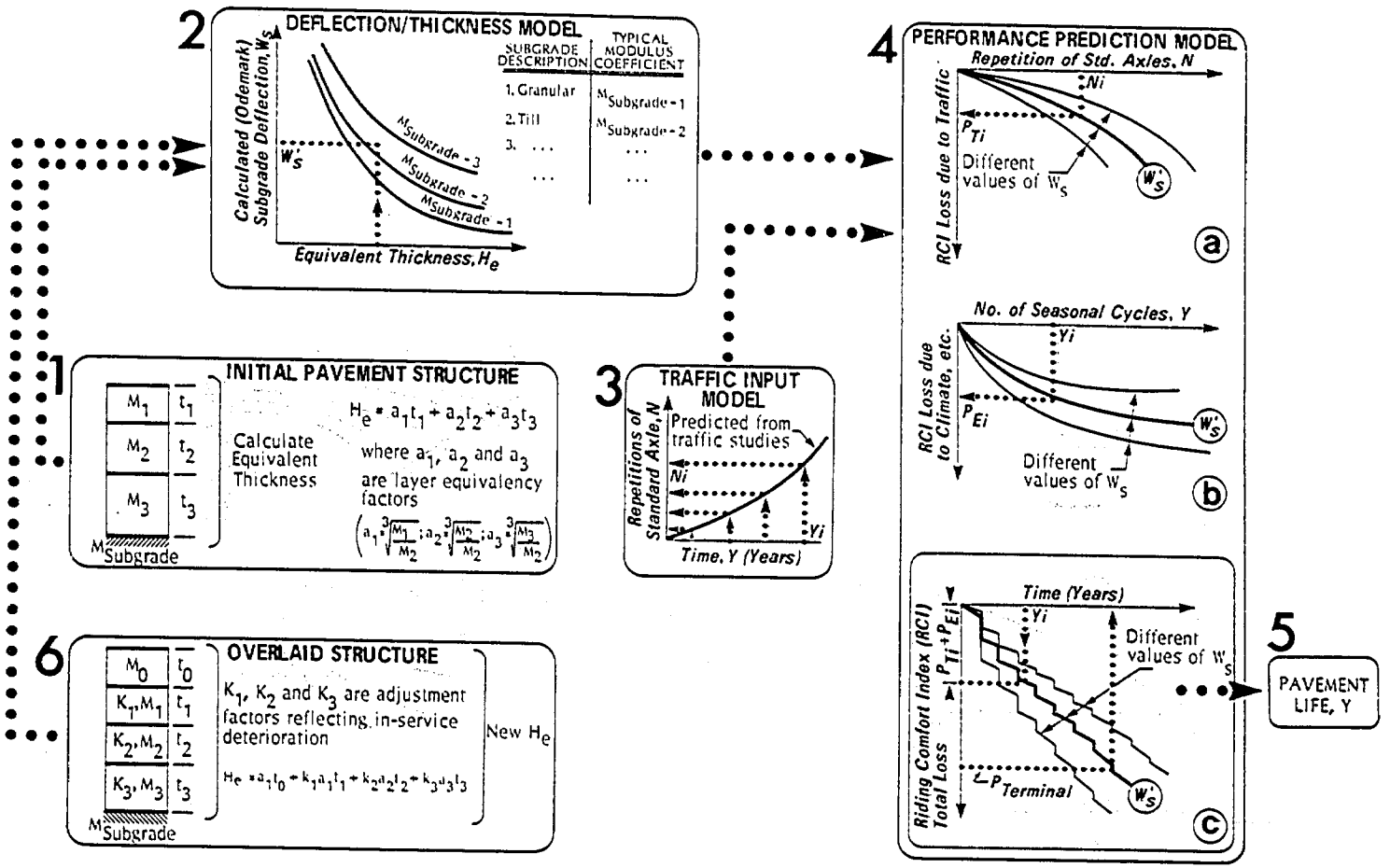
Source: National Expressway Practices in Japan. Vol. 4: Pavement Nihon Doro Kodan, 1980.



Typical Thickness Design Procedures: Canada - Empirical Model

Figure 6(a)

Source: RTAC Pavement Management Guide



Typical Thickness Design Procedures: Canada - Ontario Method

Figure 6(b)

Source: RTAC Pavement Management Guide

TABLE 1

Comparison of Temperature Indicators

Canada-Japan

Location	Annual degree days below zero	Average daily temp. in February (°C)	Average days of frost (°C)	Number of days High Temp. below 0°C
Canada				
Vancouver	41	5	149	No
Winnipeg	1940	-16	244	data
Toronto	634	-6	206	available
Quebec	1198	-11	228	
St John's	473	-5	234	
Japan				
Sapporo	No	-4	No	50
Obihiro	data	-10	data	70
Tokyo	available	4	available	0

TABLE 2

Comparison of Precipitation Levels

Canada-Japan

Location	Total Annual Precipitation in mm (Rain + Rain Equivalent Snow)
Canada	
Vancouver	1113
Winnipeg	526
Toronto	800
Quebec	1175
St John's	1514
Japan	
Hokkaido Island	1000 - 1500
Honshu Island	1500 - 2000
Kyushu Island	2000 - 2500

TABLE 3

Comparison of Snowfall Indicators

Canada-Japan

Location	Annual Average Snowfall (cm)	Number of Days Annually on which snow occurs
Canada		
Vancouver	60	10
Winnipeg	126	57
Toronto	131	41
Quebec	343	73
St John's	359	88
Japan		
Hokkaido Island	No	100
Honshu Island	data	10 - 50
Kyushu Island	available	0 - 10

TABLE 4

Japan Road Network Statistics

April 1, 1982

Category	Length (km)	Pavement			
		Excl. low cost pav.		Incl. low cost pav.	
		Ratio of paved roads (%)	Length of paved roads (km)	Ratio of paved roads (%)	Length of paved roads (km)
National	46 275	81.3	37 603	95.2	44 063
Prefectural	126 229	38.9	49 117	83.5	105 383
Municipal	947 516	10.8	102 563	44.0	416 547
Subtotal	1 120 020	16.9	189 283	50.5	565 993
Expressway	3 010	100.0	3 010	100.0	3 010
Total	1 123 030	17.1	192 293	50.7	569 003

TABLE 5

Canada Road Network Statistics

April 1, 1981

	km
Federal Highways	
Freeway	27
Arterial	2 049
Collector	700
Local	8 047
Total Length (km)	10 823
Provincial Highways	
Freeway	4 912
Arterial	62 869
Collector	55 045
Local	133 266
Total Length (km)	256 092
Municipal Highways	
Total Length (km)	661 343
Grand Total (km)	928 258
% of Total	---

TABLE 6

Summary Comparison of Road Networks

Canada-Japan

Class of Road	Canada Length in km (%)	Japan Length in km (%)
All Roads	928 258 (100%)	1 123 030 (100%)
Class 1 Roads[1]	266 915 (28.7%)	175 514 (15.6%)
Class 2 Roads[2]	661 243 (71.3%)	947 516 (84.4%)

[1] Class 1 Roads: in Canada: Federal and Provincial
in Japan : National, Prefectural and Expressways

[2] Class 2 Roads: Municipal Roads

TABLE 7

Summary Comparison of Paved vs Unpaved Roads

Canada-Japan

Class of Road	Canada Length in km (% of Total in Class) Paved	Japan Length in km (% of Total in Class) Paved
Class 1 Roads	141 465 (53%)	152 456 (86.8%)
Class 2 Roads	No Data	416 547 (43.9%)
Total	Not Available	569 003 (50.7%)

TABLE 8

Motor Vehicle Registrations

Canada-Japan

(in 1000s)

Year	Canada			Japan		
	Cars	Others	Total	Cars	Others	Total
1945	1 161	321	1 482			
1950				37	255	292
1970	6 500	1 500	8 000	7 271	8 545	15 816
1975	9 800	3 200	13 000	16 044	11 019	27 063
1980				22 751	13 639	36 390
1982	10 530	3 239	13 769			

TABLE 9

Comparison of Freight Modal Splits (1980)

Canada-Japan

	Canada	Japan
Road	18%*	40%
Rail	70%	8.6%
Water	11.9%	51%
Air	0.1%	0.1%

* Freight moved by for-hire trucking industry

TABLE 10

Comparison of Highway Expenditures as % of All Government Expenditures

Canada-Japan-U.S.A.

Year	Canada	Japan	U.S.A.
1965	10%		6%
1968		7.6%	
1970	8%	7.4%	5%
1975	7%	5.2%	4.1%
1977	6.5%	4.1%	3.4%

TABLE 11

Comparison of Permissible Loads

Canada-Japan

	Canada (kg)	Japan (kg)
Maximum Axle Loads:		
Single Axle	9 000 to 10 000	10 000
Tandem	16 000 to 20 000	20 000
Gross Vehicle Weight (GVW)	50 000 to 63 500	34 000[1]

[1] GVW normally depends on longest axle spacing

TABLE 12

Road Classification by Traffic Volume

Japan

Classification	One Way Daily Traffic Volume of of Heavy Vehicles	Cumulative 5ton Wheel Loads for 10 years
L	Less than 100	30 000
A	100 to 250	150 000
B	250 to 1 000	1 000 000
C	1 000 to 3 000	7 000 000
D	More than 3 000	35 000 000

TABLE 13

Target Values for TA and H (cm)

Design CBR	Road Classification									
	L		A		B		C		D	
	TA	H	TA	H	TA	H	TA	H	TA	H
2	17	52	21	61	29	74	39	90	51	105
3	15	41	19	48	26	58	35	70	45	83
4	14	35	18	41	24	49	32	59	41	70
6	12	27	16	32	21	38	28	47	37	55
8	11	23	14	27	19	32	26	39	34	46
12	--	--	13	21	17	26	23	31	30	36
20 or more	--	--	--	--	--	--	20	23	26	27

Table 14 - Characteristics of Thickness Design Methods*

Design Method	Text Ref. Section	Methodology	Manual or Computer	Design Effort	Input Data Requirements	Applicability
Flexible Pavements						
1. Standard Sections	5.2.1	Prescription	Manual	Low	Subgrade, Traffic, Materials	Where performance of similar pavements is well known.
2. Deflection Based						
a) RTAC b) Ontario	5.2.2	Empirical	Manual	Medium	Thickness and deflection ($\bar{x} + 2\sigma$) of test pavement; Traffic	Where deflection experience exists for various "test" pavements.
3. Stress-Strain Based						
a) Saskatchewan Shell	5.2.3.1	Theoretical Empirical	Manual	Medium	Subgrade CBR; Drainage conditions; Traffic (ESAL)	Large traffic volumes; Increased axle loads; New materials.
b) Ontario	5.2.3.2	Theoretical Empirical	Computer	High	Trial pavement thicknesses; Subgrade class and condition; Pavement layer equivalents and unit prices; Traffic volumes and ESAL RCI (initial action, after overlay)	b) Especially suitable for cost optimization.
Check Procedure By Failure Mode						
Fatigue	5.6.1	Theoretical Empirical	Computer	High	Layer thicknesses and moduli; Pavement temperatures; Asphalt mix stiffness and fatigue properties; Axle loadings	See Table 5.5.
Rutting	5.6.2	Empirical Theoretical	Manual or Computer	Medium	Layer thicknesses and moduli; Pavement temperatures; Asphalt mix stiffness; Axle loads	See Table 5.5.
Low-temperature Cracking	5.6.3	a) Prescription	Manual	Low	Design temp.; Pen. and Visc. of available asphalts	See Table 5.5.
		b) Empirical	Manual or Computer	Medium	Design temp.; Mix stiffness; Subgrade type; Layer thicknesses; Pavement age	See Table 5.5.
		c) Theoretical Empirical	Manual	Medium	Field cooling conditions; mix strength and stiffness versus temp.	See Table 5.5.
Rigid Pavements						
1. Standard Sections	5.4.2	Prescription	Manual	Low	Subgrade soil; Traffic	Where the performance of similar pavements is well known.
2. AASHTO	5.4.3	Empirical	Manual	Low	Subgrade K; Concrete flexural strength; Traffic (ESAL); Joint condition	Where performance of similar pavements is well known.New materials.
3. Brokaw	5.4.4	Empirical	Manual	Low	Subgrade; Traffic (no. of semi-trailer trucks)	Where performance of similar pavements is well known.
4. PCA	5.4.5	Semi-Theoretical Empirical	Manual or Computer	Medium	Trial pavement thicknesses; Concrete flexural strength; Axle load spectrum and repetitions; Subgrade K	Increased axle loads; New Material

* From RTAC Pavement Management Guide

Table 15 Typical Pavement Thicknesses* Canada (mm: medium subgrade)

Traffic Loading			5×10^5 ESAL (80 kN)							2×10^6 ESAL (80 kN)							
Subgrade	Type of Pavement Structure	Structure Course	B.C.	Alta	Sask	Man	Ont	Que	N.S.	N.B.	B.C.	Alta	Sask	Man	Ont	Que	N.S.	N.B.
Glacial Till Subgrade Approx. CBR = 5.0 Approx. Group Index = 10 - 12 Unified Soil Classif. = CL	Conventional (with granular base)	asphaltic concrete	100	100	100	65	75		140	100	125	100	140	125		190
		asp. stab. gr.	50							50						
		granular base	180	150	100	150	100		150	330	180	100	150	150		150
		granular subbase		180	125	380	230		455		190	305	380	230		455
		select granular					305							455		
		total	330	430	325	595	710		745	480	495	505	670	960		795
	Full Depth Asphaltic Concrete	asphaltic concrete	205	215	215				255	305	255	330				305
		subbase							305							305
		select granular														
		total	205	215	215				560	305	255	330				610
	Composite	asphaltic concrete											75			
		concrete base											180			
		soil cement											150			
		total											405			
	Deep Strength	asphaltic concrete														
		granular base											230			
		total											230			
													460			
	Soil Cement Base	asphaltic concrete	50	100	75		75		25	100	125	100		125		
		asph. stab. gravel	50							50						
		granular base					100							150		
		soil cement base	180	140	150		150		180	255	150	205		150		
		select granular					455							610		
		total	280	240	225		780		205	405	275	305		1035		

* From RTAC Pavement Management Guide

TABLE 16

Technical Issues

ENVIRONMENTAL PARAMETER	TECHNICAL ISSUE	TRANS-FERABILITY	INTENSITY CANADA	INTENSITY JAPAN	CATEGORY	SCORE
A) CLIMATIC						
. temperature	Temp. related design/repair problems	H	H	H	D, MA	9
. precipitation	snow control	M	L	M	MA	5
	ice control (vehicle)	L	L	H	MA	5
	ice control (chemical)	L	H	M	MA	6
B) ROAD NETWORK						
. system size	maintenance costs	H	H	H	MAN	9
. paved %	maintenance costs	H	H	H	MAN	9
C) ECONOMIC CONTEXT						
. veh. registrations	maintenance	H	H	H	MAN	9
. truck registrations	design life	H	H	H	D, MAN	9
. truck freight	design life	H	H	H	D, MAN	9
. government exp.	management	H	H	H	MAN	9
D) DESIGN CONTEXT						
. load regime	design life	H	H	H	D, MAN	9

Notes:

H = high
M = medium
L = low

D = design
MA = maintenance
MAN = management

A Study on Prediction of Residual Life of Existing Asphalt Pavement

Dr. M. Yamada, Dr. T. Mise
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A Study on Production of Coconut Like of Coconut Inspired Variation

Dr. M. Vasuda, Dy. I. Mtra
Osmania University

ABSTRACT

For maintenance of a pavement, its performance in service must be evaluated. The structural design principle for asphalt pavement in this country derived from the results of the AASHO Road Test. It is assumed there that the design life is ten years and the PSI drops to 2.5 at the end of the life. It is however right only under the same condition, of materials, traffic and so on, as the standard condition at the Test. In practice, pavements are constructed, and subjected to traffic, under different conditions, of course not the same as at the Test. Under different conditions, pavements will necessarily have different histories on serviceability.

It must be therefore expected that the pavement performance is variable. The performance of a pavement assessed at the designing stage must be modified during its life, if necessary and possible. Perhaps the signs of distress such as cracking and rutting on the surface and, more desirably, mechanical properties of the pavement such as deflection will serve to reassess the performance of the pavement. In this study, cracking was adopted.

A equation for predicting the residual life of an existing pavement from measurement of the crack ratio was produced by the results of the AASHO Road Test and some assumptions. A relation between residual lifes calculated by this equation and actual results on construction of strengthening, were discussed. The residual life may be useful as a performance index for maintenance.

A STUDY ON PREDICTION OF RESIDUAL LIFE OF EXISTING ASPHALT PAVEMENT

by

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and

Tadashi Mise

Osaka City University

1. Introduction

A pavement deteriorates under the action of traffic. During the process of deterioration, the road surface exhibits visible signs of distress such as cracking, rutting, and consequently the level of service to the road user decreases. When the level has falled to be unacceptable, strengthening by overlaying or reconstruction is required. It can be generally considered that the pavement is reaching the end of its life at that time.

The life of a pavement will be therefore influenced by how the level of the pavement is evaluated. At the AASHO Road Test[1)], the level of pavement was evaluated by using the present serviceability index (PSI), which is a function of the measurements of longitudinal and transverse profile variations and the amount of cracking and patching. It was considered that a PSI value of 2.5 represented the critical condition of a pavement likely to require an overlay in the near future and a value of 1.5 represented the condition of a pavement unfit to carry further traffic.

The structural design principle for asphalt pavement in this country derived from the results of the Test. It is assumed there that the design life is ten years and the PSI drops to 2.5 at the end of the life. It is however right only under the same condition, of materials, traffic and so on, as the standard condition at the Test. In practice, pavements are constructed and subjected to traffic under different conditions, of course not the same as at the Test. Under different conditions, pavements will necessarily have different histories an serviceability.

It must be therefore expected that the pavement performance is variable. The performance of a pavement assessed at the designing stage must be modified during its life, if necessary and possible. Perhaps, signs of distress such as cracking and rutting on the surface and, more desirably, mechanical properties of the pavement structure such as deflection will serve to reassess the performance of the pavement.

Recently a number of instruments available for measuring the structural condition of existing pavements, such as an automatic measuring instrument of dynamic deflection, have been developed and the pavement evaluation method are becoming more theoretical and high-technological. However, it is sure that conventional simple-methods are still used often considering practicality.

In this paper, a method to assess the residual life of pavement from measurement of the crack ratio, and a relation between residual lifes assessed by the method and actual results on construction of strengthening, are discussed to pursue the practical performance-index for asphalt pavements which deteriorate primarily from cracking.

2. Derivation of an Equation for Residual Life

The serviceability of pavements are represented by the PSI, used at the AASHO Road Test, in this study, too. The PSI decreases with the accumulation of axle load applications (W) as is illustrated by Fig-1. The life of a pavement (W_l) is defined as the accumulation of axle load applications at which the PSI is a final value. Then, the residual life of the pavement (W_r) is :

$$W_r = W_l - W \tag{1}$$

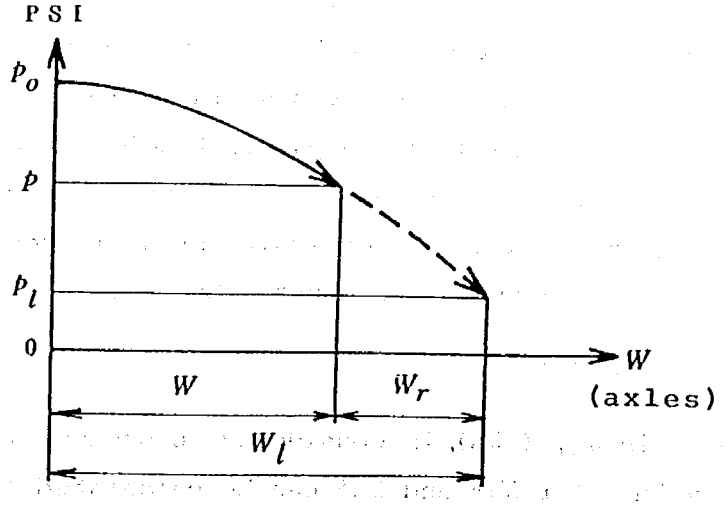


Fig-1 Relation among W, W_l and W_r

When a pavement structure is designed using the method in the manual, the design life (W_l*) is assumed. But, it is not always equal to the actual

life. At the planning of maintenance and strengthening, it must therefore be modified in the light of informations on performance obtained in service. The difference between W_l^* and W_l is denoted by ΔW_l (see Fig-2).

$$\Delta W_l = W_l - W_l^* \tag{2}$$

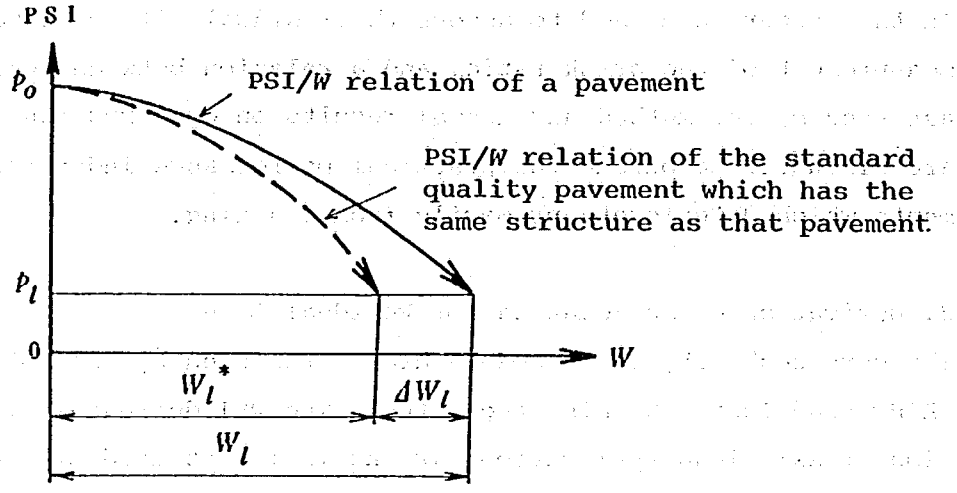


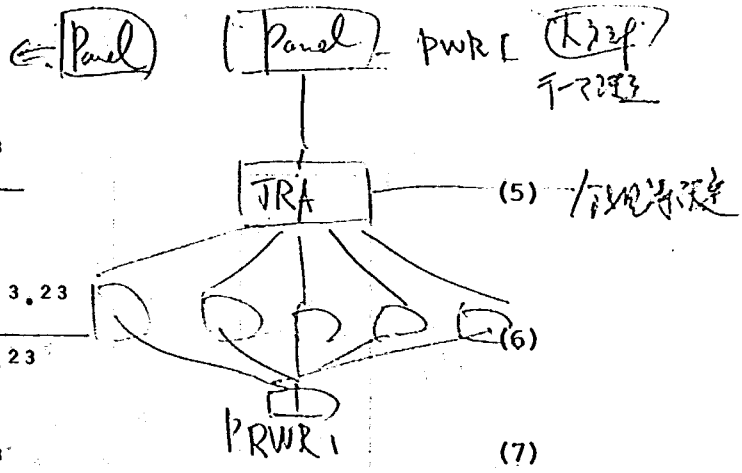
Fig-2 Relation among W_l, W_l^* and ΔW_l

Cracking is an important element of structural deterioration that detractes from serviceability of asphalt pavement. Although cracks do not in themselves have much effect on the ability of the pavement to serve traffic, the rate of development of the cracking on the surface is the most visible indicator of the performance of pavement in service. The number of axle load applications sustained by the pavement before the appearance of cracking (W_c) is selected as the index to represent the development of cracking. The value of W_c expected at the designing stage is denoted by W_c^*. The difference between W_c^* and W_c is denoted by ΔW_c.

$$\Delta W_c = W_c - W_c^* \tag{3}$$

The difference in W_c, (ΔW_c), is considered to produce the difference in W_l, (ΔW_l). The values for W_l^* and W_c^* can be calculated from the following equations derived at the AASHO Road Test[1]. The value of the W_l is hereupon calculated as one at PSI = 2.5.

$$\log W_l^* = \log\rho + \frac{\log\left(\frac{4.2 - p_l}{2.7}\right)}{\beta} = \log\rho - \frac{0.20}{\beta} \tag{4}$$



$$\rho = \frac{10^{5.93}(D+1)^{9.36} L_2^{4.33}}{(0.22L_1 + L_2)^{4.79}} \quad (5)$$

$$\beta = 0.4 + \frac{0.081\,(0.22L_1 + L_2)^{3.23}}{(D+1)^{5.19} L_2^{3.23}} \quad (6)$$

$$D = 0.173D_1 + 0.055D_2 + 0.043D_3 \quad (7)$$

$$\log W_c^* = 5.48 + 7.28\log(D_c + 1) + 2.95\log L_2 - 3.14\log(0.22L_1 + L_2) \quad (8)$$

$$D_c = 0.13D_1 + 0.04D_2 + 0.03D_3 \quad (9)$$

where, L_1 : nominal axle load, kN

L_2 : 1 for single axles or 2 for tandem axles

D_1 : surfacing thickness, cm

D_2 : base thickness, cm

D_3 : subbase thickness, cm

If the relation between ΔW_l and ΔW_c is assumed to be linear, then the following equation is given by analysis of the results of the AASHO Road Test (see Fig-3).

$$\Delta W_l = 0.18 \times 10^5 + 0.6\Delta W_c \quad (10)$$

If W_c is given from results of a follow-up survey on cracking, the residual life (W_r) can be calculated using Exs.(1)~(10). When the cracking survey is done at just one point of time, W_c must be predicted from W and crack ratio ($\overline{CP}$) at that time, where $\overline{CP}$ is a measure of cracking and patching in the pavement surface.

$$\overline{CP} = \frac{\text{Total Cracking Area + Total Patching Area}}{\text{Area of Surveyed Section}} \times 100 \quad (\%) \quad (11)$$

For the prediction of W_c, the crack ratio ($\overline{CP}$) of a pavement is assumed to increase having a straight line relation to the axle load applications (W) after the appearance of cracking. The slope of the $\overline{CP}/W$ line, (λ), is also assumed to be equal to that for the standard quality pavement with the same structure (see Fig-4).

$$\overline{CP} = \lambda\,(W - W_c) \quad (12)$$

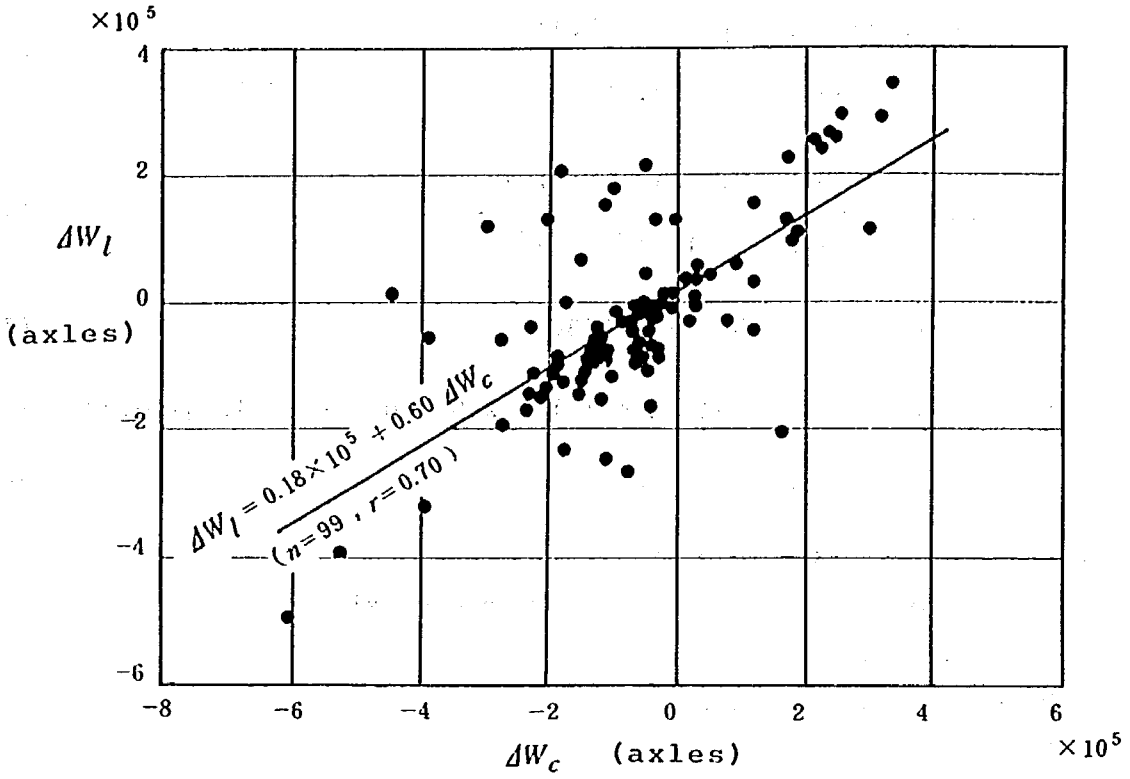


Fig-3 Relation between ΔW_l and ΔW_c (AASHO Road Test)

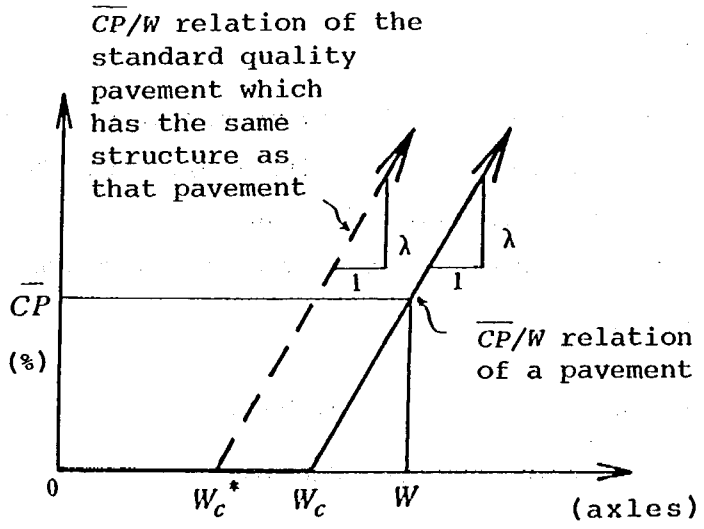


Fig-4 Assumption on λ

These assumptions are suggested by survey results at some actual roads, such as shown Fig-5 and Fig-6. It is observable in Fig-5 that $\overline{CP}$ has a linear relation to the cumulative heavy vehicles when it is within the range of a few to tens percent. Fig-6 shows survey examples of the relation

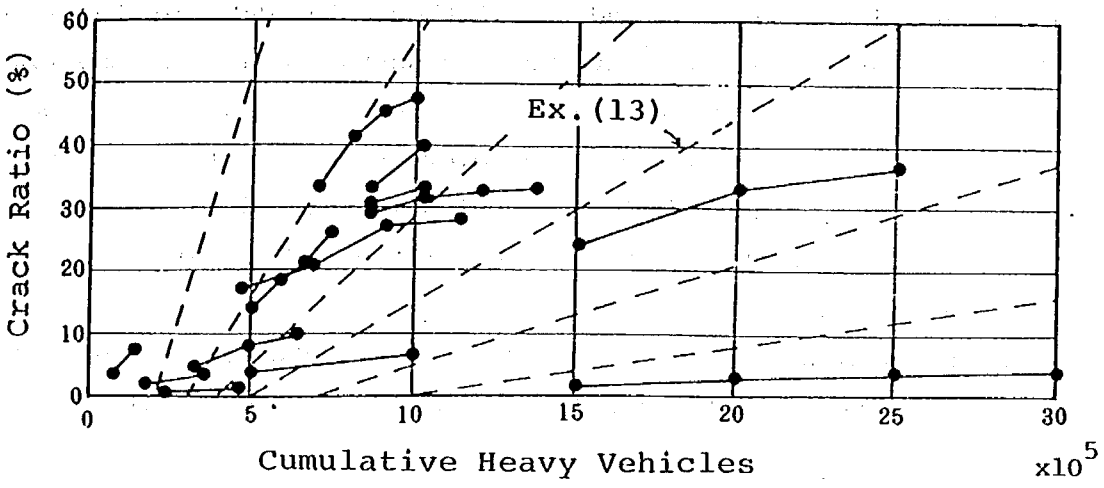
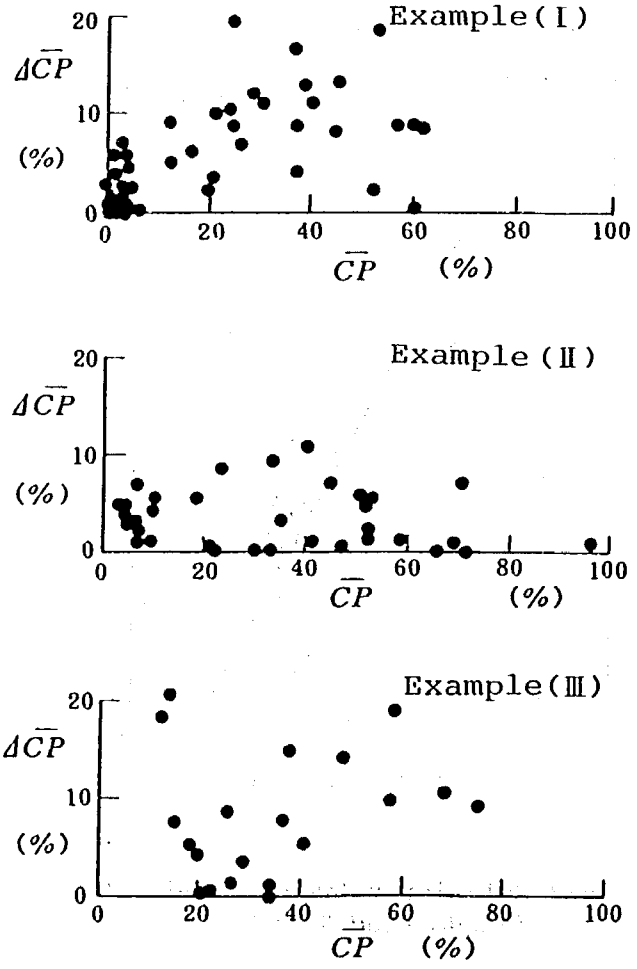


Fig-5 Crack Ratio/Cumulative Heavy Vehicles Relation
(Roads in Kobe City)



$\Delta\bar{CP}$: Annual Increase of Crack Ratio (%)
$\bar{CP}$: Initial Crack Ratio (%)

Fig-6 Examples of Survey Result on Increase
of Crack Ratio (Roads in Kobe City)

between annual increase ($\Delta\overline{CP}$) and $\overline{CP}$. Although the data plotted in each example were measured on pavements which constructed at the same time and have the same structure, there are wide differences among the values of $\overline{CP}$. It shows that W_c is very variable. However, the slope (λ) is assumed to be independent on ΔW_c, since $\Delta\overline{CP}$ is not obviously related to $\overline{CP}$ in each surveying section, although the average values of the $\Delta\overline{CP}$ are different from that in each other's section.

When λ is a function of W_c* for the pavement structure, the following equation is derived from the results of the AASHO Road Test,too (see Fig-7).

$$\log\lambda = 6.65 - 1.96\log W_c^* \qquad (13)$$

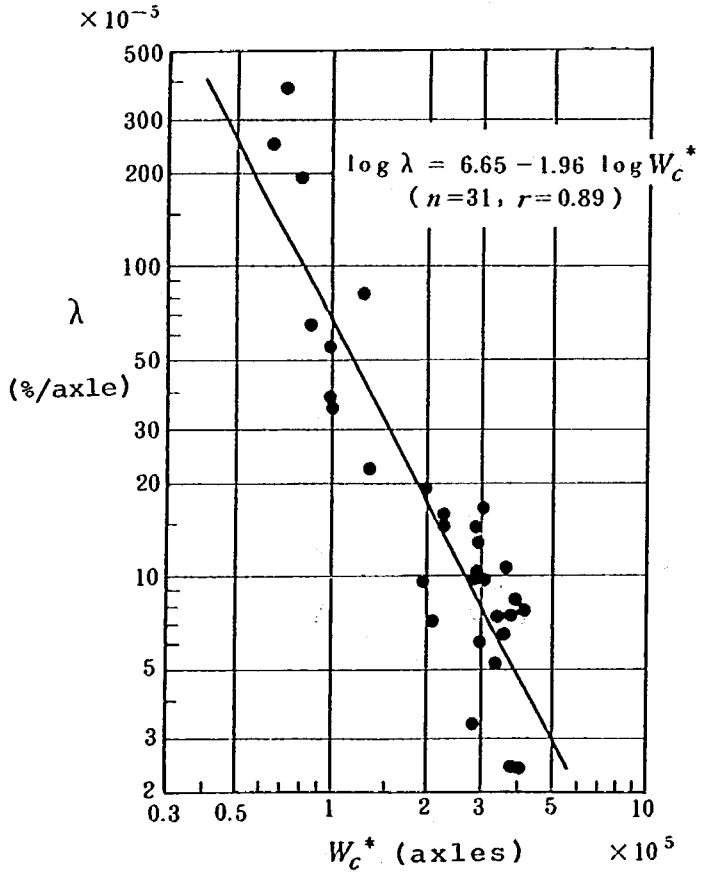


Fig-7 Relation between λ and W_c*
(AASHO Road Test)

Straight broken-lines drawn in Fig-5 illustrate Ex.(13). They can be found to be similar to the trends on the survey results in Kobe[2),3)].

Other survey results, Fig-8 and Fig-9, also show the suitability of Ex.(13).

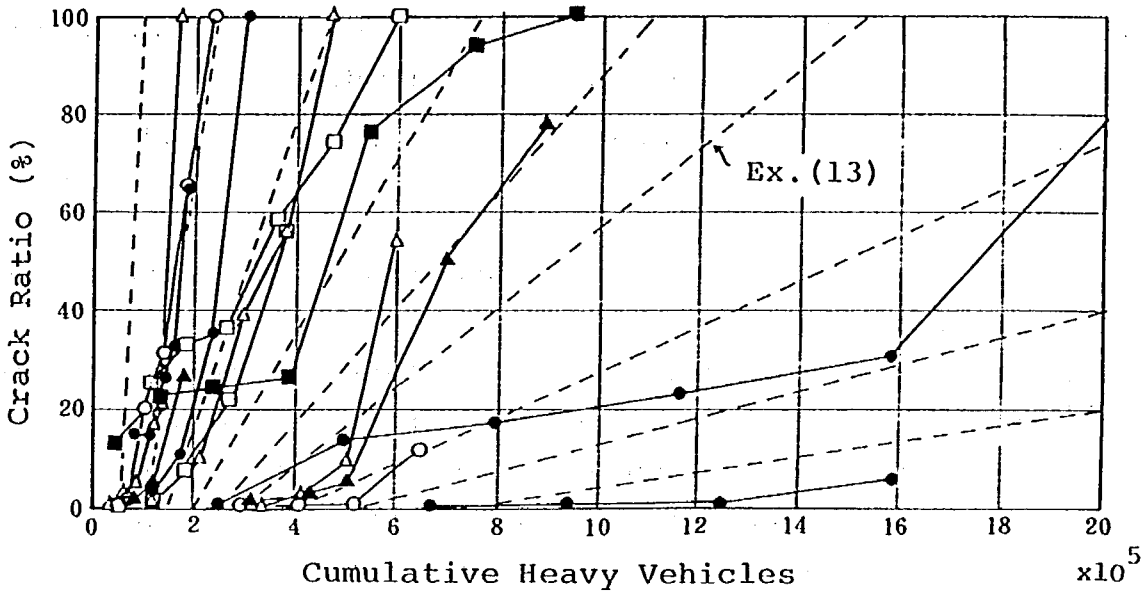


Fig-8 Crack Ratio/Cumulative Heavy Vehicles Relation
(Low cost pavements in Ehime Prefecture[4])

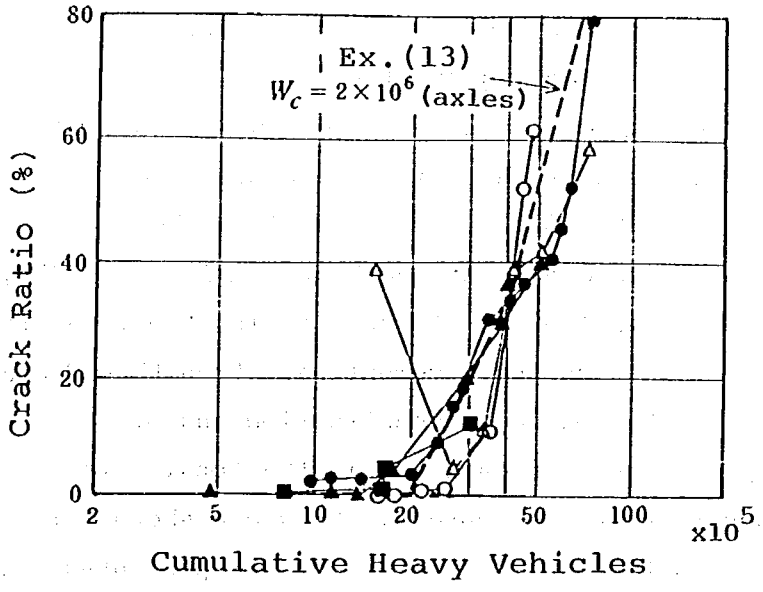


Fig-9 Crack Ratio/Cumulative Heavy Vehicles Relation
(surveyed by Public Works Research Institute,
Ministry of Construction[5])

An equation for calculating ΔW_c, of a pavement at which the crack ratio is $CP(>0)$ when the accumulation of axle load applications is W, is obtained from Exs.(3), (12) and (13).

$$\Delta W_c = W - 2.24 \times 10^{-7} W_c^{*1.96} \overline{CP} - W_c^* \quad (14)$$

Substitution of Ex.(14) into Ex.(10) gives ΔW_l. The ΔW_l and Exs.(1) and (2) produce an equation for predicting W, as following.

$$W_r = 0.18 \times 10^5 + W_l^* - 0.60 W_c^* - 0.40 W - 1.3 \times 10^{-7} W_c^{*1.96} \overline{CP} \quad (15)$$

3. Calculations of the Residual Life and their Relations to Statistics of Strengthening Construction

Residual life equations for some pavement structures designed by using the manual[6] are calculated, on two units of axles and years, as shown in Table-1, provided that the number of axle load applications per year is 10^5. The pavement structures in Table-1 are for various road classification by traffic, and the materials used in their base and subbase courses are mechanically stabilized gravel and crusher-run respectively. The value of T_A for each structure satisfies the target in the manual[6].

The equations are also put into the form of a diagram in Fig-10. Each of arrows of broken line in this figure shows just the standard development of cracking which was assumed at the derivation of Ex.(15). The residual life of the pavement of which cracking develops along the arrow falls into zero after about ten years, which is just the design life assumed in the manual. The figures also suggest that the critical values of crack ratio for pavement maintenance should be small as the traffic is heavier.

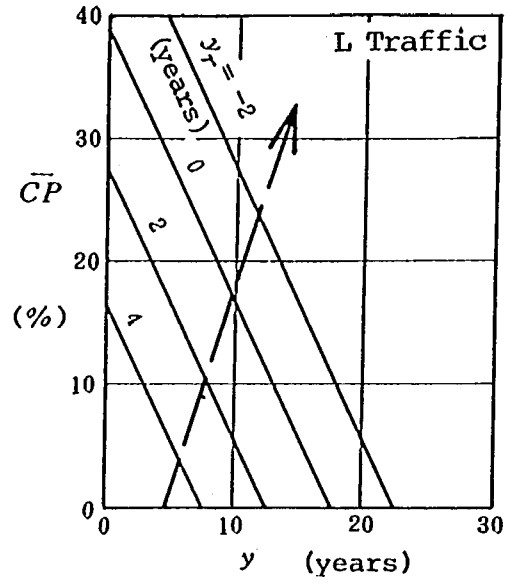
Results of a survey on pavements strengthened during the 1979, of national highways, are given in Table-2[7]. In this table, the total road length and its percentage of which the pavement was strengthened are shown for each range of years after pavement construction and of crack ratio of the pavement. The upper number in each box is the total strengthened length and the lower number is its percentage. They are shown for various road classification by traffic, and the values of the percentage tend to be larger as the years and the crack ratio increase.

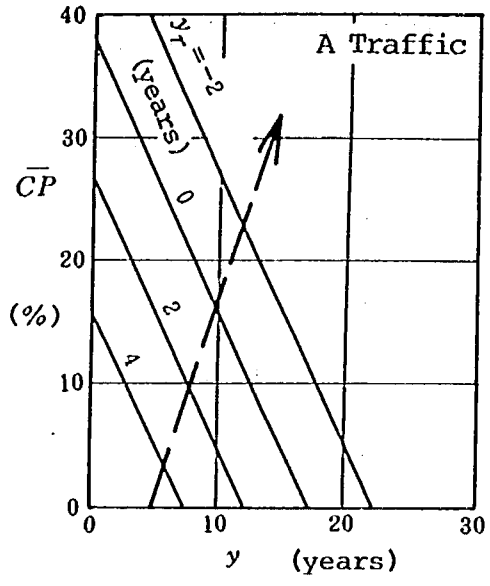
The thick lines and broken lines in Table-2 show the boundaries where the percentage is ten and twenty respectively. While the difference made by classification of traffic is not apparent, the tendency on these lines is similar to that on the lines for residual life in Fig-10.

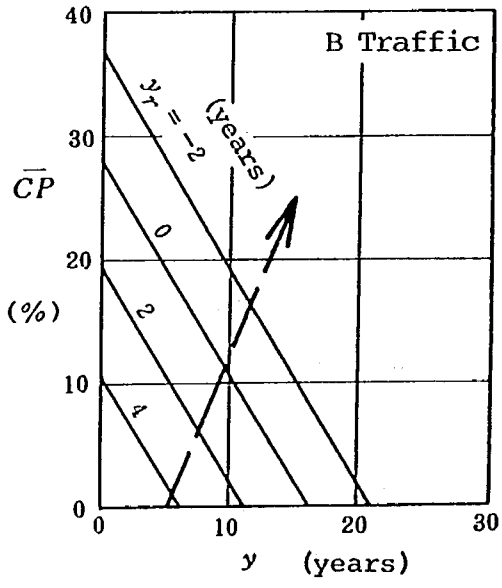
The percentage, which is the rate of the total pavement-strengthened road length to the total road length, is perhaps related to the judgement upon

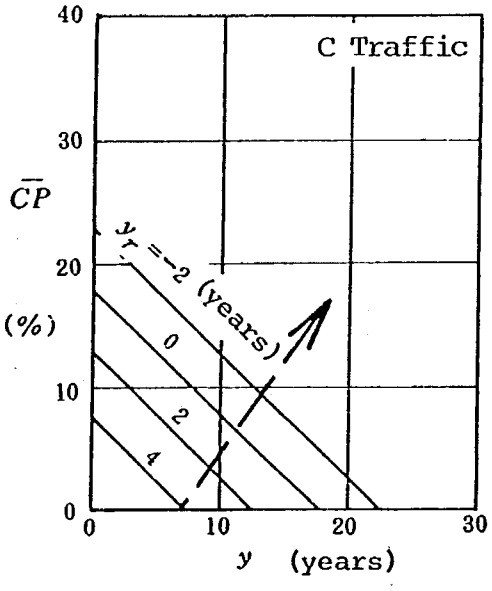
Table-1 Examples of Calculation by Ex.(15)

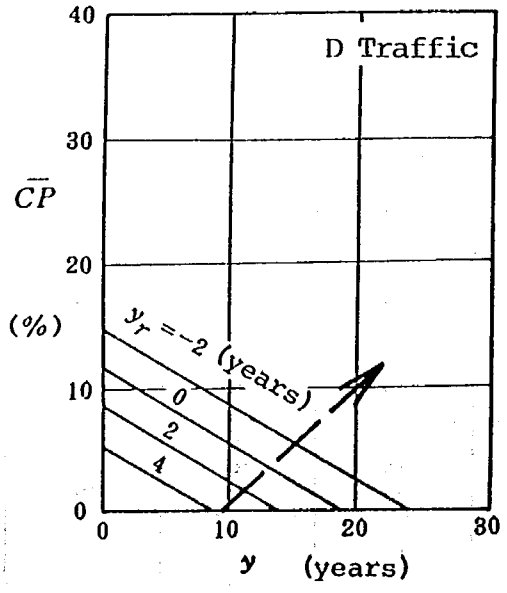
Traffic	Axle Load L_1 (kN)	CBR (%)	Thickness of Layers D_1 (cm)	D_2 (cm)	D_3 (cm)	Thickness Index D (cm)	D_c (cm)	Design Life W_l^* (axles)	W_c^* (axles)	Residual Life Equation W_r (axles), y_r (years)
L	40	3	5	15	20	2.55	1.85	9.7×10^5	4.8×10^5	$W_r=7.0\times10^5-0.4W-1.8\times10^4\overline{CP}$ $y_r=7.0-0.4y-0.18\overline{CP}$
A	60	3	5	20	30	3.26	2.35	9.6×10^5	4.8×10^5	$W_r=6.9\times10^5-0.4W-1.8\times10^4\overline{CP}$ $y_r=6.9-0.4y-0.18\overline{CP}$
B	100	3	10	25	30	4.40	3.20	9.7×10^5	5.5×10^5	$W_r=6.6\times10^5-0.4W-2.3\times10^4\overline{CP}$ $y_r=6.6-0.4y-0.23\overline{CP}$
C	160	3	15	40	25	5.87	4.30	11.1×10^5	7.2×10^5	$W_r=7.0\times10^5-0.4W-3.9\times10^4\overline{CP}$ $y_r=7.0-0.4y-0.39\overline{CP}$
D	240	3	20	50	30	7.50	5.50	12.8×10^5	9.2×10^5	$W_r=7.5\times10^5-0.4W-6.4\times10^4\overline{CP}$ $y_r=7.5-0.4y-0.64\overline{CP}$











$\overline{CP}$: Crack Ratio (%)

y : Years after Construction

y_r : Residual Life (years)

Fig-10 Residual Life of Pavement with Each Traffic Class (see Table-1)

Table-2 Survey Results on Pavement Strengthening
(Roads under the direct control of Ministry of Construction)[7)]

Upper: Total Road Length Conducted Strengthening during the 1979

Lower: its Percentage $= \frac{\text{Total Strengthened Length}}{\text{Total Length under the Control}} \times 100$ (%)

Full Line: Strengthening Percentage of 10%

Broken Line: Strengthening Percentage of 20%

Traffic / Years / Crack Ratio (%)	B					C					D				
	0～5	6～10	11～15	16～20	21～	0～5	6～10	11～15	16～20	21～	0～5	6～10	11～15	16～20	21～
0～10	1.0 0.2	5.0 0.7	4.7 1.3	1.0 2.3	0 -	8.1 0.4	31.1 2.6	29.9 4.2	6.2 3.5	2.0 6.4	3.5 .3	35.8 4.2	29.9 10.8	7.0 9.5	0 -
11～20	0 -	7.2 5.0	4.5 2.2	7.0 37	3.0 60	3.3 1.9	36.9 12.8	24.2 7.7	22.4 22.0	2.8 19	2.0 2.7	19.8 8.6	19.3 13.1	5.7 11	2.7 24
21～30	0 -	7.5 10	20.0 16.9	6.0 40	0 -	5.5 15	23.4 16.0	36.6 22.3	24.1 34.4	7.2 29	1.9 7.3	10.8 10.5	25.9 24.6	10.3 19.7	3.0 24
31～40	0 -	4.1 21	10.1 13.4	5.1 18	0 -	0.4 3	1.9 11	20.1 15.4	12.2 33.7	1.6 20	0 -	3.1 11	7.8 21	4.2 14	0.4 6
40～	0 -	1.0 24	6.3 47	0 -	0 -	2.1 19	4.1 36	1.4 3.2	2.0 17	2.0 50	0 -	0 -	2.0 25	0 -	0 -

whether or not the pavement should be strengthened. Therfore, if the relation of strengthened-percentage to crack ratio and years can obtain by analyzing data such as Table-2, the strengthened-percentage predicted by utilizing it will serve as a pavement performance index for maintenance strategy, for example suggesting the priority order of strengthening, and seems to be related to the residual life mentioned above.

4. Conclusion

The equation for predicting the residual life of a pavement is not sufficiently accurate, but will serve for maintenance strategy of asphalt pavements which deteriorate primarily from cracking.

The relation of strengthened-percentage to crack ratio and years obtained by analyzing data such as Table-2 will also be useful, and seems to be related to the residual life.

Thus, the index for evaluating the performance of existing pavements should be a function of the road-surface conditions and the years.

5. Acknowledgements

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Estimation of Fatigue Life for Asphalt Pavements

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Tokyo Institute of Technology

ABSTRACT

There are many proposals for estimating fatigue life of asphalt pavement, however, the universal fatigue criterion is still not accepted among researchers. The difference in fatigue conditions between laboratory bending test and actual failure in the field is one of the causes mentioned above. And the variable field conditions such as transverse distribution of wheels, wheel loads, vehicle speed and ambient temperatures are other reasons. These field conditions can be random variables which are distributed statistically.

In this paper, the laboratory model fatigue test was performed to check the proposed fatigue criterion, and the computer simulation of fatigue life estimation in actual conditions was made by probabilistic approach.

The laboratory model test was made by wheel tracking test equipment. A slab of asphalt mixture of about 3 cm thick was placed on a rubber foundation base, then the slab was subjected to the repeated passing of a wheel. The test temperature was 20 C and the strains at the bottom of the slab were measured using strain gauges. Proposed fatigue criterion (fatigue failure envelope) was found to be non-applicable for the hairline crack stage but can be applicable when the real cracks are formed and widened.

Computer simulation of fatigue life on an actual road was made on the national route No.357 (Tokyo Bay Ring Road). The distributions of transverse wheel positions, wheel loads, vehicle speeds and daily atmospheric temperatures were calculated using the data from the site, and actual fatigue failure process was simulated stochastically. Even though the actual site has not yet reached its fatigue life, the estimated fatigue life of asphalt pavement was found to be statistically expressed by lognormal distribution pattern. And also the transverse distribution effect of wheel position made the predicted fatigue life two or three times longer than the life estimated by neglecting this effect.

ESTIMATION OF FATIGUE LIFE FOR ASPHALT PAVEMENTS

by
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and
Kenji Himeno
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1. INTRODUCTION

Asphalt pavement is widely used in Japan, as it can be constructed quickly and repaired easily. In Meishin Expressway which is the first expressway in Japan, many cracks were found in the pavement surface which were supposed to be caused by fatigue failure of asphalt mixtures, after that Japan's policy regarding the pavement design was slightly changed and comparatively thick bituminous top layer was adopted to prevent such fatigue cracks and now the permanent deformation of asphalt pavements or rutting is being considered a serious problem. Thus the fatigue failure and the permanent deformation are mutually contradictory. And it is also quite important to develop the optimum design system which compromise both requirements of saving resources and easy maintenance.

This paper deals mainly with the fatigue failure crack. The purposes of this study are to: (1) clarify the relation between fatigue failure by a simulation model experiment and conventional laboratory bending fatigue test (controlled strain bending fatigue test), (2) investigate the effect of the binder type on the fatigue life of pavements and (3) predict the pavement life considering external force and material properties of the pavements in a stochastical way.

2. FATIGUE FAILURE CHARACTERISTICS OF ASPHALT MIXTURES

(1) Laboratory Fatigue Test

After Hveem[1] pointed out in 1955 that fatigue phenomenon is one of the causes of asphalt pavement cracking, many researchers and engineers have been studying of this problem. Most of them deal with laboratory bending fatigue tests which can be divided into the following types:

A. Controlling method, a) controlled stress test and b) controlled strain

test,

B. Loading system, a) bending test by simple beam (one force[2,3], two forces[4]), b) bending test by cantilever[5], and c) tortional test by cantilever[6],

C. Evaluating method of failure, a) only tensile stress[7], b) only tensile strain[8], c) both tensile stress and strain[9], d) tensile stress and stiffness[5,10].

To predict the fatigue life of asphalt pavements, multi-layered elastic theory or finite element method based on elastic theory is used considering tensile stress and/or strain at the bottom surface of the asphalt mixture layer. And to accumulate fatigue damage, Miner's hypothesis is applied. But as stress, loading, and temperature conditions of real pavements in-situ differ much from those of laboratory bending fatigue test, and as different definition of fatigue failure point have been proposed in laboratory test by many researchers, there is no consistent method accepted among engineers to estimate real life in-situ by laboratory fatigue test. In this study, laboratory bending fatigue test of controlled strain method with center loading simple beam is adopted as the test method and Maruyama's fatigue failure envelope[9] as the failure criterion.

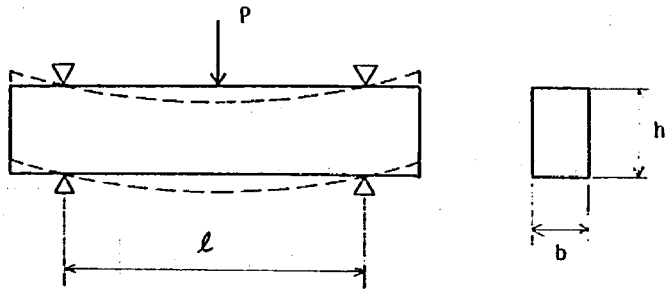

Figure 1. Schematical Illustration of Laboratory Bending Fatigue Test

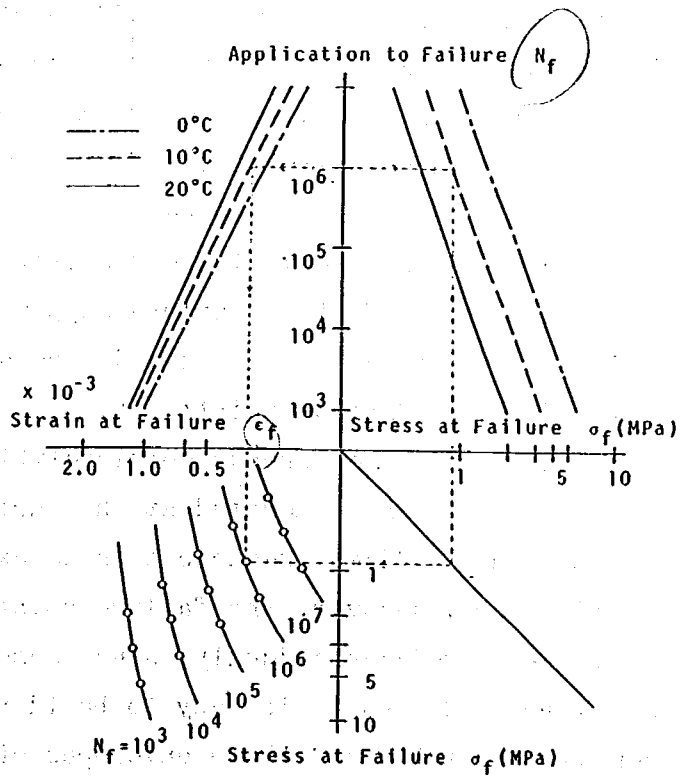

Figure 2. Concept of Fatigue Failure Envelope

(2) Fatigue Failure Envelope

The fatigue failure envelope of asphalt mixture[9] was proposed extending the theory of the

statical failure envelope [12] to fatigue failure tests, which was confirmed to yield the fatigue failure criterion depending only on the mixture and independent from the test conditions such as temperature and rate of loading. The fatigue failure envelope can be expressed as a group of curves on a logarithmic scale of stress and strain at failure with parameter N_f that is the number of load repetitions to failure. Controlled strain fatigue test was made with simple supported beam under the constant temperature and strain to obtain the required number of road repetitions N_f and stress at failure. Then varying test conditions, the envelope can be obtained as Figure 1 and Figure 2 shown. The number of load repetitions until the bending fatigue failure happens can be obtained from the envelope giving the condition of stress and strain. Several definitions have been proposed on the fatigue point:

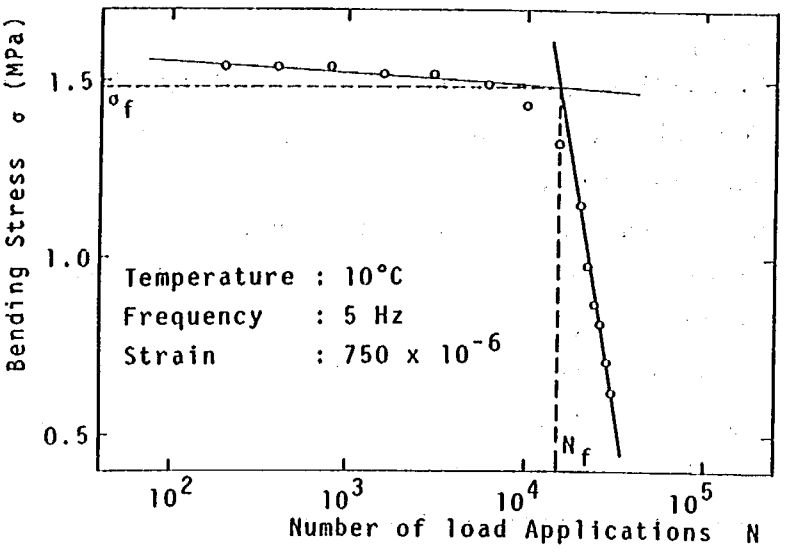

Figure 3. Definition of Failure Point

Table 1. Binder Types

Binder Type	Pen (1/100 cm)	R.& B. (°C)	P.I.
Straight Asphalt (20/ 40)	22	62	-0.4
Straight Asphalt (60/ 80)	64	49	-0.9
Straight Asphalt (150/200)	174	40	-0.7
Modified Asphalt (Type- A)	48	60	+0.9
Modified Asphalt (Type- B)	54	58	+0.8

a) when the specimen breaks completely[13]

b) when cracks are generated at the surface[14]

c) when the stiffness reduces to some extent[15,16]

In this study, however, the failure point was defined at the time when the stress level reduces suddenly, since some physical change is considered to take place and also it is easy to be identified as Figure 3 shows. Thus several fatigue failure envelopes of dense graded asphalt concrete (13 mm top) were drawn with various kinds of binder types (Table 1), and these were shown in Figure 4 a.-e. Since no experimental data were obtained in

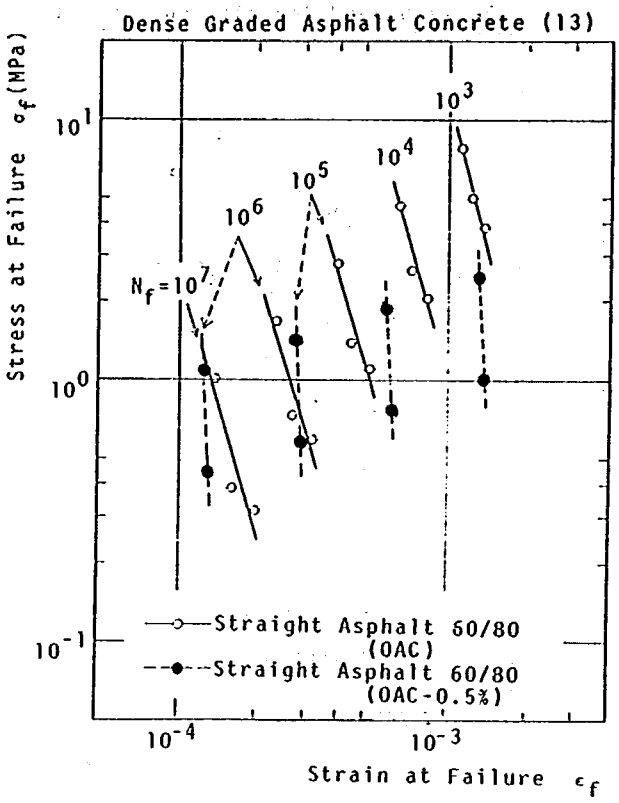

Figure 4a. Fatigue Failure Envelope

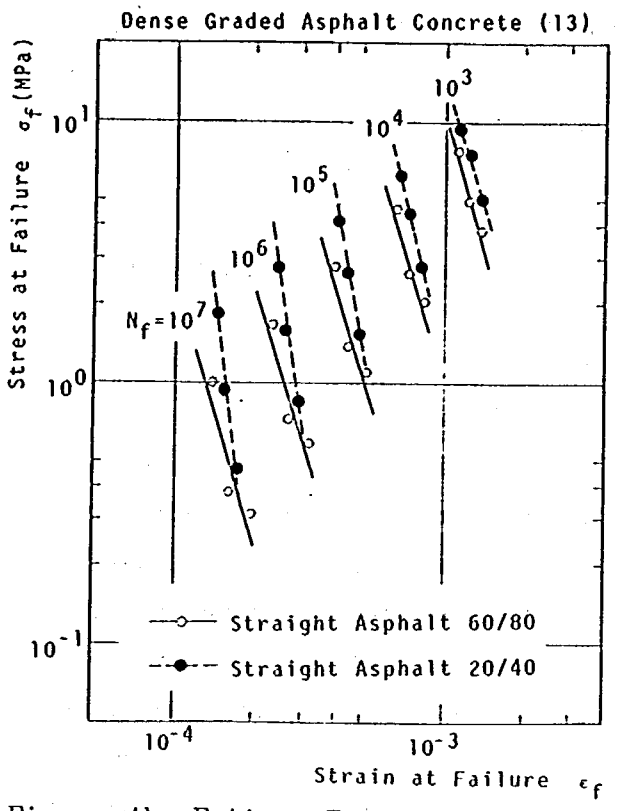

Figure 4b. Fatigue Failure Envelope

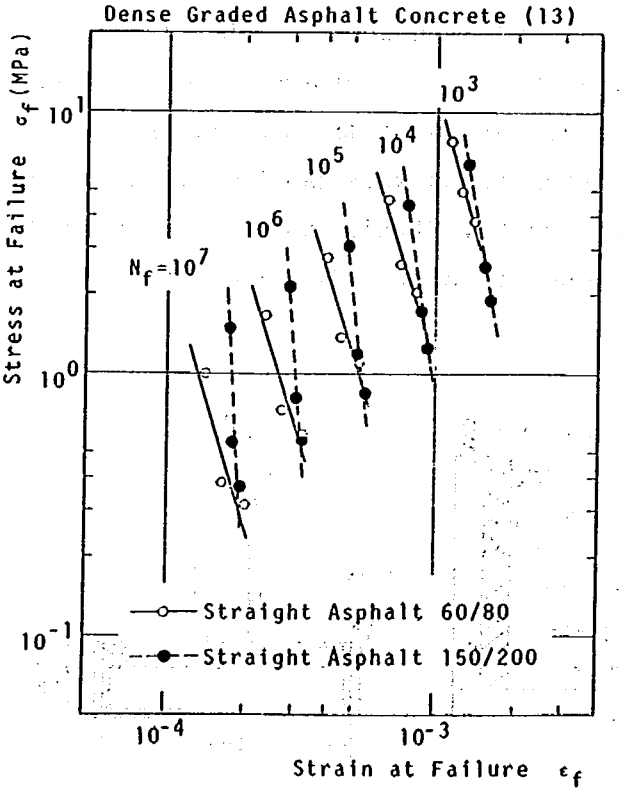

Figure 4c. Fatigue Failure Envelope

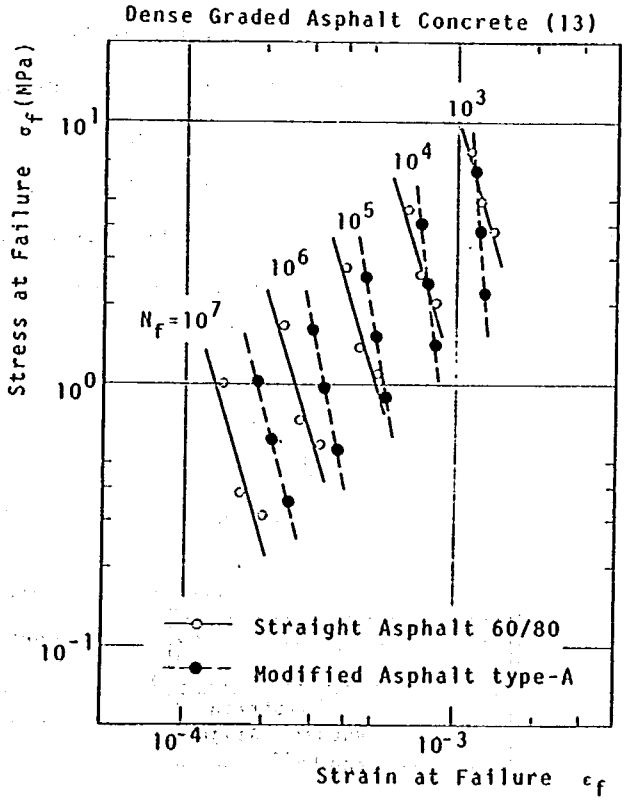

Figure 4d. Fatigue Failure Envelope

the range where N_f is over 10^6, some data were extrapolated on the basis of Pell's experimental results[19] which showed that the $N_f - \sigma_f$ and $N_f - \varepsilon_f$ relations in Figure 2 have linear characteristics.

(3) Simulation Model by a Wheel Tracking Testing Device

It is considered that in general the farther the group of curved lines are located from both axes in the failure envelope diagram, the stronger the fatigue resistance of the material is. In the real pavements, however, fatigue failure cracks grow roughly in four stages: generation of hairline cracks, propagation of them, generation of real cracks, and combination of them[16]. And the state of stress and strain associated with wheel loads is much different from that of laboratory bending tests. In order to assess these differences and to identify the failure criterion of the fatigue failure envelope with real pavement, wheel tracking tests were performed under the conditions close to the actual ones (Figure 5). Strain gauges were bonded onto the bottom surface of an asphalt concrete slab which was set on a

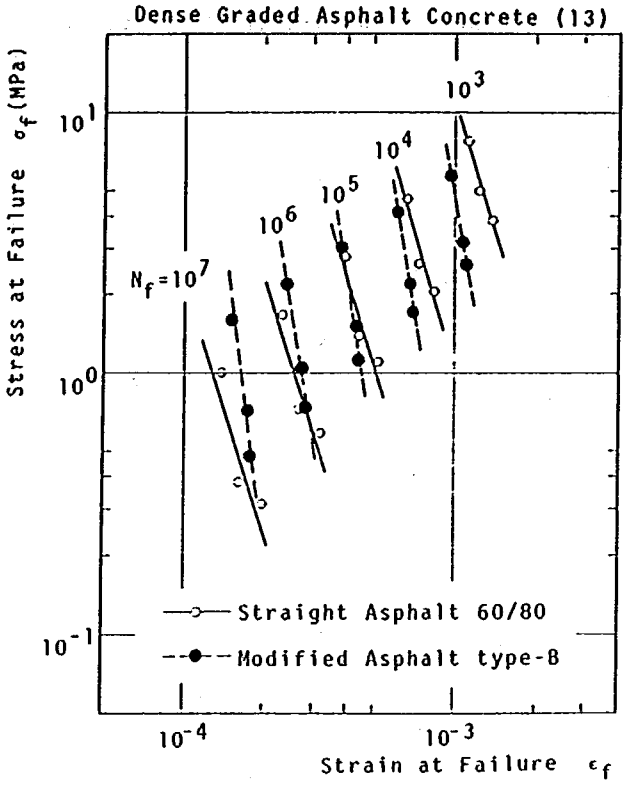

Figure 4e. Fatigue Failure Envelope

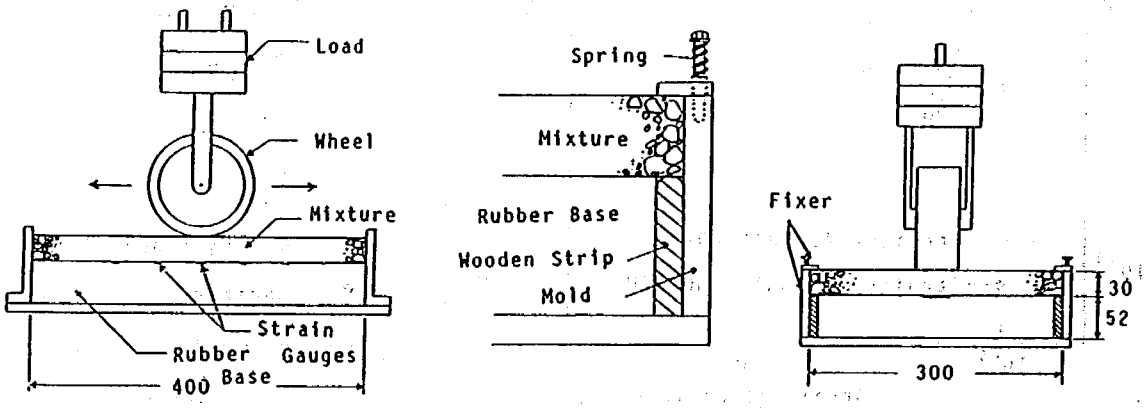

Figure 5. Wheel Tracking Test Equipment

Table 2. Wheel Tracking Test Condition

Dimension of Asphalt Mix	L 40 cm x W 30 cm x H 3 cm
Thickness of Rubber Base	5.2 cm
Elastic Modulus of Rubber Base	24.5 MPa (20°C)
Tyre Type	Solid Tyre (D 20 cm x W 5 cm)
Wheel Load	986 N
Contact Pressure	920 kPa
Tracking Speed	42 pass/min. (Sinusoidal)
Tracking Stroke	35 cm
Temperature	20°C

rubber base, the material of which was so selected that the generated tensile strain would be within the range of 200-1000 micro about the same strains in actual pavement. The details of the test conditions are given in Table 2. While the test was being carried out, the strains were measured along with the number of load repetitions and the process of the generation and propagation of cracks. The test results showed that only the tensile strain generated at the bottom surface of the asphalt slab was observed (Figure 6), whereas in case of the laboratory test the strain changed alternatively from tension to compression. In real pavement, only the tensile

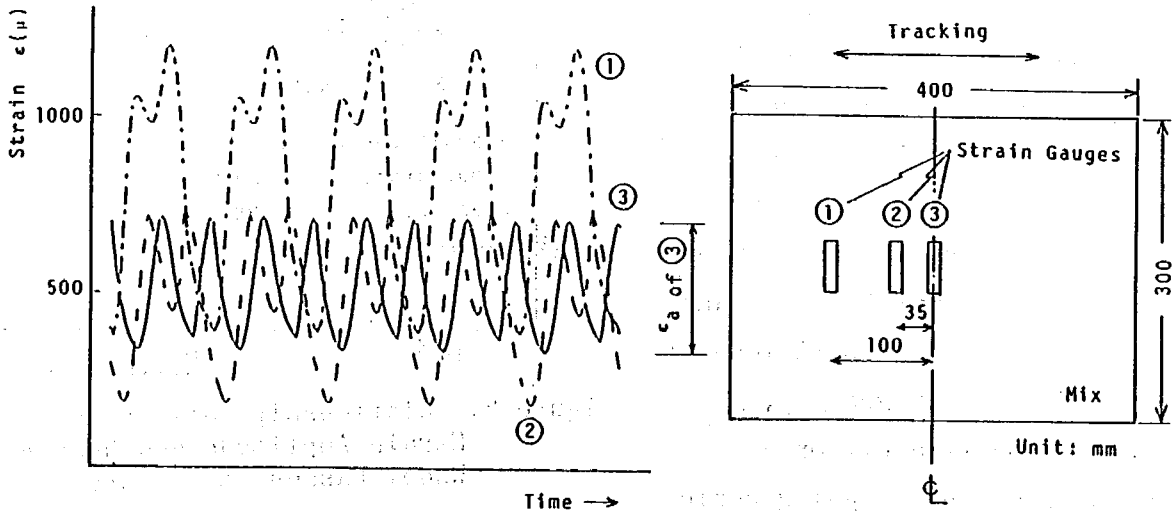

Figure 6. Measured Strains at Bottom of Slab

strain will be generated at the bottom of slab, when the wheel load passes over it. The difference between the maximum and the minimum strain in a load cycle is defined as the (double) strain amplitude and designated as ϵ_a. Strain amplitude increased gradually, with the increase of load repetitions as Figure 7 shows, and the gradient suddenly increased at N_{f1} then it dropped suddenly after its peak point at N_{f2}. As Dijk[16] observed, it was found that some hairline cracks at the bottom surface were generated at N_{f1} and these cracks developed into alligater cracks at N_{f2} which penetrated to the upper surface of the mixture. Thus it is considered that the number of load repetitions N_{f1} corresponds with the generation of cracks and N_{f2} corresponds with the structural failure of the pavements. And it was also found that the relations between initial strain amplitude ϵ_{ai} and the number of load repetitions at the two failure stages N_{f1}, N_{f2} are both approximately on a straight line on a log-log scale (Figure 8). This fact confirms the experimental results obtained from conventional laboratory bending fatigue tests.

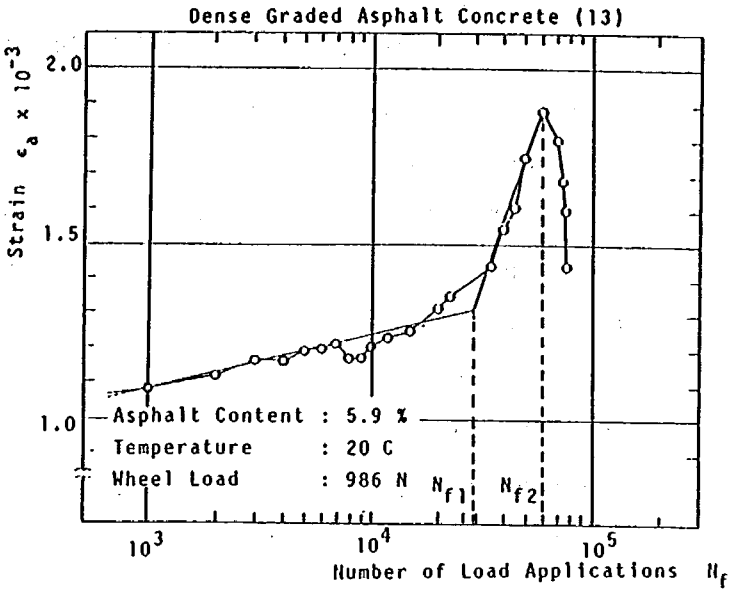

Figure 7. Change of the Strain Amplitude at Bottom of Slab

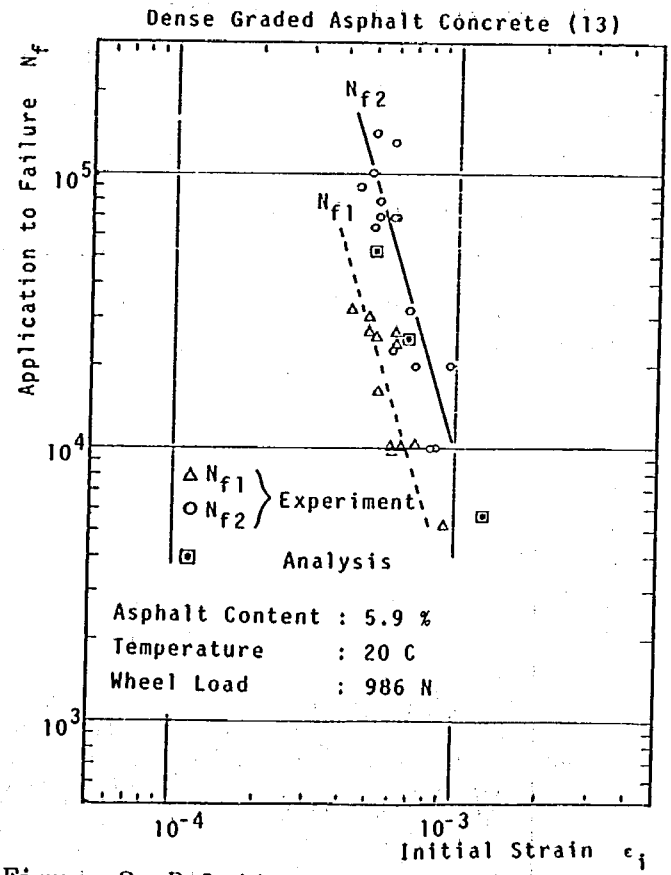

Figure 8. Relationship between Initial Strain Amplitude and Number of Wheel Passes at Failure

Fatigue life in case of the wheel tracking equipment was estimated (Figure

8) from the calculated tensile stress and the measured tensile strain at the bottom of the slab, since these stress and strain gave both ordinate and abscissa of the fatigue failure envelope (Figure 4). The failure condition in case of the wheel tracking equipment could be similar to that of the actual road condition whereas the fatigue failure envelope which was applied to estimate the life was obtained by laboratory repeated bending test different from the actual condition. The number of load repetitions at the actual failure happened in the slab was compared to the estimated values. The results indicate that the failure criterion of the fatigue failure envelope corresponds to the load repetitions N_{f2} which represents the structural failure of the pavements better than N_{f1} which represents the hairline cracks generation as square marks in Figure 8 shown.

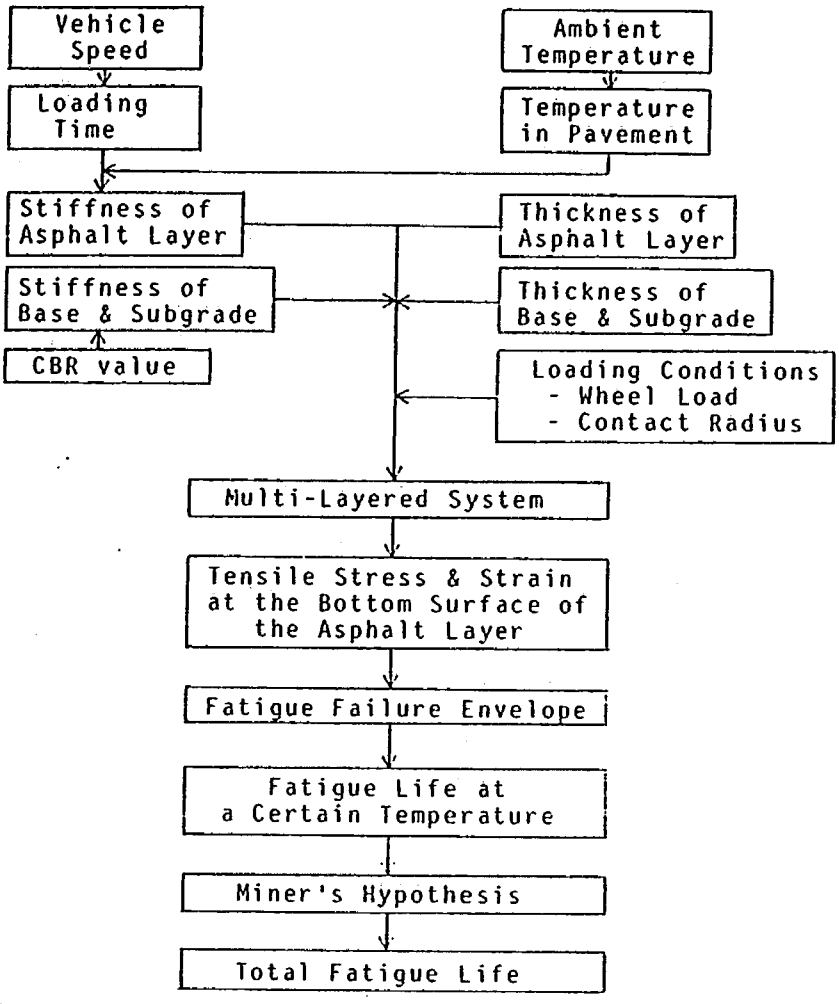

Figure 9. Flow Chart of Fatigue Life Prediction of Asphalt Pavements

3. FATIGUE LIFE OF ASPHALT PAVEMENTS

(1) Prediction System of Fatigue Life

Yoshiwara[22] proposed a fatigue life prediction system of asphalt pavements by the fatigue failure envelope and verified its validity with the investigated reports of Meishin Expressway. This prediction system was modified later by Yamada[23]. The stiffness of asphalt mixture was estimated by the results of bending fatigue test and varying the stiffness depending on the

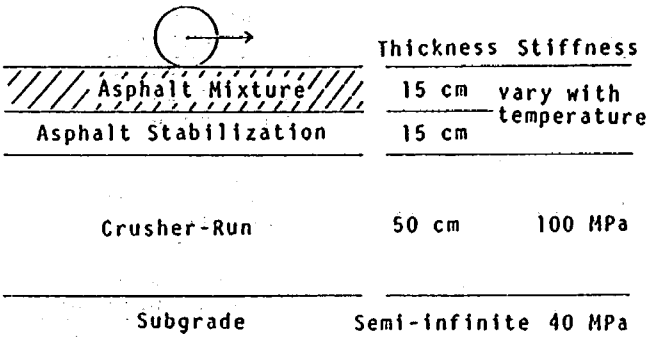

Figure 10. Analized Pavement Structure

Table 3. Analized Condition

Wheel Load (P)	: 49 kN
Radius of Contact (a)	: 17.0 cm
Contact Pressure (p)	: 540 kPa
Vehicle Speed (v)	: 11.1 m/s
Location	: Tokyo

weather condition and the amount of load, the fatigue life corresponding to each month was estimated by stress and strain obtained from multi-layered elastic system analysis adopting fatigue failure envelope. The monthly damages were accumulated by Miner's hypothesis and the flow chart of this system is shown in Figure 9.

(2) Effect of Binder Properties

Since the mechanical properties of asphalt mixtures are affected considerably by the binder type and its content, the fatigue life of asphalt pavement is considered to be dependent on them. A case study was done according

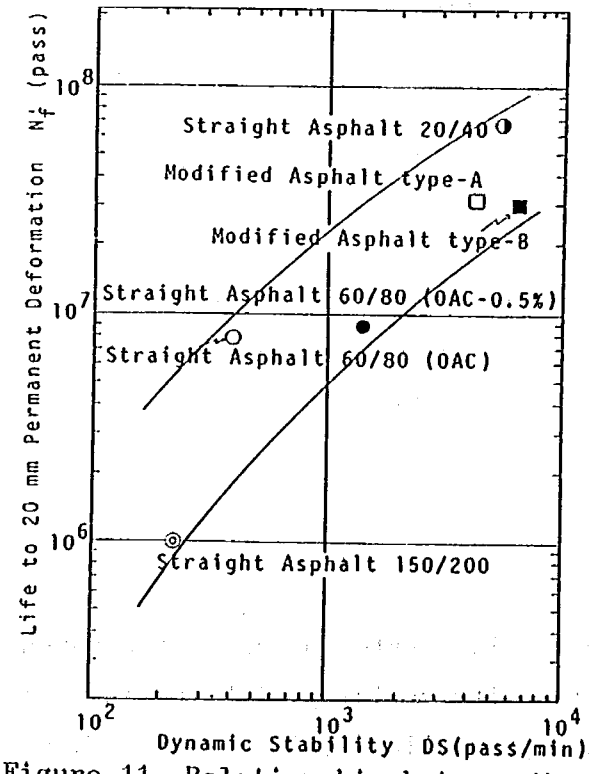

Figure 11. Relationship between Measured Dynamic Stability (DS) and the Estimated Life due to Rutting

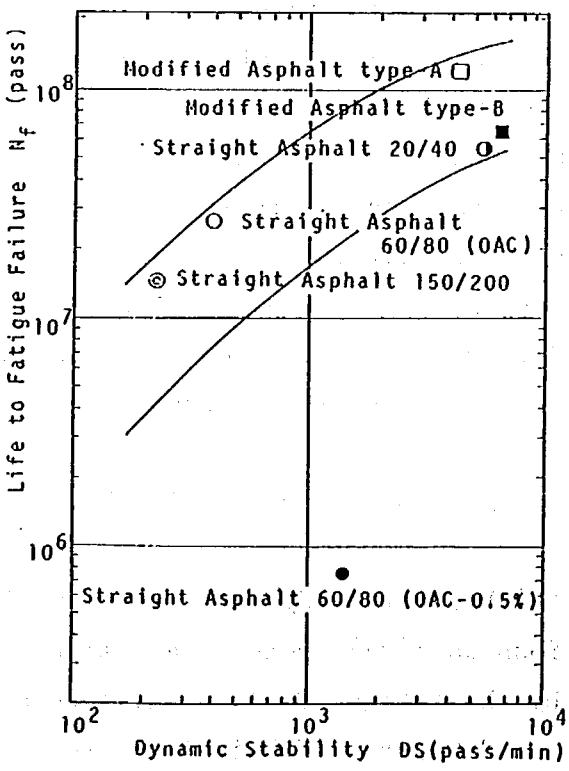

Figure 12. Relationship between Measured Dynamic Stability (DS) and the Estimated Fatigue Life

to Figure 9 to clarify the effect of binder type on the fatigue life of asphalt pavement. The pavement structure, the binder type of the surface course and the analysis conditions are shown in Figure 10, Table 1, and Table 3 respectively. And also the life in which the rutting depth reaches 20 mm (which will be described simply as "the life due to rutting") was predicted to compare with fatigue life following Claessen's method [25] and to investigate the effects of high viscosity and low content of asphalt on the life due to rutting. Dynamic Stability (DS), which was verified to correlated to the flow resistance[24], was measured by wheel tracking test according to the Japanese "Manual for Design and Construction of Asphalt Pavement (1980)."

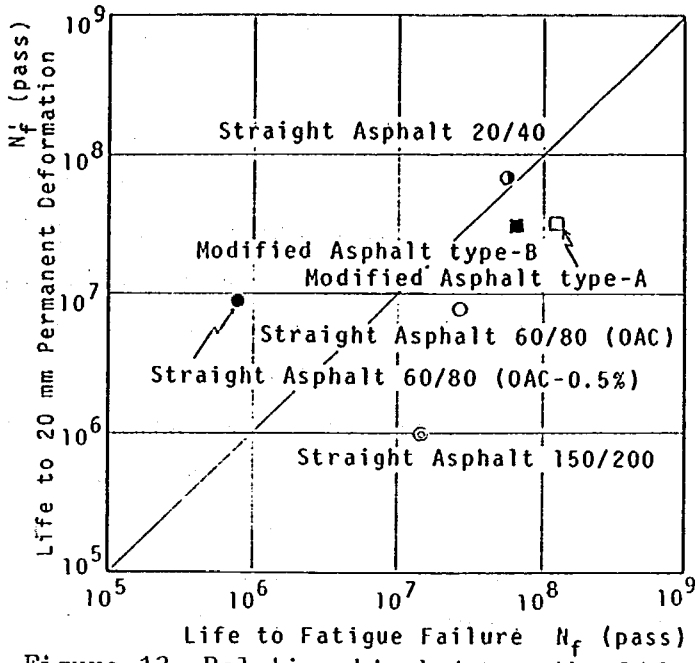

Figure 13. Relationship between the Life due to Rutting and the Fatigue Life of Different Mixtures

The results of the study suggest that DS has a large correlation to the life of the pavement due to rutting and also the results indicate that increasing viscosity and lessening asphalt content are effective in inhibiting the progression of rutting. However, to lessen asphalt content has an adverse effect on fatigue, whereas to increase viscosity (by using high viscous asphalt and/or by additive agent ----- a polymer with high molecular weight) has a similar tendency toward rutting (Figure 12).

By eliminating DS from Figure 11 and Figure 12, Figure 13 can be obtained showing directly the relationship between the fatigue life and the life due to rutting of the pavement. Assuming that the cause of pavement failure is only by fatigue and rutting, and also the life depends on the smaller one of these, it can be indicated that using a high viscous asphalt and/or adding a polymer with high molecular weight to base straight asphalt of 60/80 would increase the life of the pavement, whereas using a low viscous asphalt and/or lessening binder content would decrease it, comparing the above with the case when using straight asphalt of penetration class 60/80 for the binder of asphalt mixture.

4. EVALUATION OF EXTERNAL FORCES ON PAVEMENTS

The fatigue cracks in asphalt pavement occur due to the generation of stress and strain in the asphalt mixture due to wheel load repetitions. The fatigue damage of the pavement caused by the passage of a vehicle depends on its wheel load, transverse wheel positions, and vehicle velocity. It also depends on the fatigue resistance of asphalt mixture and its stiffness modulus which vary with the natural conditions of the asphalt mixture. In the fatigue life prediction method using the fatigue failure envelope (Figure 9), which was stated previously, wheel loads are converted to five tons on basis of AASHO road test results, and also transverse wheel positions, vehicle velocity, and fatigue resistance of asphalt mixture are assumed to be constant. Also the stiffness of asphalt mixture is evaluated by monthly ambient air temperature. All these assumptions are considered to be much different from the real pavement conditions. In this study, national route 357 (Tokyo Bay Ring Road) Shinonome section was selected as a test section. All the variables were estimated as stochastic variables on basis of laboratory experiments and field investigative results, and consequently the damage of the pavement due to the passage of a vehicle was also evaluated in a probabilistical way.

(1) External Forces on Pavements

a) Wheel Load: The statistical distribution of wheel loads was obtained on basis of measured results by a truck scale installed in the section which was performed by Kawasaki National Highway Work Office (Kantoh Regional Construction Bureau, Ministry of Construction). A significant difference was not found in the results of measurements for 24 hours in July and October, the average value was used for the analysis (Figure 14). Since the fatigue damage of the pavement caused by the passage of a vehicle is proportional to its wheel load to 3rd-6th power[9], all wheels

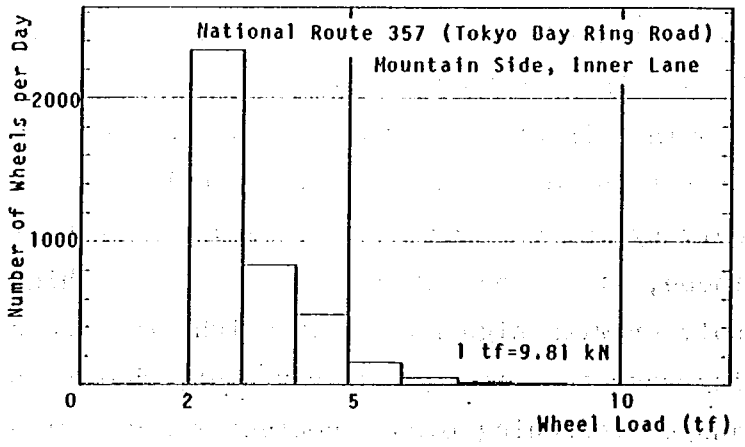

Figure 14. Distribution of Wheel Loads (Disregarded less than two tons)

weights less than two tons were ignored in order to shorten the time required for computations.

b) Wheel Position: The statistical distribution of the transverse wheel positions is an important factor for the pavement distresses, since the stress and strain distribution induced in asphalt mixture layer due to a vehicle depends on its wheel position significantly. In this study, the transverse wheel positions from the center line of the street were investigated by means of video tape recorder (VTR). However, the proper observing location to set the VTR camera was not found at the site, so that the statistical distribution was estimated from the results of preliminary investigations performed at Nakahara Street (Figure 15).

c) Vehicle Speed: Fatigue cracks of asphalt pavements are not affected by vehicle speed directly, but since the stiffness modulus varies with loading time of external forces besides their temperature, the internal distribution of stress and strain changes according to it, and consequently it has an effect on

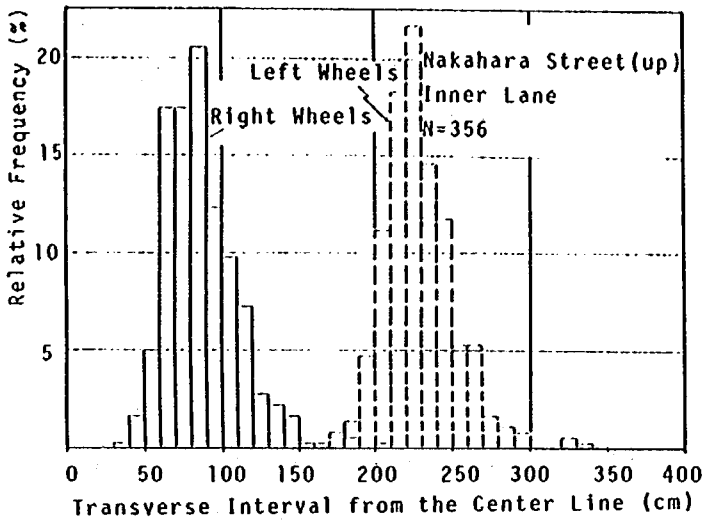

Figure 15. Distribution of Transverse Wheel Positions

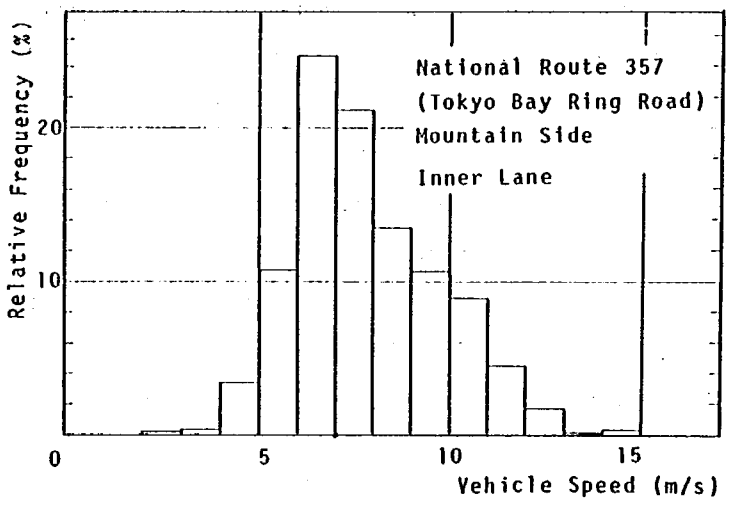

Figure 16. Distribution of Vehicle Speeds

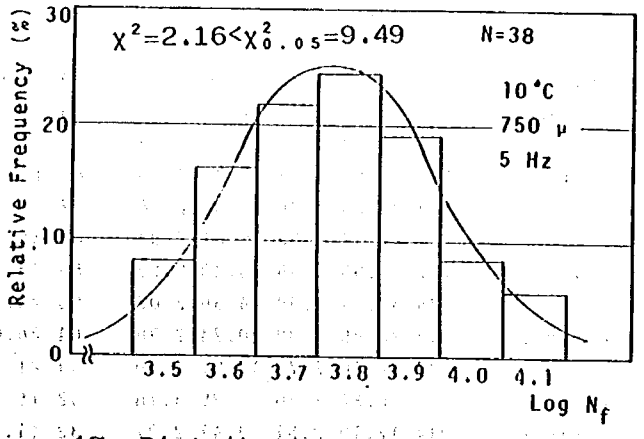

Figure 17. Distribution of Fatigue Resistances of an Asphalt Mixture

the fatigue life of asphalt pavements. These measurements were also obtained using the VTR. Time headway of each vehicle was obtained at two points separated by some distance by means of the clock incorporated in the micro computer (FM-11, Fujitsu), then it was converted to vehicle speed.

(2) Strength of Pavements

a) Fatigue Resistance: Fatigue resistance of asphalt mixtures scatters to some extent even if they are subjected to the same stress and strain under the same conditions. It was reported by Pell[6] that fatigue resistance follows a lognormal distribution through controlled stress testing by using rotating bending cantilever machines. This fact was also confirmed in this study by means of controlled strain bending fatigue test (Figure 17).

b) Stiffness: The stiffness modulus of asphalt concrete varies with loading time of external forces and with the temperature of the mixture, hence the stress and strain generated in the pavement is always changing even though the applied load is constant. The loading time was evaluated using the vehicles speeds, and also the temperature was estimated by means of the theoretical equation of thermal conductivity combined with Akiyama's formula[26] which yields the surface temperature of the pavements from the ambient air temperature. The thermal diffusivity of the mixture, which appears in the equation, was obtained from the results of laboratory experiments. The distribution of ambient air temperature was determined by referring to the weather data for the period between 1973 and 1981[27] (Table 4).

Table 4. Distribution of Monthly Maximum and Minimum Daily Air Temperature by Weather conditions in Tokyo

	Fine					Cloudy					Rainy				
		Maximum		Minimum			Maximum		Minimum			Maximum		Minimum	
	N	AV	SD	AV	SD	N	AV	SD	AV	SD	N	AV	SD	AV	SD
January	214	9.74	2.50	0.90	1.81	30	8.03	2.89	2.14	1.99	35	9.05	4.23	2.74	2.45
February	176	10.19	2.98	1.39	2.42	28	9.51	3.35	3.20	2.47	50	9.23	3.94	3.21	2.52
March	156	13.53	3.12	4.25	2.49	52	11.74	3.45	5.11	2.74	71	11.84	4.13	5.40	2.66
April	118	19.56	3.08	9.91	3.00	60	17.77	3.52	11.04	3.13	92	17.06	4.62	10.18	3.27
May	139	23.86	2.36	14.34	2.26	57	22.25	3.22	15.47	1.93	83	20.75	3.27	14.62	3.01
June	79	27.57	2.65	19.81	2.87	80	25.08	3.15	18.80	2.67	111	24.13	2.91	18.84	2.31
July	127	31.05	2.35	23.48	2.53	66	28.48	2.95	22.20	2.60	86	26.41	3.68	20.79	2.40
August	136	32.09	1.99	24.56	2.02	56	28.98	2.73	22.72	2.17	87	28.47	3.54	22.55	2.28
September	113	28.85	2.88	20.74	2.98	64	26.03	2.90	20.24	2.40	93	24.35	3.59	19.49	2.54
October	137	22.40	2.70	13.91	2.60	59	21.53	2.69	15.51	2.66	83	20.24	3.34	15.08	2.54
November	161	16.88	3.20	8.29	3.18	52	15.66	3.32	9.78	3.06	57	15.50	3.59	10.25	2.35
Decrmber	214	12.10	2.51	3.41	2.21	32	11.80	3.12	5.15	1.90	33	11.37	3.67	5.35	2.52

(Note: N: Number of the Days, AV: Average, SD: Standard Deviation, Unit: °C)

(3) Method and Results of the Analysis

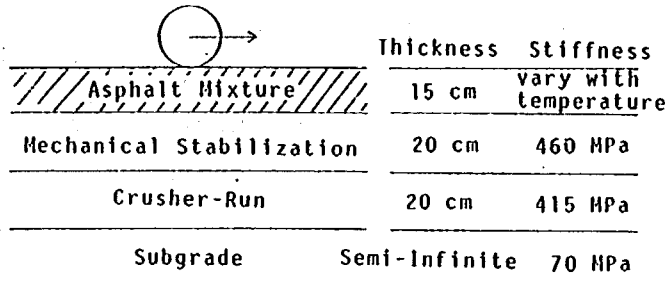

Figure 18. Pavement Structure for simulation analysis

The pavement structure chosen is shown in Figure 18. The simulation was performed based on Monte Carlo method by means of the supersized electronic computer (M-280, Hitachi). The flow of the analysis is as follows: Firstly, the hourly temperature in the bituminous pavement was estimated based on the maximum and the minimum air temperatures for the day concerned which were determined by means of pseudo random numbers generated according to its distribution. Secondly, the wheel load, the vehicle speed, and the wheel position were also determined from pseudo random numbers according to their measured distributions. Thirdly, stress and strain at the bottom surface of the asphalt mixture were computed by means of multi-layered elastic theory. Then the fatigue damage due to one car passage was evaluated using Figure 19. This figure shows the fatigue failure envelope of the recycled asphalt mixture used for the binder course of the pavement which was obtained from the laboratory fatigue test performed on the material collected from the construction field. Fourthly, the damage was computed by one wheel pass which should be a definite value when a environmental and loading conditions were given. However, the amount of damage at one set of conditions should be scattered to an extent due to the fluctuation of the material properties (Figure 17). The effect of this material fluctuation was simulated stochastically in the damage computation. The distribution of the daily fatigue damages were determined for every month by repeating the procedure mentioned above. The whole process was simulated from April 1983, when the street was opened to traffic, until the total damage sums up to 1.0 by Miner's rule ----- until the pavement fails.

In summer when the temperature was high in the bituminous pavement, the calculated theoretical damage was extremely large in some cases. However, when the wheel tracking fatigue test, as mentioned in 2. (3), was performed at temperature higher than 30 C, it was found that measured strain amplitude did not increase with load repetitions and no cracks appeared at all. So July and August were not included because of high temperature in the analysis according to Celard's proposal[28]. The number of load repetitions to

failure N_f was expressed as a cubic equation of stress σ_f and strain ε_f at failure for numerical calculation instead of graphical reading from Figure 19. The statistical distribution of the fatigue life of the pavement was obtained by repeating the simulation analysis mentioned above (Figure 20). The final life of the pavement considering statistical factors was found to follow the lognormal distribution similar to the fatigue resistance of asphalt mixture. And the probability of pavement fatigue failure was obtained by integrating this statistical distribution (Figure 21). Thus the life of the pavement should be expressed by means of failure probability, considering external force action, climate conditions, and the scattering of the material strength. Then it would be possible to make a rational plan of the management system according to the importance of the roads, for example a trunk road should have less probability of failure than a minor road. Defining the failure point as the time when the failure probability reached 50 %, the predicted life of the pavement became approximately three times longer than the conventional mean value method, whereas the predicted life was almost the same when the distribution of the transverse wheel positions

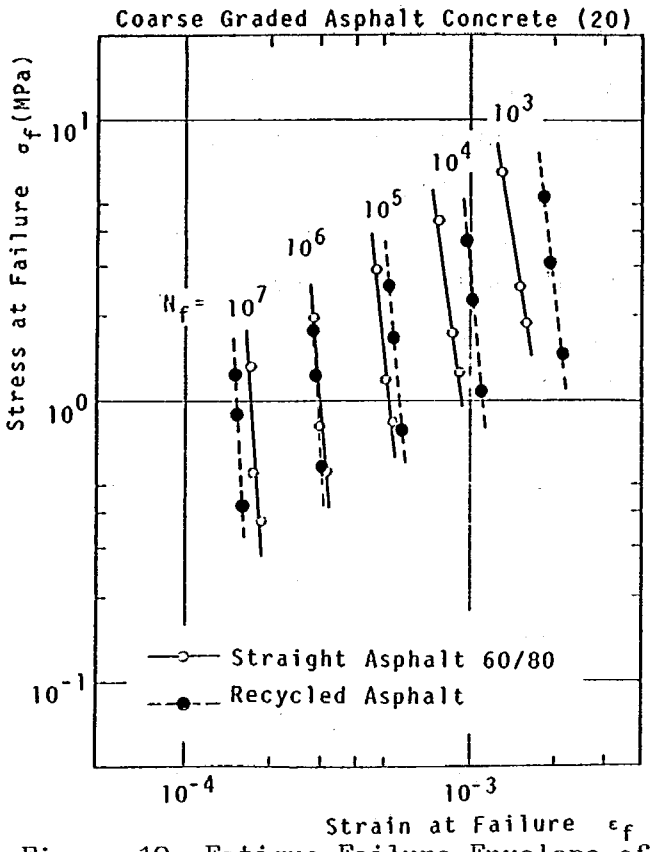

Figure 19. Fatigue Failure Envelope of Fresh and Recycled Asphalt Mixture

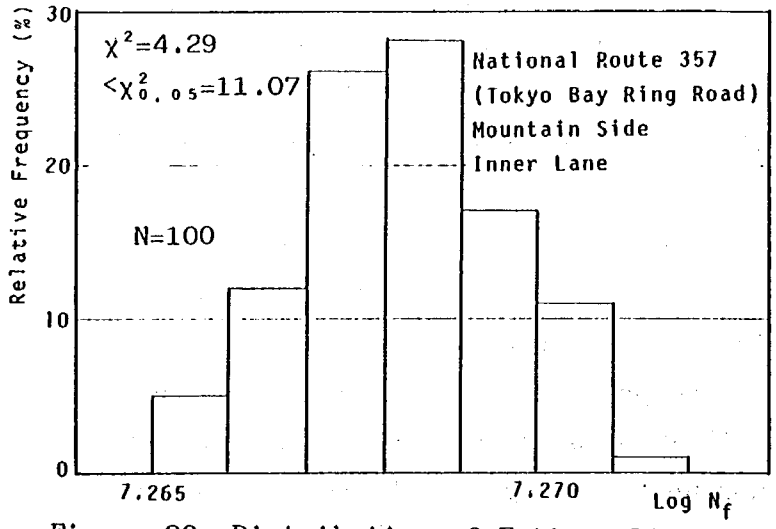

Figure 20. Distribution of Fatigue Lives of an Asphalt Pavement

were disregarded (Table 5). Consequently, it was found that the transverse wheel position was an important factor not only for rutting but also for fatigue life.

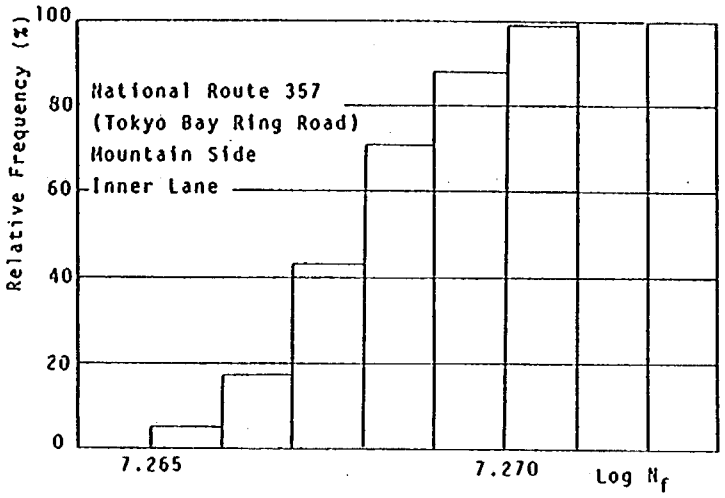

Figure 21. Failure Probability of an Asphalt Pavement

5. CONCLUSIONS

The results obtained in this study are as follows:

(1) The failure criterion by the fatigue failure envelope is not corresponding to the point when hairline cracks appear but to the point when real cracks appear on the pavement and the pavement loses its bearing capacity.

(2) Adding an adequate additive agent to the binder to increase its viscosity against permanent deformation would have a good effect on the pavement fatigue life, whereas decreasing binder content from optimum gives shorter fatigue life.

(3) The fatigue life of the pavement can be expressed not as a constant value but as a probabilistic variable which follows the lognormal distribution.

(4) The transverse distribution of wheel position is an important factor for the pavement fatigue life.

Table 5. Result of Simulation Analysis

Predicted Load Repetitions to Failure	Conditions
1.9×10^7 (8.7 years)	All variables are considered as random variables.
4.2×10^6 (2.0 years)	All variables are considered as random variables except transverse distribution of wheel positions.
3.4×10^6 (2.7 years)*	Conventional method

* Number of equivalent 5 ton wheels

ACKNOWLEDGEMENTS

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Cold Weather Influences on the Airport Pavements and Management Practices

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COLD WEATHER INFLUENCES ON AIRPORT PAVEMENTS AND MANAGEMENT PRACTICES

ABSTRACT

The Canadian Air Transportation Administration operates the national system of airports in Canada. Airport pavement facilities are provided in climatic environments ranging from frost free areas on the west coast to the permafrost areas of northern Canada. As a result, the Administration's technical practices for the management of airfield pavements must address a wide range of cold weather effects.

Airport pavement operational characteristics of most concern are bearing strength, roughness, skid resistance and structural integrity. The effects of cold weather on these operational characteristics are discussed along with the management practices which have evolved in response to these effects. A brief discussion is also given on pavement restoration methods and on the provision of airport pavements in northern Canada.

COLD WEATHER INFLUENCES ON AIRPORT PAVEMENTS AND MANAGEMENT PRACTICES

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1. INTRODUCTION

The Canadian Air Transportation Administration (CATA) of the Federal Department of Transport operates an extensive system of airports in Canada. One of the major objectives of technical management practices for the airfield pavements at these airports is to ensure safe operating conditions for aircraft with respect to pavement bearing strength, roughness, skid resistance and structural integrity. The major influences of cold weather on these operational characterstics, and on associated CATA management practices, are summarized in this paper. Cold weather is the most significant and pervasive of the many physical factors affecting Canadian airport pavement performance and life-cycle management practices.

2. THE CATA AIRPORT SYSTEM

CATA has regulatory responsibilities for all aerodromes in Canada. For historical and geographical reasons, the Administration is also directly involved in airport construction and operaton. With few exceptions, CATA owns and operates all major airports which support the Canadian commercial air transportation system. In addition, CATA is very active in airport construction in remote areas of the country for purposes of access and development.

The geographical distribution of CATA airports covers all of Canada (Figure 1) and cold weather severity experienced throughout the system ranges from negligible to extreme (Figure 2). Limited areas on the west coast are frost free, and seasonal frost occurs throughout southern Canada. Discontinuous and continuous permafrost conditions are encountered in the mid and far north (Figure 3). This climatic envi-

ronment and the variety of soil and construction conditions thoughout Canada have resulted in experiences representing most types and degrees of cold weather effects on pavements.

The CATA airport system currently includes 205 airports of which 106 are operated directly by CATA and the remainder are subsidized. Many other airports have been constructed through CATA capital grants, but no continuing responsibilities exist for facilities management at these sites. The inventory of airfield pavement in the CATA airport system is 38.8 million square metres, of which 63% is asphalt surfaced and 24% is concrete surfaced. The remaining 13% is mostly gravel surfaced pavement at remote airstrips. The average service life achieved is close to 20 years for original asphalt surfaces, 26 years for original concrete surfaces, and 15 years for asphalt resurfacing overlays (Figure 4).

3. BEARING STRENGTH

Cold Weather Effects

Seasonal frost penetrations cause substantial variation in the bearing strength of pavements. The lowest values of bearing strength occur during a two to four week period during spring thaw, and the recovery to normal summer and fall strength levels is usually not fully complete until July. In most instances, and particularily for asphalt pavements, bearing strength is of little concern during the four to six months over the winter period when pavement structures are frozen.

In some circumstances, winter frost heaving may cause problems with the bearing strength of concrete pavements. These pavements are designed on the assumption of uniform bearing support for the concrete slab, and highly localized frost movements may leave portions of the slab without support. In such cases, heavy traffic soon results in fractured slabs. However, the countermeasure to this effect lies not in structural design practices, but in the provision of pavement frost protection.

Design and Evaluation

For the design and evaluation of airport pavement bearing strength, CATA utilizes a series of twelve standard aircraft loadings which span the range of modern aircraft loadings. Flexible pavement structural thickness requirements for these standard loadings are a function of subgrade bearing strength (Figure 5), and rigid pavement slab thickness requirements are a function of the bearing modulus of the surface beneath the slab (Figure 6).

The structural design of new pavements at an airport is based on subgrade bearing strengths obtained from previous testing programs on existing pavements at the site. In the case of new sites, conservative values of subgrade bearing strength are estimated on the basis of the subgrade soil classification (Figure 7). Field measurement of actual bearing strength is conducted two or more years following construction.

The bearing strength of airport pavements is reported in terms of the standard design loading for which the pavement provides sufficient bearing strength, as determined from the design requirements given in Figures 5 and 6. This reported value is referred to as the pavement load rating. Load ratings for actual aircraft have been published, and the evaluation of a proposed aircraft operation simply requires a comparison of the aircraft load rating to the load rating reported for the pavement.

Bearing Strength Measurement

CATA measures the bearing strength of flexible airport pavements by repetitive static plate load testing (ASTM D-1195). Field testing programs are non-destructive in nature with plate load tests conducted only on the surface of the pavement. Subgrade bearing strengths are calculated from the results of surface tests using relationships developed during extensive field studies conducted from 1940 to 1960. The bearing strength of concrete pavements is normally not measured

directly, but is determined from known slab thickness and a bearing modulus calculated from the subgrade bearing strength of adjacent flexible pavements.

CATA practices for the structural design and evaluation of airport pavements are based on subgrade bearing strengths which prevail during the spring thaw period. Bearing strength measurements are carried out during the summer and fall months, and a reduction factor must be applied to these measurements to estimate spring strength levels. The reduction factor applied depends primarily on the classification of the subgrade soil (Figure 7). The representative spring and fall bearing strength values given in Figure 7, if used in conjunction with Figure 5, give some appreciation of the effect which the spring reduction in bearing strength has on flexible pavement load ratings and thickness requirements.

CATA practices for the design and evaluation of airfield pavement bearing strength have been established for some years. Little need has arisen for changes because instances of failure due to inadequate bearing strength are rare. When load associated failures occur, other factors are usually the principal cause and the construction of thicker pavements for purposes of greater bearing strength would not have prevented the failure.

4. ROUGHNESS

Cold Weather Effects

Seasonal frost penetration into frost susceptible subgrade soils will produce ice lensing in the frozen zone, and heaving at the pavement surface. When subgrade conditions are not uniform, the surface movement is differential in nature and the difference in surface roughness characteristics from summer to winter can be substantial. Following spring thaw and resettlement during the summer, some of the differential movement is often retained and there is a slow accumulation of roughness as the pavement ages.

Highly localized frost bumps can result from the raising of a boulder embedded in a fine grained subgrade soil, or from drainage patterns feeding a concentrated flow of water to a subsurface freezing zone. Frost bumps resulting from this latter cause can appear and disappear within a few days during the spring thaw when an abundance of surface water exists.

At pavement edges, water reaching the pavement subgrade from the shoulders may produce heaving at the pavement edge which is significantly greater than heaving of the pavement interior. Differential pavement edge heaving is detrimental to lateral surface drainage and may produce a longitudinal crack in pavement surfaces about 2 or 3 metres in from the pavement edge.

Another form of differential frost heaving is represented by the raising of manholes and catchbasins above the pavement surface. In addition to having an adverse effect on drainage, raised manhole tops are a safety hazard for snow plow operators. Frost heaving of manholes and catchbasins also tends to shear off the connecting subsurface drainage pipe.

Roughness Measurements

The roughness levels on runways at major CATA airports have been monitored at two year intervals for the past twenty years and roughness levels are reported in terms of a Riding Comfort Index (RCI) on a scale of 0 to 10. On the average, the RCI of original asphalt surfaces decreases slowly during the first few years and this decrease probably reflects roughness accumulated from differential frost movements (Figure 8). Eventually, however, roughness begins to accumulate at an accelerating rate as surface breakdown commences. Original concrete surfaces seem to accumulate frost related roughness uniformly with age with an average loss of one RCI unit in about 15 years. Asphalt overlays develop roughness significantly more quickly than original asphalt surfaces, probably as a result of reflection cracking.

Roughness is currently measured with a Wisconsin type vehicle response meter. However, some of our major roughness problems occur during the winter months and vehicle response meters are not entirely suitable for measuring roughness during this part of the year because of the effect of freezing temperatures on the suspension system of the test vehicle. The investigation of runway roughness during winter months must usually rely on the measurement and analysis of runway profile elevations.

Frost Protection Requirements

CATA design practice for asphalt and concrete pavements constructed on frost susceptible subgrade soils requires that a minimum thickness of pavement be provided for frost protection purposes. This minimum thickness for frost protection is a function of site average freezing index (Figure 9) and it varies from the average depth of frost penetration in milder climates, to about one-third of the average depth of frost penetration in colder climates. This frost protection requirement dictates the thickness of granular base and sub-base placed under concrete slabs. In the case of asphalt pavements designed for moderate to light aircraft loadings, minimum frost protection requirements usually exceed structural thickness requirements.

A comparison of the average service lives shown in Figure 4 to the average roughness performance curves given in Figure 8 reveals that at the time of restoration, the average pavement has not yet reached unacceptable levels of roughness. Only ten to twenty percent of restoration projects are undertaken because of excessive roughness.

5. SKID RESISTANCE

Winter Operations

Runway skid resistance reaches critical states under winter snow and ice conditions and there are periods when continuous monitoring is necessary as an advisory service to pilots. All CATA airports serving scheduled jet aircraft traffic use a Tapley meter to monitor the skid

resistance of snow and ice covered runways. The Tapley meter is a simple deceleration measurement device which is installed in a maintenance vehicle and which records the coefficient of friction developed when the vehicle is braked.

Snow and ice control and removal practices have a number of indirect implications with respect to pavement structural performance. Sweepers used in conjuction with plows and blowers for snow removal have steel bristles. Hard, durable surfaces are required to withstand sweeping operations and partially for this reason, CATA has avoided the use of surface treatments on airfield pavements. Urea is used as an ice control chemical on airport pavements rather than salt because of the corrosive effects of salt on aircraft. While urea is non-corrosive with respect to aircraft metals, laboratory investigations suggest that the chemical may promote aggregate degradation. Another chemical, glycol, is used for aircraft de-icing and anti-icing. Glycol is not thought to have any major effect on pavement materials, but complaints have been received concerning slipperiness due to the presence of this chemical on apron surfaces. Glycol reaching the drainage system also poses environmental problems.

Summer Operations

The skid resistance of bare runway surfaces is measured under wet conditions with a Saab surface friction test vehicle, and is reported in terms of the runway surface coefficient of friction. The major problem experienced with runway skid resistance under normal wet conditions is caused by heavy rubber deposits on the runway surface in the touchdown zones of heavily trafficked runways. The rubber is deposited during the spin-up of wheels on landing aircraft, and periodically skid resistance may be reduced to a level where removal of the rubber deposits is necessary (Figure 10). Winter snow and ice control operations seem to be benefical with respect to controlling the rate of rubber build-up, and may even assist to some degree in removal.

6. STRUCTURAL INTEGRITY

Cold Weather Effects

In terms of pavement material and structural deterioration, the most common effect of cold weather is the cracking which occurs in asphalt surfaces due to thermal contractions during winter. Thermal cracks form areas of weakness which ultimately break down under traffic loadings, and which may produce differential frost heaving as a result of surface water entering the crack. A related type of surface fracturing is the opening and eventual breakdown of longitudinal paving lane joints. When pavements are restored by the application of an overlay, cracks and joints in the original surface that exhibit thermal movement will reflect through the overlay within one or two years.

Another phenomenon leading to the structural deterioration of asphalt pavements is stripping of the asphalt mix in which water causes loss of bond at the asphalt/aggregate interface. Stripping results in the loss of mix cohesion and tensile strength, and in extreme cases the asphalt mix can revert to an unbound granular state. Stripping most frequently occurs in the bottom layers of the asphalt, and in recent years a number of instances have been encountered where severe stripping occured in the bottom layers of an overlay while the top layer of the overlay and the underlying original asphalt have remained sound. Asphalt stripping is a complex phenomenon with several variables which may affect the process, but laboratory investigations have indicated that freeze-thaw cycles can be a major contributing factor.

Due to the jointing practices followed by CATA, concrete pavements do not in general suffer the thermal cracking experienced in asphalt pavements. However, unless crack control steel is used, misaligned joints and isolated structural cracks are frequently propagated into adjacent slabs by thermal movements of the concrete.

Few concrete pavement joint sealants remain water tight for more than a few years and incompressibles entering an open joint during cold weather can prevent the joint from closing during hot weather. The

result can be an expansion of the pavement as a whole with pressures exerted against adjacent structures. For similar reasons, edge slabs may become permanently displaced.

The disintegration effect of freeze-thaw cycles on concrete with insufficient air voids is well known. With current technology and good quality control practices, this problem is rarely encountered. A possible related problem is 'D' cracking which is manifested by fine patterned cracking which first appears at the corners and edges of affected slabs. It is thought that 'D' cracking is initiated by water penetrating coarse aggregate particles and splitting these particles under freeze-thaw cycles.

FOD Hazard

Thermal cracking in itself does not detract from the operational characteristics of an asphalt pavement. However, it forms an area of weakness which promotes adjacent, load associated secondary cracking that may eventually lead to loose asphalt. Many other forms of environmentally induced deterioration in pavement materials or structure have a similar end result - fracturing, loosening and breakout of pavement surface material under aircraft traffic.

In the airport context, loose material on a pavement surface constitutes a FOD (foreign object damage) hazard. FOD is of particular concern with pavements serving jet aircraft as major damage can result from loose material being injested by an engine. Also, loose material blown by jet blast is a safety hazard to persons or vehicles in the vicinity. Pavement failure and the need for restoration is most frequently determined by the development of a FOD hazard which cannot be controlled through normal maintenance effort.

Condition Evaluation

Formal engineering surveys are conducted annually or biennially at CATA airports to assess pavement structural condition on the basis of visual examination. If warranted by surface manifestations or unex-

pected performance, more detailed engineering investigations may occasionally be carried out, such as the drilling of test holes and the collection of samples for laboratory analysis. A major objective of these structural condition investigations is to estimate remaining service life for restoration programming purposes. While the extent and severity of structural damage from cold weather or other factors can be determined through the condition survey process, little technical guidance is currently available to relate remaining service life to this damage, and restoration programming depends heavily on engineering judgement.

Objective, quantitative methods exist for the measurement and reporting of other operational characteristics; bearing strength in terms of load rating, roughness in terms of RCI and skid resistance in terms of coefficient of friction. During condition surveys, a structural condition rating on a scale of 0 to 10 is assigned to the pavement structure by the survey inspector. This condition rating is dependent on subjective judgement, and significant variations ocurr from one inspector to another. Considering that well over fifty percent of restoration projects are undertaken because of problems related to a deteriorated structural condition, quantitative methods of measurement or more objective methods of assessment are needed.

Design and Construction Practices

The influences of cold weather on the deterioration of pavement materials and structural integrity are combated largely through material specifications and construction quality control practices. The thermal cracking problem has been the subject of considerable research in Canada, and this research has shown that the thermal cracking of original asphalt surfaces may be reduced by the use of specifications which control the temperature susceptibility characteristics of the asphalt cement. Such specifications are still not in standard use in CATA because agreement has not been reached with Canadian industry on appropriate specifications. At present, asphalt cements are specified by penetration grade, and grades of 85/100 to 150/200 are used, depending on the site freezing index.

Many older asphalt pavements at CATA airports have extensive thermal cracking and the materials used to seal these cracks have been the source of problems when an overlay is placed for restoration purposes. These crack sealant materials frequently bleed through the hot overlay asphalt immediately after placement. The instability of the asphalt at the location of bleeding may result in shoving of the mix under a compactor, and a reflection crack can actually be rolled in during construction. As a result of these experiences, cold poured liquid sealant materials used in previous years are being replaced by hot poured rubberized asphalt sealants which will not reliquify at asphalt placing temperatures.

The stripping of asphalt mixes is still a widespread problem. Past practice was to use a Marshall 24-hour immersion test to measure stripping susceptibility during the mix design process, but this practice has not been successful in controlling the problem. Currently, another test procedure is being investigated based on the split tensile strength testing of briquettes conditioned through vacuum saturation followed by freeze-thaw cycling. This latter test method seems promising but several years will be required to correlate test results against actual field performance.

Specification requirements for aggregate hardness and durability are based on Los Angeles abrasion and magnesium sulphate soundness testing. These requirements have worked reasonably well in general, but aggregate related problems are sometimes still encountered, such as concrete pavement 'D' cracking or deterioration through alkali reactions.

Rigid pavements at CATA airports are constructed of plain concrete and thermally induced movements have a major impact on jointing practices for these pavements. To control cracking from thermal contractions and shrinkage during curing, slab widths are 6 metres for slabs with thicknesses less than 300 mm and 7.5 metres for thicker slabs; contraction joints are cut at 6 metre intervals. If shoulders are not paved, a joint is cut along the centerline of the edge slab to control cracking from differential edge heaving. Crack control steel is used

in areas of mismatched joints to prevent thermally induced sympathetic cracking in adjacent slabs, and tie bars are used to anchor irregularly shaped partial slabs which sometimes occur at the edge of the pavement because of the jointing pattern.

7. PAVEMENT RESTORATION

The traditional method of restoring airfield pavements has been the application of a resurfacing asphalt overlay, or a thicker strengthening overlay when upgrading of the pavement load rating is required. However, the average service life achieved with an asphalt overlay on CATA airport pavements is about five years less than the average service life provided by an original asphalt surface. This shorter service life of asphalt overlays is attributed for the most part to the loss of structural integrity of the asphalt surface which is overlayed, and to the reflection of thermal cracks through the overlay shortly after construction.

Some experimental projects have recently been carried out by CATA to investigate the cost-effectiveness of various techniques which may extend the service life of restoration construction. In 1982, runway 16-34 at Lynn Lake, Manitoba, was restored by removing and replacing the asphalt layer. In 1983, the same method was used to restore runway 08-26 at Brandon, Manitoba, and a similar approach was taken in the restoration of runway 15-33 at Halifax, N.S. The Halifax runway had been overlaid in previous years and the thickness of the asphalt had grown to 300 mm; the top 200 mm was removed and replaced using a recycled asphalt mix. Also in 1983, runway 03-21 at Charlottetown was restored by milling the asphalt surface and compacting the milled cuttings in place as a base for a new asphalt surface.

Due to their short period of service, no firm conclusions can be reached on the longer term performance of these projects. To date, their performance has been satisfactory with thermal cracking being no greater than that which might be expected with original construction.

From an economic viewpoint, the cost-effectiveness of restoration by replacement depends on a number of factors. Cost-effectiveness is usually assured if replacement can be combined with recycling of the removed asphalt materials. Removal of the asphalt surface permits major grade corrections to be made with base course materials rather than with expensive asphalt. Grade corrections for adjacent structures such as manholes and shoulders are also a consideration. The replacement approach has been strengthened by the recent advent of high production milling equipment which permits a high degree of selectivity in the depth and extent of area to be replaced.

Restoration by replacement usually requires closure of the facility during construction whereas partial operations can be maintained if necessary when traditional overlay methods are used. The type and frequency of traffic at the airport, and the availability of alternative facilities is therefore a major consideration when selecting the method of restoration. Within these constraints however, restoration by replacement of the asphalt surface is expected to be adopted with increasing frequency in future years.

8. NORTHERN AIRPORTS

To establish reliable year-round transportation services for remote communities, a policy was adopted in 1974 to provide airport facilities in the Yukon and Northwest Territories at 62 communities having a permanent population in excess of 100. More recently, and based on the same type of criteria, airstrips are being constructed or planned for 14 communities on the Labrador coast, and for another 14 communities in northern Quebec. Most of these airstrips will serve only low volumes of light, propeller driven aircraft and consequently the facilities provided are gravel surfaced.

In areas of permafrost, a design principle followed for gravel surfaced pavements is to minimize disturbances to the underlying thermal regime. At the site selection stage, every effort is made to locate the facility in an area where earth cuts are minimized and where construction can be accomplished mainly through embankment fill. In

many cases, particularly in the eastern and central arctic, few problems with thermal disturbances are encountered because the insulating value of organic material in the active layer is minimal, and the thermal properties of the pavement structure closely resemble those of the in-situ active layer. At sites where the active layer contains significant insulating materials, fill is placed directly on the organic material as the removal of this material prior to embankment construction would lead to a considerable increase in the depth of thaw. Any site clearing necessary is carried out during the winter to avoid damage to the insulating layer.

Ideally, the embankment for a pavement on permafrost would be designed so that the depth of thaw after construction does not exceed the depth of fill. However, experience has shown that provided sufficient granular fill is placed to distribute aircraft loads to the subgrade, some thawing of the former active layer can be tolerated and the resulting settlement or frost heaving can be corrected through a modest continuing program of maintenance regrading. Hence, as a rule, gravel airport pavements constructed in areas of permafrost consist of a minimum of one metre of granular fill placed directly on the undisturbed subgrade, topped with 150 mm to 240 mm of well graded pit run gravel or crushed gravel surfacing. Few serious problems have been reported with this design. However, the stability of gravel surfaces is reduced during spring thaw and after periods of prolonged rain, and a current operational problem is the lack of a simple field test to measure surface stability and determine allowable tire pressures.

The paving of aircraft operational surfaces is a necessity at major airports in the north because most jet aircraft cannot operate on a gravel surface. Paving increases the depth of the active layer and in the discontinuous permafrost areas of the mid-north, it may eliminate the permafrost entirely. If the subgrade is a fine grained soil with a high ice content, the result is deep seated consolidation and major surface settlements which may continue for many years. Although expensive in remote areas, the only economic approach to this situation is frequent programs of extensive patching and leveling until subsurface conditions stabilize.

Fortunately, major problems with permafrost degradation are not widespread and preventive measures are usually possible through detailed geotechnical investigations and careful site selection. Of the nine CATA operated airports with paved surfaces in the arctic, permafrost has been a significant engineering factor at only three and conditions at these sites are now stabilized. The airport at Thompson in northern Manitoba is the only airport in the CATA system which is currently experiencing major permafrost problems of the type described above.

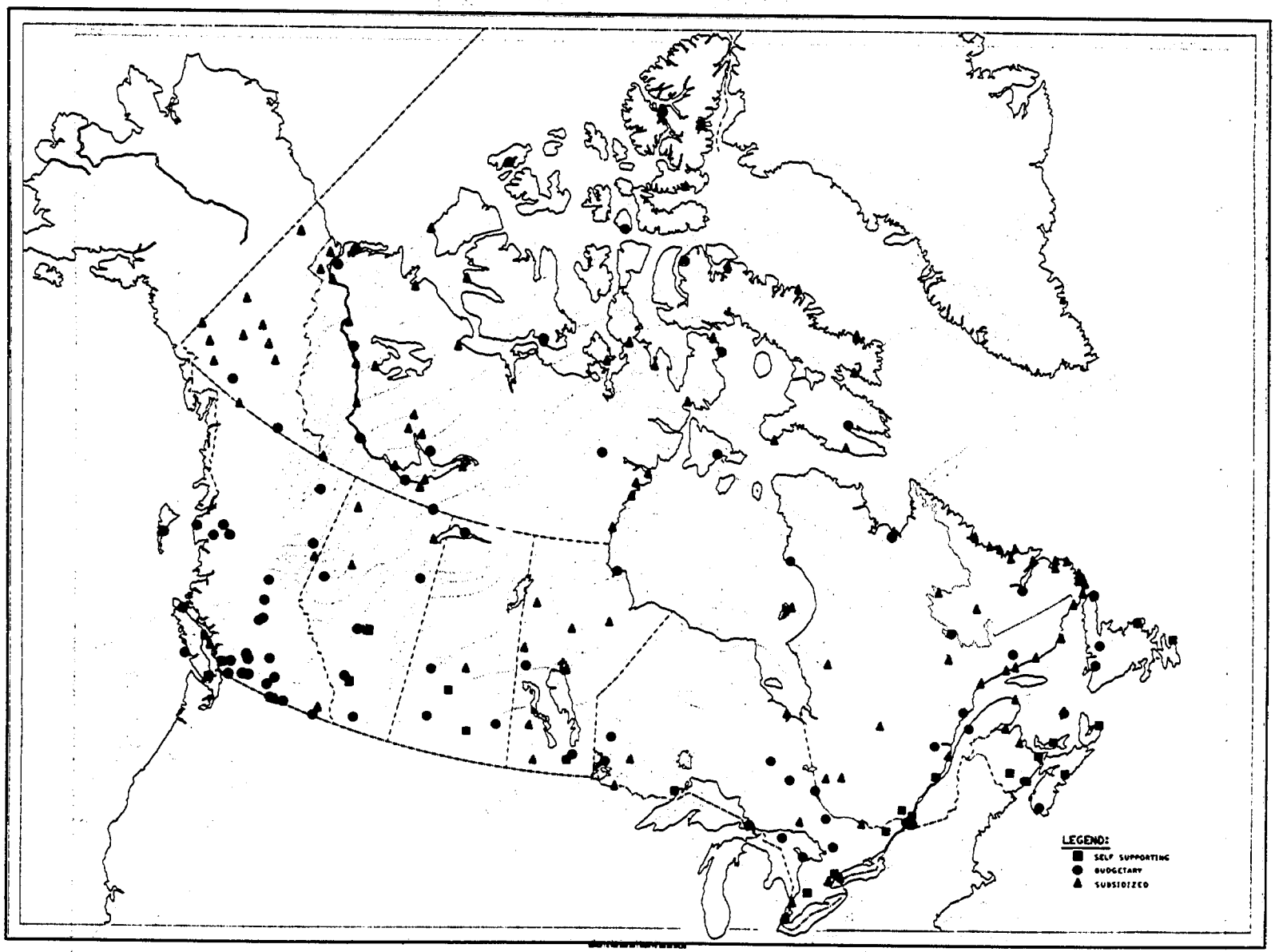


FIGURE I
CATA AIRPORT SYSTEM

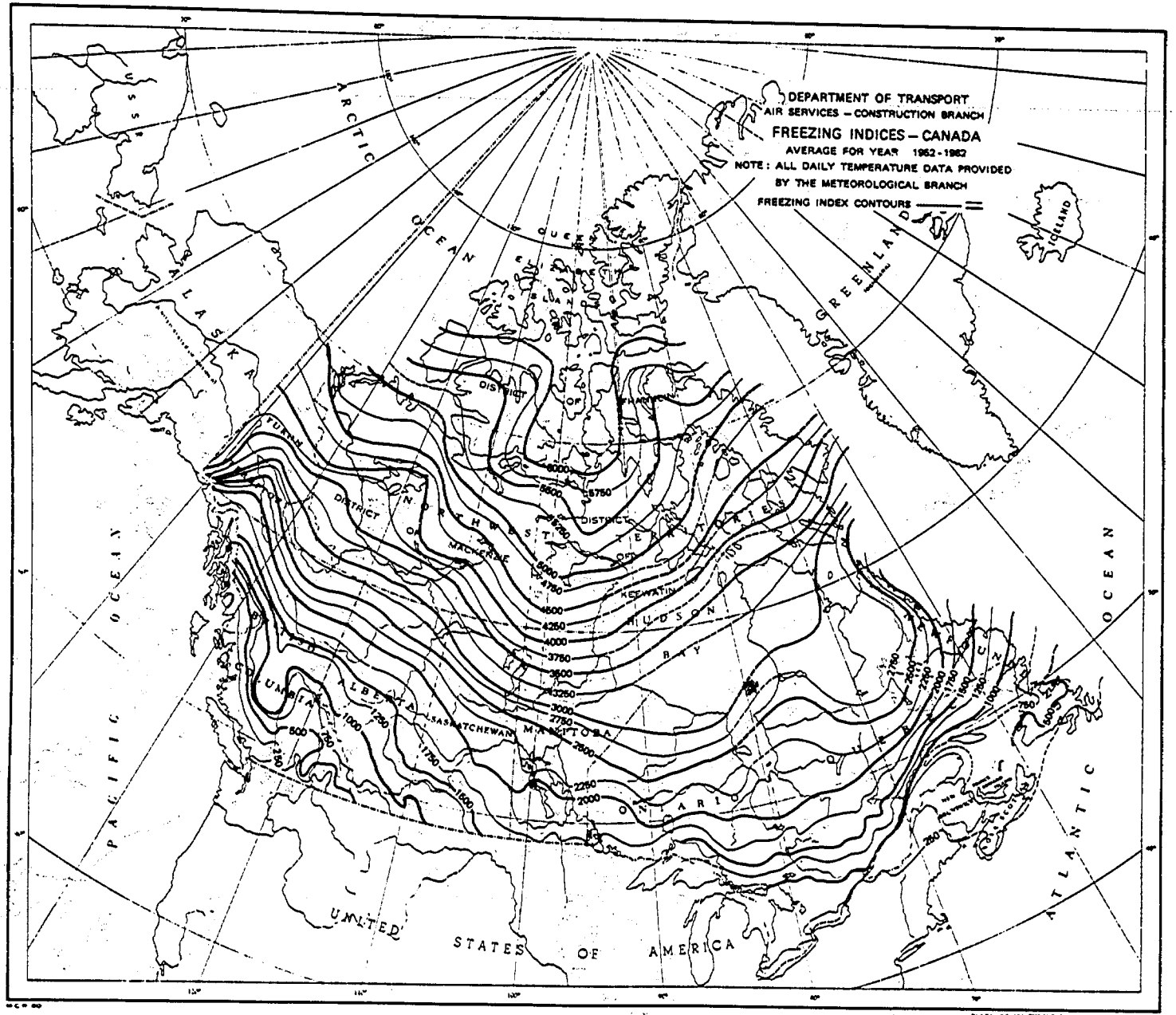
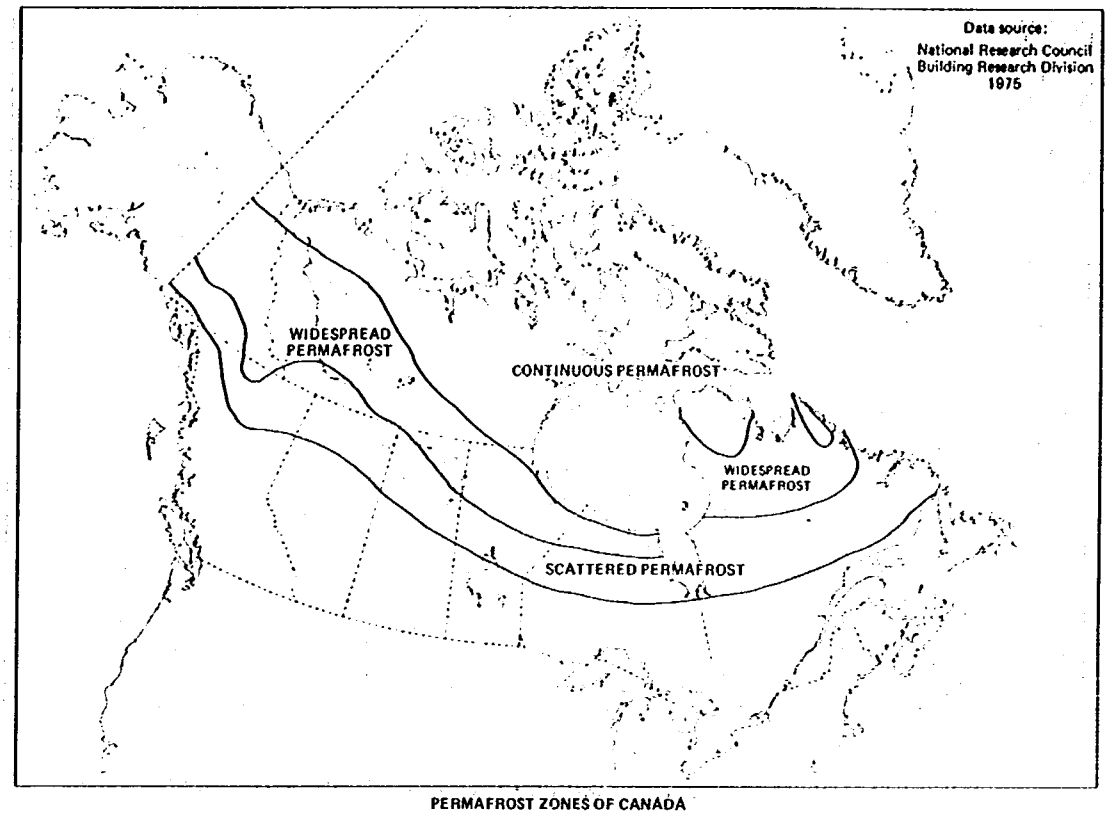


FIGURE 2 CANADIAN FREEZING INDICIES



PERMAFROST ZONES OF CANADA

FIGURE 3

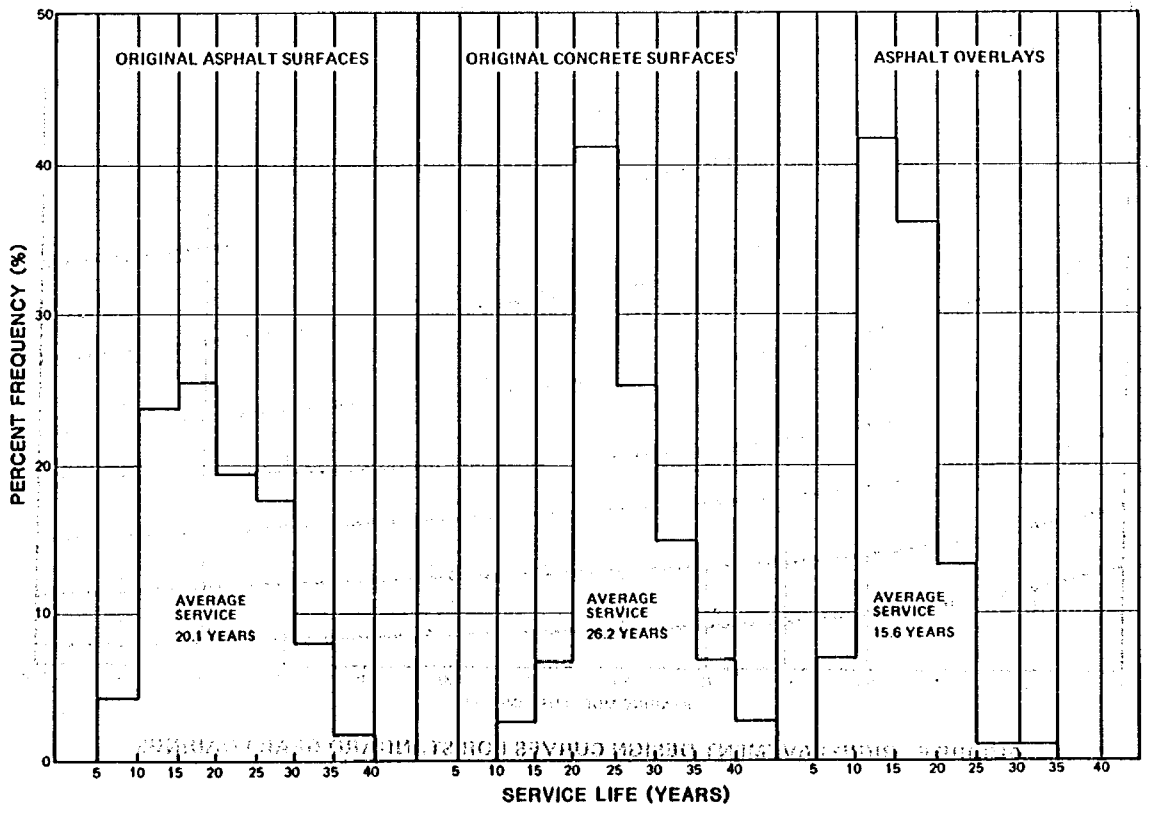


FIGURE 4 DISTRIBUTION OF PAVEMENT SERVICE LIVES

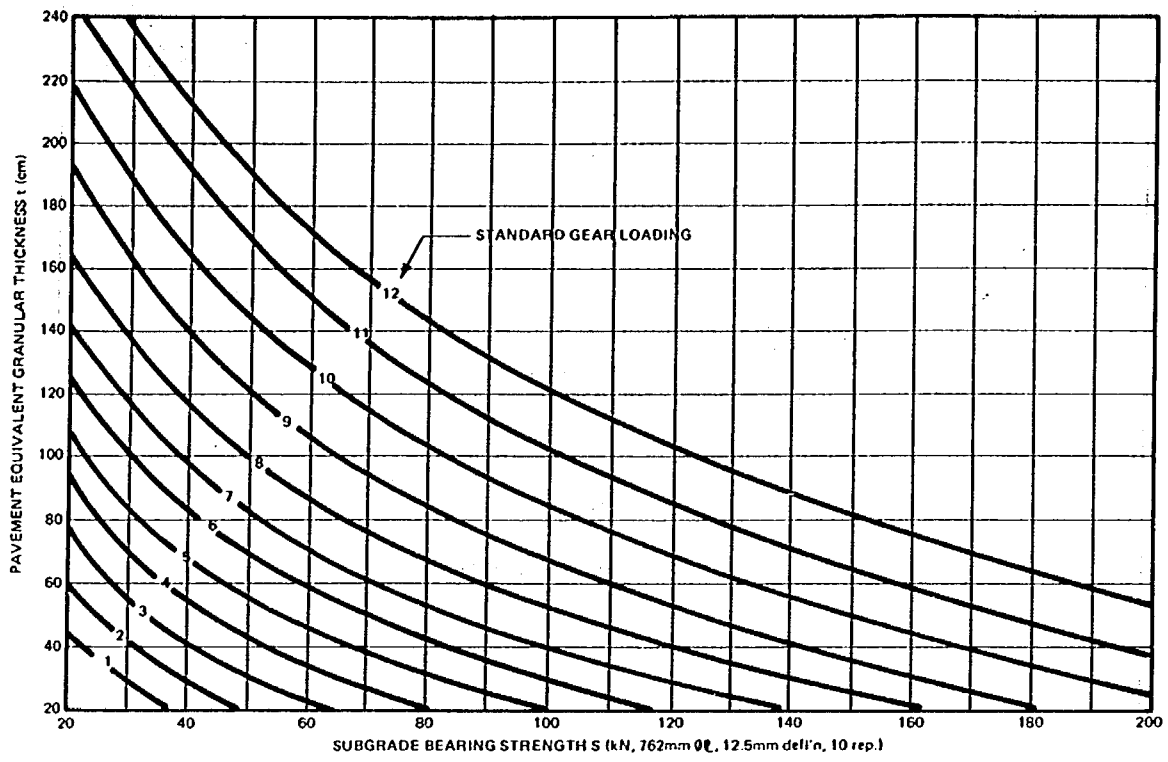


FIGURE 5 FLEXIBLE PAVEMENT DESIGN CURVES FOR STANDARD GEAR LOADINGS

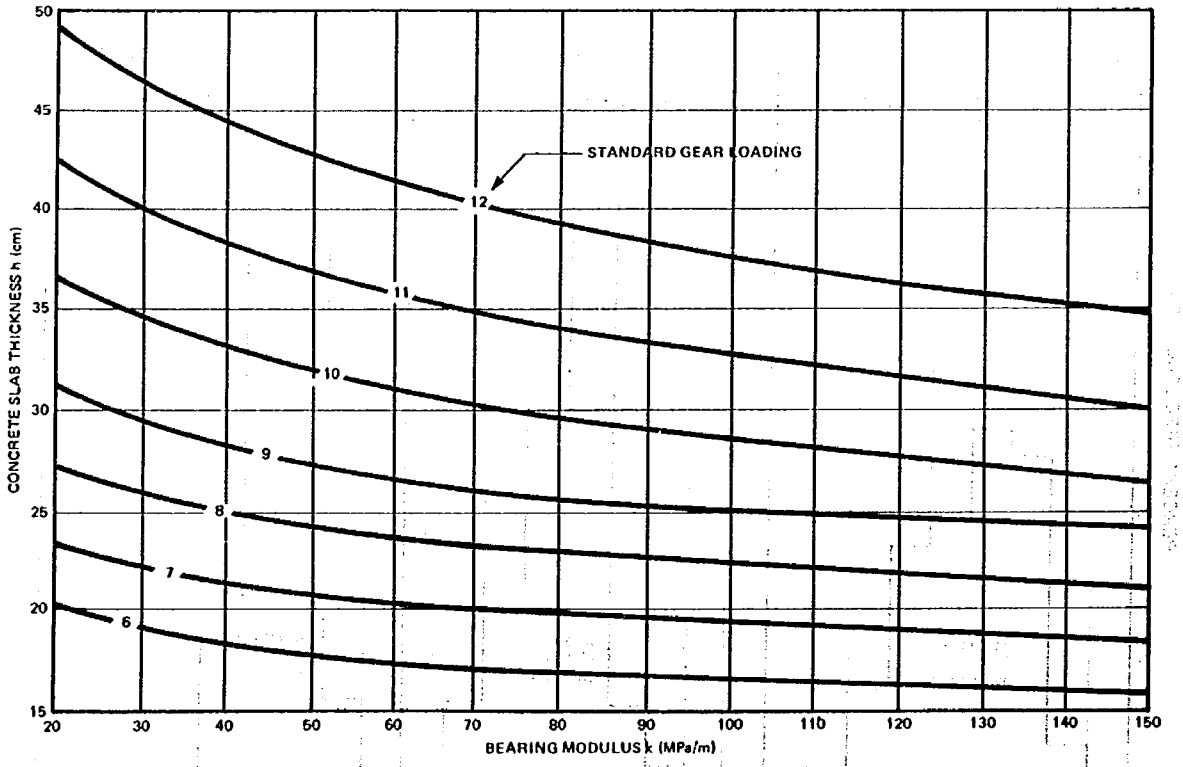


FIGURE 6 RIGID PAVEMENT DESIGN CURVES FOR STANDARD GEAR LOADINGS

Subgrade Soil Type	Usual Spring Reduction %	Subgrade Bearing Strength (kN)		
		Fall Range	Design Value	
			Fall	Spring
GW - well graded gravel	0	290-400	290	290
GP - poorly graded gravel	10	180-335	220	200
GM - gravel with silty fines	25	135-335	180	135
GC - gravel with clay fines	25	110-245	145	110
SW - well graded sand	10	135-335	180	160
SP - poorly graded sand	20	110-200	135	110
SM - sand with silty fines	45	95-190	120	65
SC - sand with clay fines	25	65-155	85	65
ML - silt with low liquid limit	50	90-180	110	55
CL - clay with low liquid limit	25	65-135	85	65
MH - silt with high liquid limit	50	25-90	40	20
CH - clay with high liquid limit	45	25-90	55	30

FIGURE 7 TYPICAL SUBGRADE BEARING STRENGTHS

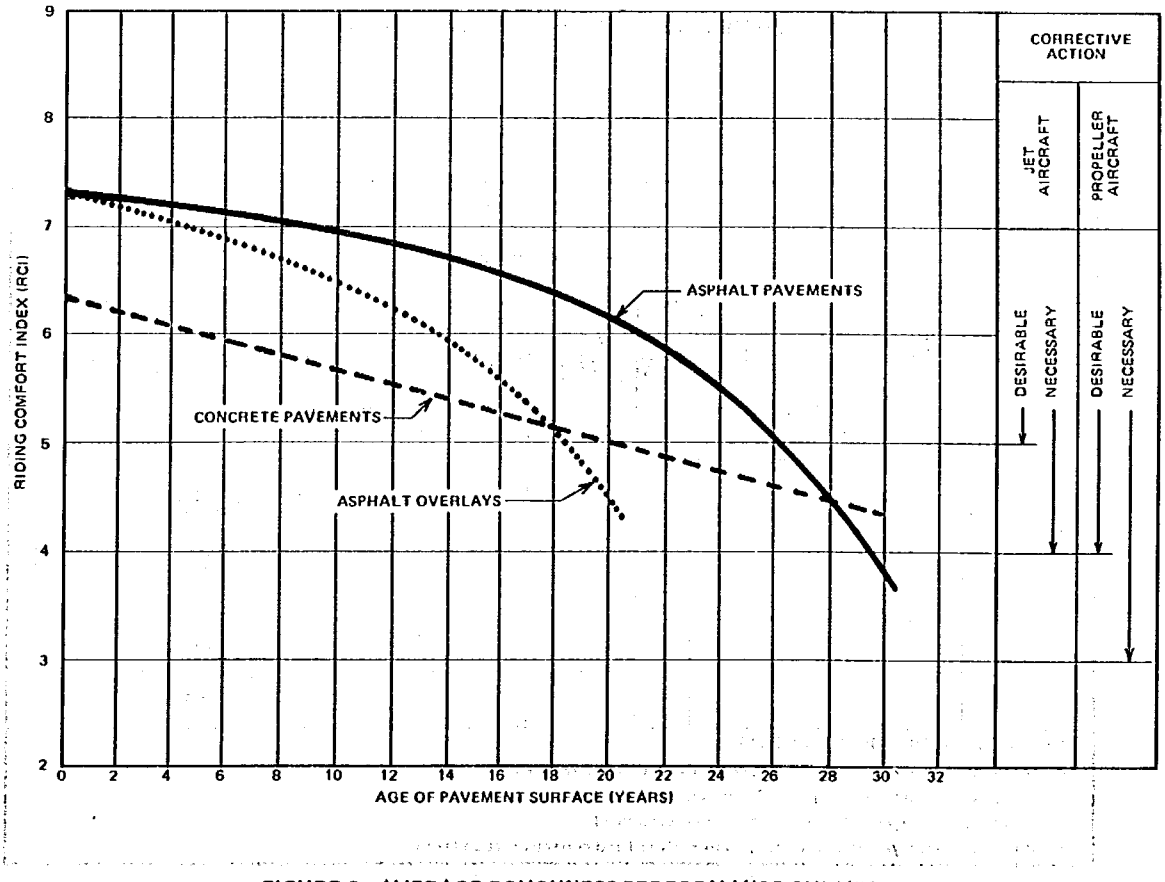


FIGURE 8 AVERAGE ROUGHNESS PERFORMANCE CURVES

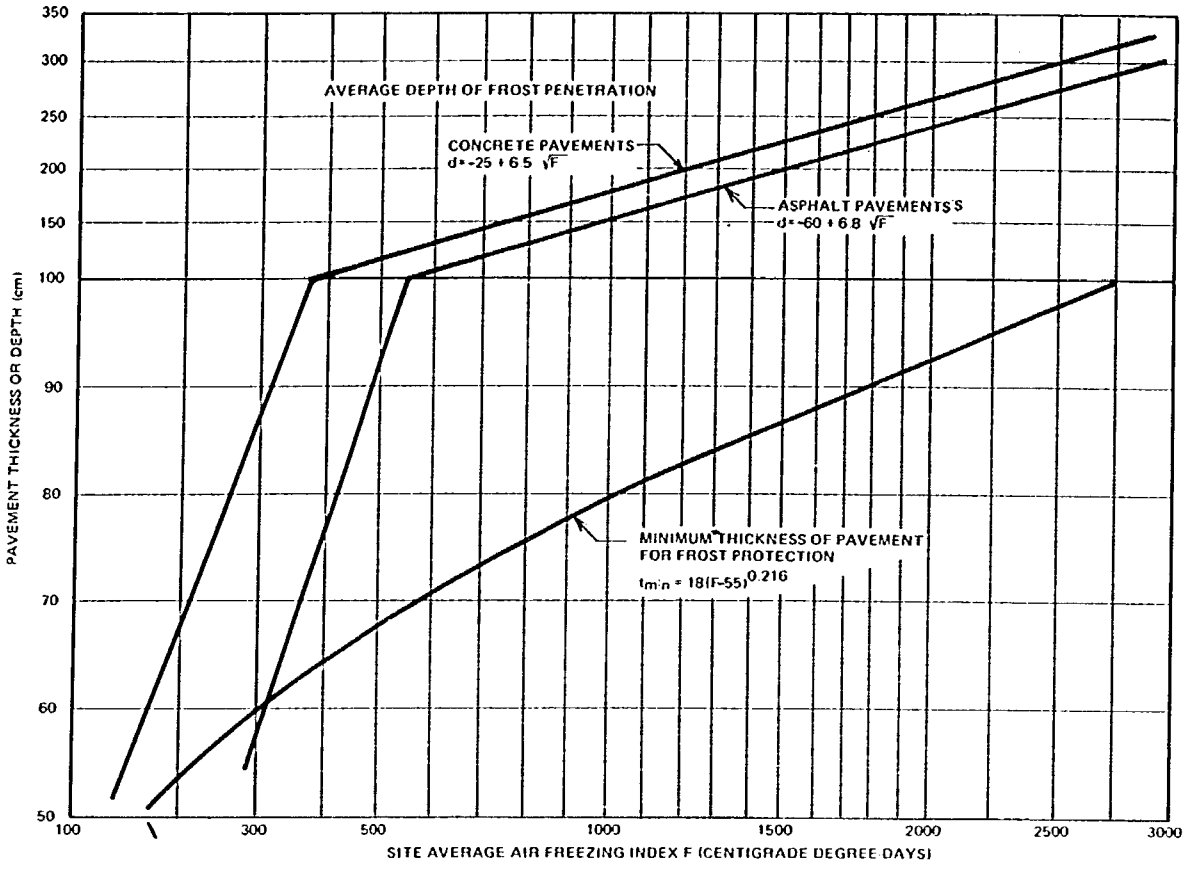


FIGURE 9 PAVEMENT FROST PENETRATION AND PROTECTION

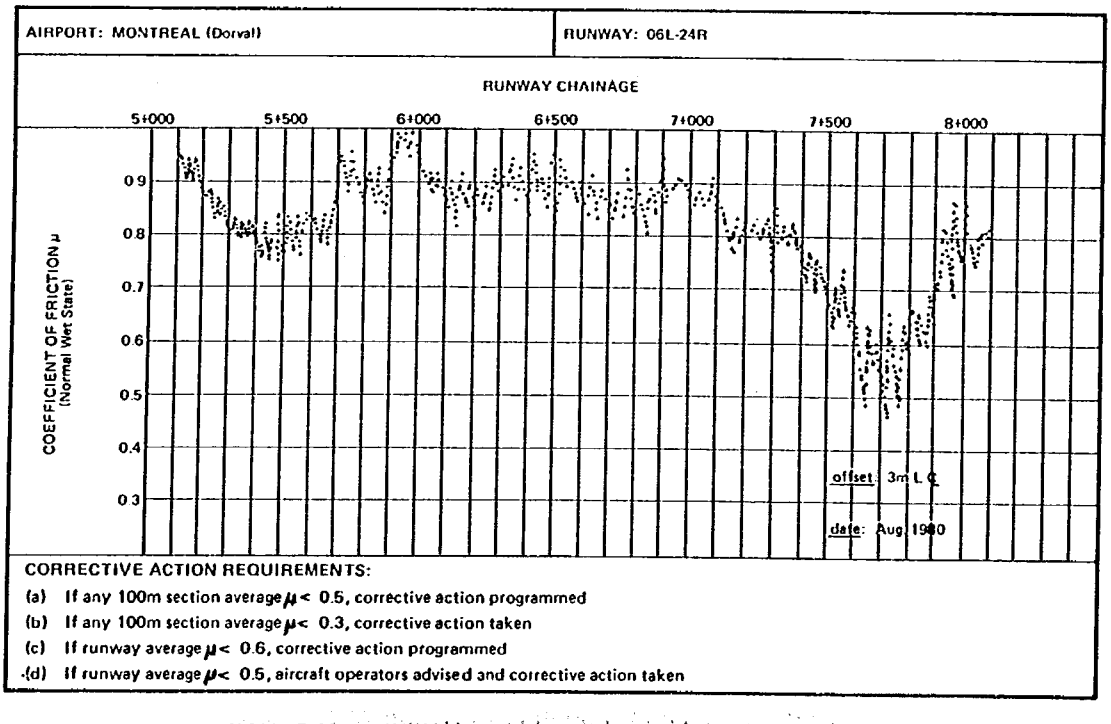


FIGURE 10 RUNWAY COEFFICIENT OF FRICTION PROFILE

Priority Programming for Network Level Pavement Management

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PRIORITY PROGRAMMING FOR NETWORK LEVEL PAVEMENT MANAGEMENT

by

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ABSTRACT

Funding for maintenance and rehabilitation has not been able to keep pace with the needs in most highway agencies. Consequently, it has become more and more necessary to develop a system for managing the pavement network, including the efficient allocation of resources so as to make the best possible use of the limited funds available.

This paper provides a summary description of the pavement management process and then concentrates on the use of priority programming methods for allocating funds. Existing priority programming methods and models for network level pavement management are reviewed and their specific features, advantages and limitations are discussed.

The paper also provides a case application example of the use of two priority programming methods, one involving a comprehensive optimization model and the other involving a simple ranking method.

INTRODUCTION

The basic purpose of a pavement management system is "to achieve the best value possible for the available public funds and to provide safe, comfortable and economic transportation. This is accomplished by comparing investment alternatives at both the network and project levels, coordinating design, construction, maintenance and evaluation activities, and making efficient use of existing practices and knowledges"(1).

Therefore, a pavement management system encompasses a wide variety of activities. The levels of management involved range from policy-related decisions for a number of projects to practice-related, detailed decisions within particular projects. The function of management at all levels involves comparing alternatives, coordinating activities, making decisions and seeing that they are implemented in an efficient and economical manner (2).

It is convenient for operational purposes to consider pavement management as involving two basic levels: network and project. Then, within these two levels there are blocks of major activities, as identified in Table 1(3).

The objectives of this paper are to describe the role of priority programming in pavement management at the network level, to evaluate the available approaches and models and to provide an example application to a highway network.

TABLE 1

PAVEMENT MANAGEMENT: BASIC BLOCKS OF ACTIVITIES FOR THE TWO MAJOR MANAGEMENT LEVELS

BASIC BLOCKS	NETWORK MANAGEMENT LEVEL (Administrative Procedures)	PROJECT MANAGEMENT LEVEL (Technical Management Decisions)
A. DATA	1. Sectioning 2. Data a) Field inventory (roughness, surface distress, friction, deflection, geometrics b) Other (traffic, unit costs, etc.) 3. Data Processing	1. Detailed data (structural, materials, traffic, climate, unit costs, etc.) 2. Subsectioning 3. Data Processing
B. CRITERIA	1. Minimum serviceability, friction, structural adequacy, max. distress 2. Maximum user costs, maint. costs 3. Maximum program costs 4. Selection criteria (max. of benefits; max. of cost effectiveness, etc.)	1. Minimum as-built roughness, structural adequacy and friction 2. Maximum project costs 3. Maximum traffic interruption 4. Selection criteria (i.e., least total costs)
C. ANALYSES	1. Network needs (now) 2. Perf. predictions and future needs 3. Maint. and rehab. alternatives 4. Technical and economic eval. 5. Priority analysis 6. Eval. of alternative budget levels (ave. serviceability of network, amount of deficient, km)	1. Within-project alternatives 2. Technical analyses (performance and distress predictions) 3. Life cycle economic analyses
D. SELECTION	1. Final priority program of capital projects 2. Final maintenance program	1. Best within-project alternative (rehab. or new pavement) 2. Maintenance treatments for various sections in network
E. IMPLEMENTATION	1. Schedule, contracts 2. Program monitoring 3. Budget and financial planning updates	1. Construction, activities, contract control and as-built records 2. Maintenance activities, maint. management and records

BASIC STEPS AND FUNCTIONS OF PRIORITY PROGRAMMING

There are many kinds of priority establishment methods varying from simple subjective ranking methods to sophisticated mathematical programming methods. Each has specific features in terms of the pavement rating parameters, type of economic analysis applied and so forth. However, every priority programming method developed to date for network level pavement management is basically composed of four major steps: information, identification of needs, priority analysis, and output reports, as shown in Figure 1.

The basic function of priority programming can be defined as the process of answering the following three questions (4).

1. What projects (or sections) should be built? (Selection of projects)
2. How should they be built? (Selection of alternative strategies, i.e., within-project alternative)
3. When should they be built? (Selection of project timings)

The models which can deal with these three questions at the same time are complex.

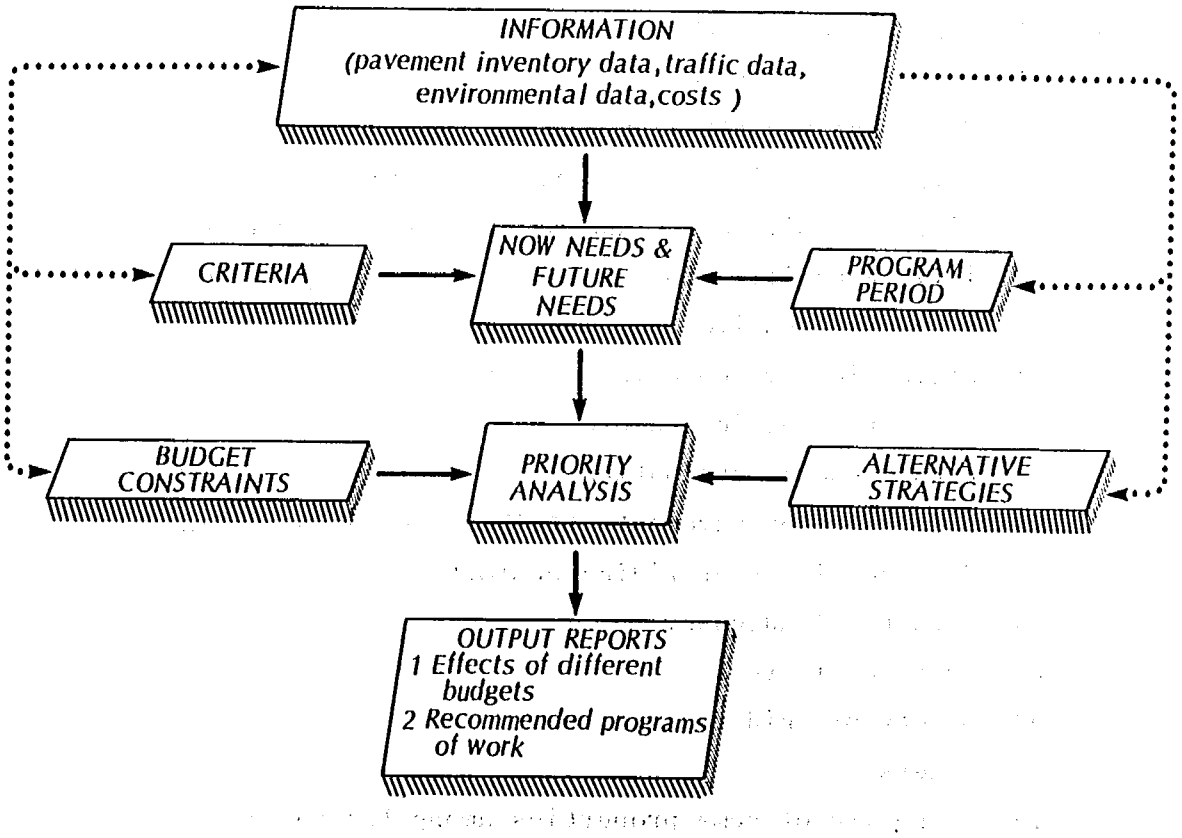

Figure 1 Basic steps in priority programming

Consequently, most of the models used by highway agencies can only deal with the first or first two questions. If the available funds are to be optimized, however, all three questions need to be dealt with.

CLASSIFICATION OF PRIORITY PROGRAMMING METHODS

Priority programming methods can conveniently be classified to reflect the procedures used in the model. The most simple is to divide them into ranking and optimization methods. However, a more comprehensive approach is to clarify them as follows:

1) Classification by prioritization procedures involved
   a) Ranking method
   b) Optimization method
2. Classification by functions or outputs
   a) Selection of projects to be built (What)
   b) Selection of projects and timings (What and When)
   c) Selection of projects and alternatives (What and How)
   d) Selection of projects, timings and alternatives (What, When and How)
3. Classification by within-project alternatives involved
   a) Single alternative
   b) More than two alternatives
      i) Rehabilitation alternatives only
      ii) Rehabilitation and maintenance alternatives
      iii) Rehabilitation, maintenance and capital construction alternatives
4. Classification by economic evaluation
   a) Without economic evaluation
   b) With economic evaluation
5. Classification by pavement attributes used for evaluation
   a) Serviceability or riding comfort
   b) Structural adequacy
   c) Surface distress
   d) Safety or skid resistance
   e) Others
   f) Composite of some properties among (a) to (d)

REVIEW OF EXISTING PRIORITY PROGRAMMING METHODS

The previous section suggested that classification by the prioritization procedures involved is the best for purposes of reviewing and evaluating each method. Some of the existing methods in North America which are categorized into ranking or optimization methods are briefly reviewed as follows.

Ranking Methods

With ranking methods, in order to quantify the degree of adequacy or acceptability and to do the comparison between pavement sections, scores are generally calculated for each section using a procedure established within the particular agency involved. These scores can then be used for establishing priority listings.

Figure 2 shows a flow chart of typical ranking methods. A brief explanation of some ranking methods that have been developed in North America follows.

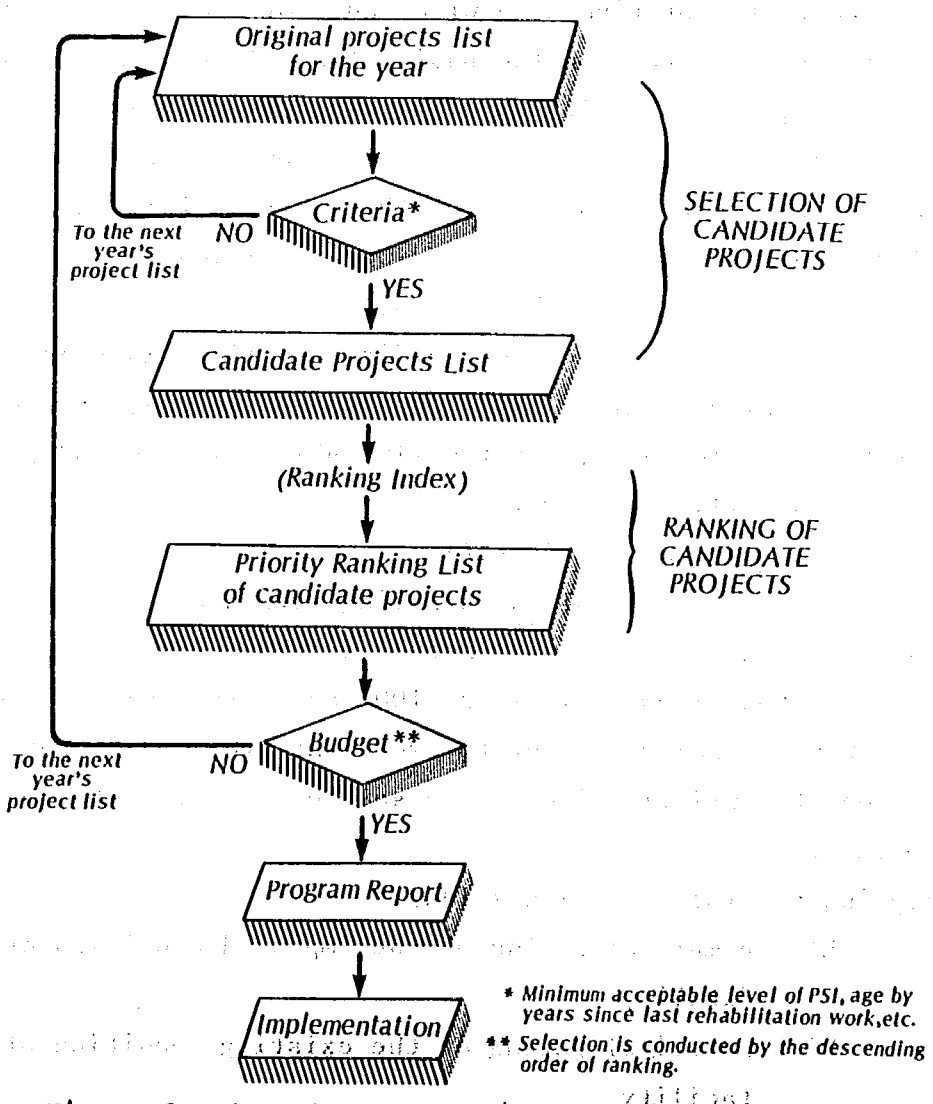

Figure 2 Flow chart of typical ranking methods

Sufficiency Rating Method of the Arizona Highway Department(5)

In the Arizona Method, a highway sufficiency rating system is used to determine the needs of the existing network. To rate the degree of deficiency, a total of one hundred points is broken down into several categories: condition(35), safety(30) and service(35). Each highway section is rated separately through inspection by highway engineers, and a total score is then calculated to describe the deficiency. Another one hundred points are used to describe the socioeconomic factors; environment(40), economic development(35), and traffic safety(35). A composite score is then used for priority analysis. Projects are selected for implementation, starting with the lowest composite score until the budget for the year is exhausted.

Rational Factorial Rating Method(6)

In this method a fractional factorial design was used to develop a priority index. The procedure is to firstly select a number of pavement sections that are representative of a wide range of field conditions. Then a panel of engineers rate them on the basis of a selected set of attributes. The rating obtained, together with physical measurements made on the pavement sections, can be used to arrive at an equation for prioritization. One of the regression equations obtained from the analysis is:

$$Y = 5.4-0.0263*X1-0.0132*X2-0.41og*X3+0.749*X4+1.66*X5$$

where Y is the priority index in which a value of 1 represents very poor and a value of 10 represents excellent condition. X1 to X5 represent, respectively, the following: rainfall(5 inches to 40 inches), freeze and thaw 0 cycle/year to 60 cycle/year), traffic (100 ADT to 100,000 ADT), PSI (4.0 to 2.0) and distress (+1.0 to -1.0).

Priority rating system by Thiers(7)

This method is a good example of a rating method based on both sufficiency rating and economics. In this method a total of 450 points are divided between the cartway (300), sidewalk (100) and curb (50). Six categories of ratings which vary from very poor to excellent are used to reflect their condition. Then, the priority index of a section is calculated with the following formula.

$$\text{Priority index} = (R1 - R2)*W/C = I*W/C$$

where R1 = numerical rating of the improved condition of the facility

R2 = numerical rating of the existing condition of the facility

W = numerical weighting factor representing the relative importance of the facility

C = incremental cost to effect the proposed improvement, and

I = numerical measure of incremental improvement, R1-R2.

Ranking Method of the Ontario Ministry of Transportation and Communications(8)

This method employs a similar approach to that of Thier by incorporating the costs of the projects in the priority analysis. A total of 100 points is divided into a number of parameters which are assumed to constitute the overall condition of the existing facility. The total cost of performing the work required is calculated by using a "bench mark" cost table. This information is then used in the following formula to find the priority guide number of each project.

Priority guide number = (100-Condition Rating)/(Cost per vehicle mile)

Optimization Methods

Mathematical optimization techniques have recently been introduced for programming investment priorities for pavement management. The use of computers for processing inventory data and carrying out the calculations has made this type of analysis not only possible but also effective and practical. In these methods, mathematical modeling of the decision process is undertaken in order to find the best solution. It involves decision variables, constraints and an objective function which is a mathematical representation of the criteria for judging alternate solutions. Figure 3 shows a flow chart of a typical optimization method. A brief description of some of the optimization methods that have been developed for network level pavement management in North America follows.

Benefit maximization with linear programming

In this optimization approach, maximum benefit is derived from the expenditure without violating the budget constraints. The cost of the improvement, the benefits accruing from the improvement and the budgets are all considered in the analysis.

This method firstly calculates, for each improvement project, the optimum year of implementation and compares the total cost of the projects that will be implemented in a particular year with the budget available in that year. Then, by using a linear programming model, the timing of each project is rearranged in such a way so as to minimize the total benefit loss.

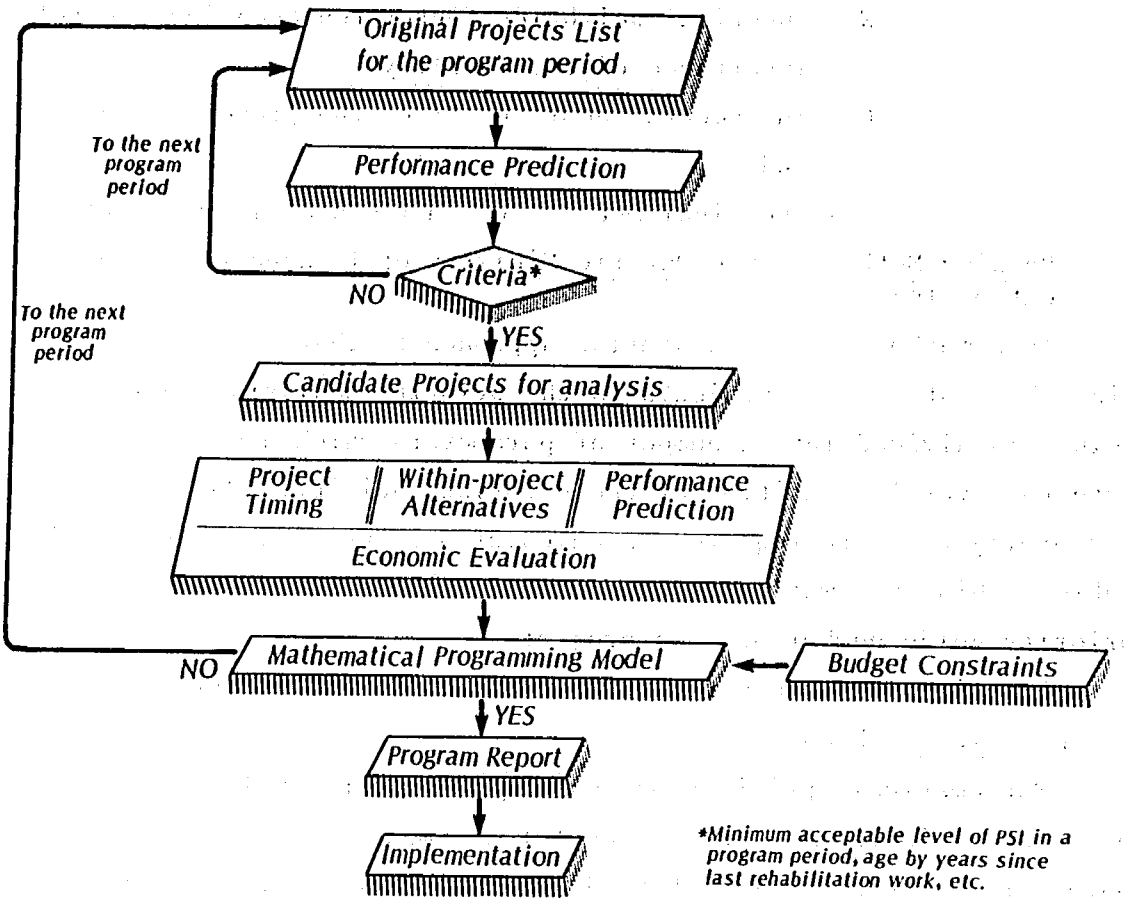

Figure 3 Flow chart of typical optimization methods

Dynamic programming model of the Kentucky Department of Transportation

The Kentucky Department of Transportation has developed methods of applying dynamic programming techniques to optimize the allocation of expenditures over hundreds of highway improvement projects each year (8,9). The cost of the improvement and maintenance through the expected life of the improvement, is considered in the model. Similarly, benefits due to the improvement are taken into account. A multi-stage decision process has been used and the optimal set of projects, each with several alternatives, can be selected based on the evaluation of candidate projects.

Network optimization system (NOS) of the Arizona Department of Transportation

This system (NOS) is currently being used on a fully implemented basis in the Arizona Department of Transportation(10,11). NOS uses linear programming to solve decision variables defined as the proportion of roads in the network at the beginning of a specified time period at a specified condition that will receive a specified rehabilitation action. The total unit cost of these proportions is minimized, constrained by performance standards and mathematical limitation (i.e., sum of proportions is equal to one).

Multi-year priority programming model

Linear programming is used for solving the multi-year priority programming problem in this method(12). This approach explicitly recognizes the trade-off between benefit losses and project timing. It treats each possible implementation year as an independent alternative by delaying the execution of the project over the programming period. In addition to this, the within-project alternatives are also considered in the analysis. The linear programming model is formulated so as to maximize the total present value of m pavement improvement projects, each with k within-project alternatives, for a y year programming period.

Program analysis of rehabilitation system (PARS) of the Ontario Ministry of Transportation and Communications

The Ontario Ministry of Transportation and Communications has developed a method known as "program analysis of rehabilitation system", (PARS). It is divided into two main components: financial planning and priority planning. Financial planning determines levels of funding for selected pavement performance and usage goals. The second component, priority planning, selects projects within a fixed level of funding. Linear programming models are used to optimize the selection of projects from a large set of feasible alternative.

Zero-one integer linear programming model

In this model, project level analysis outputs are used as input data for network level analysis(13). Dynamic programming is used for optimization within budget limits over a program period. In the research on which this method is based, 150 features (sections) with 3 alternatives (3 level of budget limit) over 20 years program period were chosen. The network level decision process is modeled as a 0-1 integer linear programming problem.

Rehabilitation and maintenance system (RAMS) of the Texas Transportation Institute

RAMS consists of seven computer programs, each with its own specific task(14). The combined and sequential use of these programs is expected to facilitate planning, cost estimation, and fund allocation at all levels of management. The mathematical programming techniques used in these seven programs include: 0-1 integer linear programming, 0-1 integer non-linear programming, and dynamic programming. This system is being considered for

implementation in the Texas Department of Highways and Public Transportation.

Rehabilitation information and priority programming system (RIPPS) of Alberta Transportation

RIPPS was developed following the implementation of a Pavement Information and Needs System (PINS) which provides the information on the current and future needs for rehabilitation improvement for each section in the network, using performance prediction models(15,16). RIPPS basically processes the output of PINS, and carries out the selection of candidate rehabilitation alternatives for each section and conducts economic and performance analysis for each alternative for each possible implementation year. A heuristic procedure has been developed as a priority programming model which employs marginal cost-effectiveness analysis. The model can be operated both in cost minimization (given performance constraints) and the effectiveness maximization (given annual budget constraints) modes.

EVALUATION OF EXISTING PRIORITY PROGRAMMING METHODS

A comparison of existing priority programming methods is presented in Tables 2 and 3 for ranking methods and optimization methods, respectively. These tables show that the ranking methods and optimization methods are quite different in terms of function, program period and economic analysis involved. The ranking methods generally do not include detailed economic analysis, nor can they treat the effect of project timings. Some of the optimization methods can deal with this problem of project timings by considering the economic effects. The ability to select project timings is significant because a project, for example, might be needed in a particular year, but when its priority is examined at the network level it might be more economical to delay it or advance it.

Ranking methods are subject to bias because most of them depend on opinions. Also, the weighting factors of the parameters used are based on the personal judgements of engineers. Too much commitment to subjective judgement raises the possiblity of excessive inconsistency and discontinuity in the priority establishment. The optimization methods seem to overcome these deficiencies. In fact, most of the optimization methods reviewed in this paper are based on objective economic analysis and take into account the effect of project timings. The RIPPS system of Alberta Transportation, for example, can provide a priority list for each year in the programming period on the basis of evaluation of project timings and within-projects

Table 2 Comparison of Existing Network Level Priority Programming Methods (Ranking Methods)

Name of Method	Function or End-result			Economic Analysis With--Y Without --N	Program Period Single--S Multi --M	Within Project Alternatives Single--S Multiple--M	Pavement Performance Parameters Used For Evaluation	Other Factors or Parameters Used For Prioritization
	*What	When	How					
Ranking Method of the Arizona H.D.	**Y	N	N	N	S	S	Structural Adequacy, Riding, Comfort, etc.	Alignment Roadway Width Economics, etc.
Priority Index Method (Rational Factorial Rating Method)	Y	N	N	N	S	S	PSI, Surface Distress	Rainfall, Freeze-Thaw, Traffic
Priority Rating System by Thiers	Y	N	N	Y	S	S		Condition Rating of Facility, Costs, etc.
Ranking Method of Ontario M.T.C.	Y	N	N	Y	S	S	Serviceability, Surface Condition, etc.	Surface Width, Drainage, Maintenance Demand, etc.

* What: Selection of Project
When: Selection of Timing
How: Selection of Within Project Alternatives

** Y: Considered
N: Not Considered

Table 3 Comparison of Existing Network Level Priority Programming Methods (Optimization Methods)

Name of Method	Function or End-result			Economic Analysis With--Y Without --N	Program Period Single--S Multi---M	Within Project Alternatives Single--S Multiple--M	Pavement Performance Parameters Used For Evaluation	Mathematical Programming Model Used For Analysis
	What	When	How					
Benefit Max. Model with L.P.	Y	N	N	Y	M	S	***	Linear Programming
Dynamic Programming Model of Kentucky	Y	N	Y	Y	S	M	***	Dynamic Programming (D.P.)
NOS Of Arizona D.T.	Y	Y	Y	Y	M	M	Distress, Roughness	Linear Programming
Multi-Year Programming by Karan	Y	Y	Y	M	M	M	Service-ability	Linear Programming (L.P.)
PARS of Ontario M.T.C.	Y	Y	Y	Y	M	M	Service-ability	Integer L.P. and L.P.
Zero-One Integer L.P. by Artman	Y	Y	Y	Y	M	M	PCI (Pavement Condition Index)	0-1 Integer L.P. + D.P.
RAMS of Texas T.I.	Y	N	Y	Y	M	M	Pavement Condition, Roughness	0-1 I.L.P., D.P., etc.
RIPPS of Alberta Transportation	Y	Y	Y	Y	M	M	PQI (Pavement Quality Index)	Marginal Analysis

*** Originally developed for highway improvement programmings

alternatives at the same time. However, optimization methods also have some limitations. One of them is complexity. More important, they have the limitation of performing detailed economic analysis and optimization based only on the factors that can be expressed in monetary or numerical form. In other words, they do not generally take into account the factors which can not be expressed in numerical forms. They can not handle subjective factors such as empirical judgement of engineers or public demands except in the form of adjustments to the priority programs.

Table 4 presents a summary comparison of the advantages and disadvantages of the two classes of methods.

TABLE 4 Advantages and Disadvantages of Priority Programming Methods

	TYPE OF METHOD	ADVANTAGES AND DISADVANTAGES
RANKING	Simple Subjective Ranking of Projects Based on Judgement	Quick, Simple; Subject to Bias and Inconsistency; May be Far from Optimal
	Ranking Based on Parameters, Such as Serviceability, Deflection, etc.	Simple and Easy to Use; May be far from Optimal
	Ranking Based on Parameters with Economic Analysis	Reasonably Simple; Should be Closer to Optimal
OPTIMIZATION	Optimization by Mathematical Programming Model for Year by Year Basis	Less Simple; May be Close to Optimal; Effects of Timing not Considered
	Comprehensive Optimization by Mathematical Programming Model Taking into Account the Effects of Project Timing	Most Complex; Can Give Optimal Program (Max of Benefits)

EXAMPLE PRIORITY PROGRAMMING APPLICATION

General Descriptions of the Example Highway Network

This section summarizes a typical case study of both optimization and ranking methods applied to a highway network. The methods employed are multi-year priority programming model by Karan (i.e., LP model) and rational factorial rating method by Fernando (i.e., Ranking model).

The network involves the Trans Canada highway system of the Province of Prince Edward Island (P.E.I.) in Canada. Approximately 120 kilometers of the network was broken into 25 homogeneous sections on the basis of traffic volume, pavement types and thicknesses and geometric characteristics,

in which the length of a section varies from 900 m to 11600 m. Field inventory data which had been collected in 1979 by the P.E.I. Department of Highways could be used in this study without any modifications ( ).

Linear Programming Model

Data Requirements

The inventory information used for analysis involves the following:

1. Deflection Measurement
2. Roughness Measurement
3. Condition Survey
4. Overall Serviceability Index, and
5. Traffic Information.

Deflection measurements were taken in the outer wheel path of each road section with a Dynaflect unit, and these are used along with the appropriate traffic data to determine the Structural Adequacy Rating (SAR) of each section. The SAR for a pavement section reflects the degree of structural deficiency. Values of SAR ranging between 0 and 4.9 indicates structural inadequacy whereas values between 5.0 and 10.0 indicate structural adequacy.

The Automatic Road Analyzer unit (ARAN) was used to measure road surface roughness on one full lane length of each section. This involved sampling at 500 meter intervals at a travel speed of 50 km/h. The ARAN roughness data collected on each road section were converted to a Riding Comfort Index (RCI) values using a previously established correlation between ARAN roughness and subjective panel ratings. The RCI ranges between 0 and 10, where 0 indicates the worst imaginable riding comfort and a value of 10 indicates a perfectly smooth riding pavement.

The surface condition of each road section was also rated by evaluating and recording various types of surface defects or distress on a specially developed keyboard in the ARAN unit. Each type of distress, its extent and its severity were recorded. The various types, amounts and severities of distress surveyed were then weighted and translated into a Condition Index (CI), with values ranging from 0 to 10. A value of 10 indicates no observable distress whereas a value of 0 indicates very severe and extensive amounts of distress.

In this study, the overall Serviceability Index (SI) of a pavement section was assumed to be a function of its Structural Adequacy Rating and Riding Comfort Index, as follows:

$$SI = a*(RCI) + b*(SAR)$$

where a and b are weighting factors which satisfy; a + b = 1.
The weighting factors used reflect the agency's policies and objectives.

The traffic data obtained for each section include AADT volume, percent annual growth rate and commerical traffic content. The AADT was directly used in the study. Table 5 presents value of SAR, RCI, CI, SI, AADT and section length for each section analyzed in the study.

Basic Assumptions

In order to conduct the analysis, the following assumptions were used.

1. A minimum acceptable Serviceability Index level of 5.o was used in the analysis. Thus, a section which had a present SI of 5.0 or less was identified as requiring a major rehabilitation in the year.
2. A programming period of five years was selected. For the purpose of predicting future performance of pavement sections, curves originally developed using a Markov model by Karan, et al.(18), were used with some modifications in which nonlinear relations were converted into linear relations.
3. The following three alternative rehabilitation strategies for each road section were analyzed by the priority program.

   a) Surface treatment - A double 9.5 mm (3/8 inch) chip seal costing $2.40 per square meter.
   b) Overlay - An 89 mm (3½ inches) thick asphalt concrete overlay placed on top of the existing pavement at a cost of $7.65 per square meter.
   c) Reconstruction - A completely new pavement with a structural design of 152 mm (6 inches) asphalt concrete surface and 254 mm to 305 mm (10 to 12 inches) base at a cost of $22.50 per square meter.
4. Three levels of expected annual rehabilitation budget, $2,000,000, $1,000,000 and $500,000 were considered in the study. They were assumed to remain constant over the 5 years programming period.
5. Economic analyses were conducted for each implementation year and for each combination of project and rehabilitation strategies. Direct agency costs (rehabilitation costs and routine maintenance costs) and user benefits (i.e., savings in vehicle operating costs due to improved pavement conditions) were calculated for each project in the programming period of 5 years.

Table 5 Sectional Summary of Inventory Data

Section Number	SAR	RCI	CI	SI	AADT	Section Length (m)
50	7.5	7.5	8.0	7.5	1300	11600
51	8.0	7.5	8.4	7.8	1300	13350
52	5.8	8.2	8.2	5.8	1500	7900
53	4.3	7.7	8.5	5.7	1950	4650
54	4.3	5.1	4.3	4.8	1950	5400
55	2.0	5.0	10.0	3.8	3750	4800
56	6.3	6.0	6.3	6.1	3750	11500
57	5.0	5.0	4.1	5.0	10100	5450
58	6.3	5.5	8.2	5.8	7450	2150
59	6.3	7.8	10.0	6.9	4500	1650
60	7.0	7.7	10.0	7.3	3500	4750
61	6.3	7.8	10.0	6.9	3500	3250
62	5.3	8.4	9.9	5.3	3500	3300
63	5.0	2.9	2.8	2.9	3500	2100
64	5.0	8.6	10.0	5.0	3500	3050
65	1.5	7.7	10.0	4.0	3500	1000
66	7.8	8.1	10.0	7.8	3500	5850
67	4.8	8.1	10.0	4.8	3500	2000
68	5.5	5.4	10.0	5.4	2700	6500
69	4.8	5.3	10.0	5.1	2700	4750
71	4.3	6.5	6.2	5.2	3200	6200
701	1.0	3.0	5.2	3.0	2700	1000
702	3.5	1.9	4.5	1.9	2700	900
703	8.0	2.4	7.1	2.4	2700	1300
704	8.5	3.1	5.7	3.1	2700	1450

Linear programming formulation and computer program

The LP formulation was used for the analysis. The numbers of decision variables and constraints were 375 (25*5*3) and 401 (including 375 non-negativity constraints) respectively. A Linear Programming package called LP1 (Cyberware Computer System Ltd.) was used for the optimization.

Pavement improvement priorities

Table 6 lists the pavement rehabilitation needs for each year in the programming period. This would have been the priority list if unlimited funds were available. However, under budgetary constraints some projects can not be built in the year in which they require major rehabilitation.

TABLE 6 - Pavement Rehabilitation Needs by Sections During the Programming Period of 5 Years

Year	1	2	3	4	5
Section Numbers	54		52		56
	55		53		59
	57		62		61
	63		68		
	64		69		
	65		71		
	67				
	701				
	702				
	703				
	704				
Total number of Sections	11	0	6	0	3

Table 7 gives the recommended pavement improvement priorities for a budget level of $1,000,000/year, based on the LP model results. Inspection of the table indicates that only 1 of the 11 projects identified as candidates in the first year (see Table 6) could actually be built in that same year. Also, the best type of rehabilitation for each section is given in Table 7. Tables 8 and 9 show the priority lists for budget levels of $2,000,000 and $500,000 respectively.

The average serviceability levels (SI) of the network that would result from the implementation of the recommended priority lists for the four budget levels (including that of 0 capital funds) are shown in Figure 4. Bud-

get level 2 ($1,000,000)/year would keep the network at approximately its current SI level, while budget levels 3 and 4 would result in significant deterioration.

TABLE 7 Pavement Rehabilitation Improvement Priorities By Sections Over 5 Years Programming Period (Available Fund $1,000,000/year)

Year 1	Year 2	Year 3	Year 4	Year 5
57-R	63-0	55-R	54-R	
	64-0	67-R	704-R	
	65-0			
	701-0			
	702-0			
	703-0			

* S: Surface Treatment
O: Overlay
R: Reconstruction

TABLE 8 Pavement Rehabilitation Improvement Priorities by Sections Over 5 Years Programming Period (Available Funds $2,000,000/year)

Year 1	Year 2	Year 3	Year 4	Year 5
55-R	63-R	54-R		
57-R	64-R	703-R		
	65-R			
	67-R			
	701-R			
	702-R			
	704-R			

* S: Surface Treatment
O: Overlay
R: Reconstruction

TABLE 9 Pavement Rehabilitation Improvement Priorities by Sections Over 5 Years Programming Period (Available Funds $500,000/year)

Year 1	Year 2	Year 3	Year 4	Year 5
57-R		55-0	64-0	54-0
		63-R	701-0	65-R
		67-R	702-0	
			703-0	
			704-0	

* S: Surface Treatment
O: Overlay
R: Reconstruction

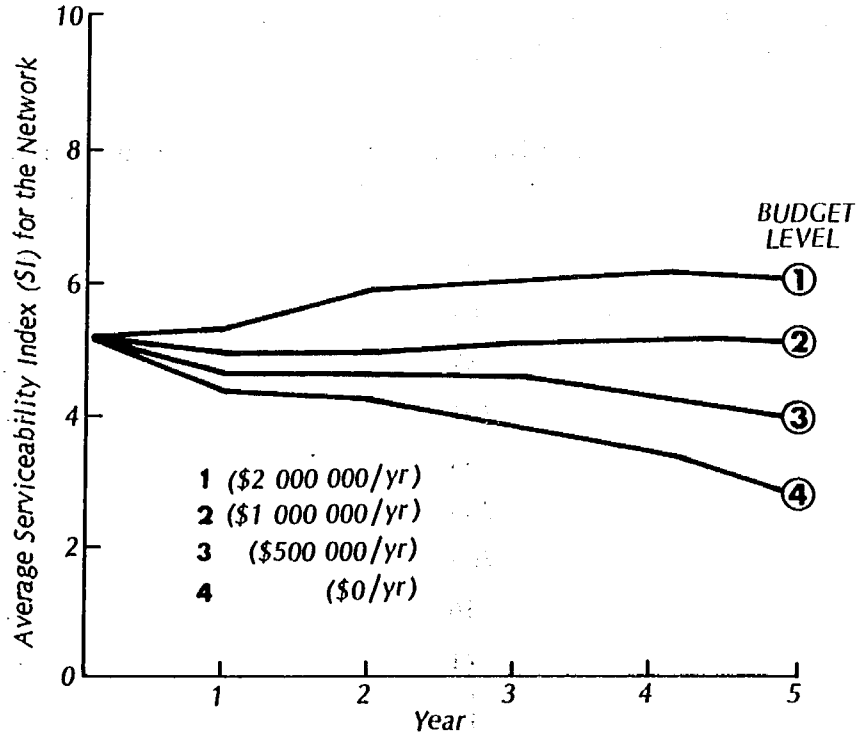

Figure 4 Budget Level Analysis

Ranking Model

Data requirements

The regression equation developed by Fernando and Hudson was used to analyze the network, without any modifications. However, it should be noted that the equation is practically applicable only to the particular region where the original analysis was conducted.

In this case study, the inventory information (RCI, AADT and CI) described in the preceding section were used. The values of RCI and CI were converted into the scale of the original regression analysis. Also the variables on amount of rainfall and amount of freeze-thaw were considered to be constant throughout the area where the network exists.

Computation and outputs

A prioritization index was calcualted from the regression equation for each section listed (Table 10). Then, candidate projects can be selected in the order of prioritization index (starting from project 63, then 702, 67, ---) until the available funds are exhausted.

TABLE 10 Priority Ranking of Pavement Sections Based on the Calculated Prioritization Indices

Ranking	Section Number	Prioritization Index
1	63	3.34
2	702	3.80
3	67	3.85
4	57	3.90
5	701	4.20
6	54	4.27
7	704	4.38
8	703	4.48
9	56	4.95
10	71	5.02
⋮	⋮	⋮
25	64	6.58

While Table 10 identifies most of the projects from the optimization method (Tables 7, 8 and 9), the timing would involve some major differences. For example, section 57 would be done in year 1 for all three budget levels but it is 4'th in Table 10. Also, section 55 appears in the first or third

year for all three budget levels but is not within the top 10 in Table 10.

Thus, while the ranking model approach is simple, it may result in a program which is significantly different from an optimal one.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the state-of-the-art of network level priority programming methods and the evaluation and applications of the methods described in this paper, the following are recommended as key points which highway agencies should take into account when applying priority programming methods to their network.

1. Prior to the establishment of a model, general features of the network to which the priority programming is going to be applied, should be totally examined.
2. Specific objectives of the agency involved must be clearly identified. These can be divided on the basis of whether the model is to be used as a budgeting tool, or as an allocation tool.
3. Suitable types and accessibility of inventory data are indispensable.
4. The scope of the priority analysis required should be determined; i.e.,
   a) whether to choose a single year or multiple year programming,
   b) whether the effect of project timings is to be considered,
   c) whether within-project alternatives are to be considered, and
   d) whether an economic evaluation is to be employed.
5. In order to conduct economic analysis, the user costs associated with varying levels of pavement condition should be quantified.

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Evaluation of the Serviceability of Pavement and Forecast of the Surface Performance by MCI

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ABSTRACT

To quantitatively express the performance of the pavement by the surface characteristic values of cracking ratio, rutting, etc., the PSI formula derived from the AASHO Road Test results, is widely known.
In Japan, a modified performance evaluation formula for the PSI formula has been used. Its validity has been tested by the advancement of the level of maintenance of road in recent years.
In the report, a newly prepared performance evaluation formula (MCI - Maintenance Control Index formula) is described, and the surface characteristics of the national highways and their changes evaluated with this formula used are discussed. First, the surface performance of an extension of about 100km of the national highways was examined with respect to three characteristics of cracking, rutting and evenness, along with a visual survey conducted simultaneously by a group of several surveyors. Then, from the results, a performance evaluation formula or MCI was derived, and the differences from the PSI formula were tested. Next, using the MCI formula, changes in the surface performance when repairing was made and when it was not, were forecast.
What is reported here is part of the findings of the "Study on Planning Maintenance and Repairing of Pavement" carried out over a period of from 1979 to 1981 as a theme of the Technical Research of the Ministry of Construction, and the data bank used for planning the maintenance and repair of the national highways based on the investigation will also be introduced briefly.

EVALUATION OF THE SERVICEABILITY OF PAVEMENT AND FORECAST OF THE SURFACE PERFORMANCE BY MCI

by

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and

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Foreword

Roads are required to have a certain level of serviceability including the width, radius of curvature, gradient, safety facilities, pavement performance, etc. depending on their importance, function, etc. Of these factors, the pavement performance is considered to be the aptitude of the pavement as seen from the drivability of the vehicle, and its evaluation is generally made by evaluators of various standings. Then, it is foreseeable that the evaluation of the same pavement may differ from evaluator to evaluator in that the factors taken as important are subject to change with the standpoint and subjectivity of the evaluator, and that the drivability is subject to vary with the vehicle.

Thus, the performance of pavement is an abstractive, sensitive and subjective concept, and when seen from the position of maintaing and administering the road, determining the performance from the drivability of vehicle and reflecting it in the road maintenance and administration comprise many points of difficulty. However, in order to cope with the situation requiring a higher level of maintenance because of the increasing extension of road administration as well as the requirements of the roadside inhabitants for maintenance and repair of the pavements in recent years, it is required to effectively use the limited budgetary appropriation for the maintenance and repair. And, it is desired to formulate a practical pavement evaluation method so that the maintenance and repair plan is made more adequately along such purpose for reasonable maintenance and administration of the pavement.

As a quantitative expression of the serviceability of pavement by the surface characteristic values such as cracking and rutting, the Present Serviceability Index derived from the AASHO Road Test results is known. But, the test was carried out in 1956 to 1960 in America, and when the difference of the conditions in Japan to those in American, and also the

changes in the sense of the evaluators due to subsequent development of the automobile technology and changes of the social conditions, there will be some room of consideration left for direct application of PSI to the present pavement in Japan.

In Japan, similar tests were made by various organizations, and some performance evaluation formulas were proposed. But, as the objectives of investigation were restricted, their scope of application was limited.

In consideration of the foregoing, an examination was made of the evaluation of the serviceability of pavement, and four performance evaluation formulas were prepared.

Next, using the formula of Maintenance Control Index or MCI, forecasting of the surface performance of the national highway was made for examination of a desirable way of maintenance and repair. Examination was also made of the data bank for more effective use of the values of measurement of the surface performance. These are reported here.

1. Formulation of Serviceability Evaluation

1.1. Investigation

1.1.1. Principle

For the investigation, it is important that appropriate and sufficient evaluators be chosen and that the area of investigation and the number of the points of investigation, investigation period and items be considered carefully.

The occupations and composition of the evaluators will have a great effect on the result of the evaluation, and the investigation area relates to the area of application of the evaluation formula, and the number of the points of investigation to the exactness of the evaluation formula. If the investigation period is short, the result of evaluation may be affected by the social conditions at that time so that slightly longer period of investigation is required. For the items of investigation, it is desirable to investigate all of the items which it is considered are related to the performance, and for those items which are hardly determined quantitatively, it is necessary to examine the method of examination carefully.

With these considered together, it was thought that much expense, time and labor would be required for conducting a new investigation concerning the evaluation of serviceability. Thus, an analysis was made mainly of the available results of the surface maintenance survey which was considered would generally satisfy the foregoing requirements after some supplementary

investigations were made.

1.1.2. Method of Investigation

The surface maintenance survey is one carried out throughout the country except Okinawa from 1972 as part of the survey of the administration of national highways under the direct jurisdication of the government and is comprised of the basic survey and general survey.

The basic survey is intended to obtain the basic data for providing reasonable criteria for maintenance and repair. Under the survey, a section of about 300m of asphalt pavement of known pavement composition, etc. is chosen to measure the cracking ratio, rutting, evenness, coefficient of skid resistance and traffic volume once a year, along with an overall evaluation of the surface by visual observation. This was carried out at 138 places up to March 1980.

The general survey is intended to obtain the data for preparing a maintenance and repair implementation plan. Under the survey, an administration section is divided into unit sections of maintenance and repair (a section of about 2∿3km), and for each of those sections which it is considered maintenance and repair will be made in one to two years, a section of about 200m is taken, and a similar survey to the basic survey is carried out.

1.2. Surface Performance and Pavement Serviceability

1.2.1. Surface Characteristic Values Representing the Surface Performance

The surface performance is expressed by such characteristic values as cracking ratio, rutting, evenness and coefficient of skid resistance as well as the surface roughness and uniformity. These are characteristics greatly related to the performance of pavement, and when these characteristics or characteristic values are used for evaluation of the performance of pavement, it is convenient practically that they be as few as possible and be quantifiable with relative ease. Also, they must be such that the measurement is made relatively easily, with reproducibility.

When the performance of pavement degrades, maintenance repair is to be made. Therefore, it is necessary to consider the characteristics which are used for determination if the repair is required or not under the present condition. In consideration of the foregoing, three characteristic values of cracking ratio, rutting and evenness were taken as representing the surface characteristics expressing the performance of pavement. These characteristic values are outlined below.

(1) Cracking ratio (%)

The cracking ratio represents the proportion of the cracked area to the whole area of investigation. To calculate the cracking area, two methods are available; one is a measurement method according to which a total of the area of cracks in the form of a net and the length of linear cracks multiplied by 0.3m is taken as the cracking area; and the other is a mesh method according to which the total of the value obtainable by multiplying the number of meshes in which cracks are produced in the form of a net by $0.25m^2$ and the value obtainable by multiplying the number of meshes in which linear cracks are produced by $0.15m^2$. In general, the mesh method gives greater values of cracking ratio than the measurement method.

The surface maintenance survey was made generally by the measurement method, with the mesh method applied only to very small part. Recently, however, the mesh method has been employed increasingly for the sake of labor economy.

(2) Rutting (mm)

The rutting is given by measuring the cross-sectional form of the surface by a traverse profilometer and by taking the difference between the highest and lowest parts as the rutting of the cross-section. With this measurement is made at an interval of 20m, the mean value is taken as the rutting of the section. As a value for the road administrator to use in determining if maintenance repair is necessary or not, it is often maintained that the largest of the values calculated for the respective cross-sections should be used. But, the maximum value is apt to be governed by an abnormal value. This involves a problem that it is hardly forecast. Then, it is sometimes maintained that a value of combination of the mean value and a statistic value $\sqrt{V}$ expressing the extent of variation should be taken as the rutting of the section concerned. In this report, the rutting is represented by the mean value.

(3) Evenness (mm)

The evenness is provided by measuring the longitudinal form of the track of running of the left wheel on the traveling lane with a 3m profilometer, reading the values at a 1.5m interval of the form thus obtained and calculating the standard deviation of them to take it as the vertical profile. When measured with a 8m profilometer or any other measuring instrument, the value is converted with a conversion formula used.

1.2.2. Surface Characteristic Values and Pavement Performance

From the results of the surface maintenance survey containing the values of

measurement of the foregoing three characteristics as well as the overall evaluation by visual observation, there 1808 sets of data were obtained. The overall evaluation by visual observation is the rating of the performance of pavement through visual observation by the road administrator of the surface condition with respect to the items shown in Table-1, as below.

(Overall Evaluation)

A - No defect observed (good).

B - Some defects notes, but generally good.

C - Defects notes, but no repair required.

D - Minor repair required (patching, partial seal coat, etc.).

E - Major repair required (overlay, construction, etc.).

Table-2 is a list of the 1808 sets of data by district, and the data are obtained in a number commensurate with the extension of administration of the directly supervised national highways in the respective districts.

Fig.-1 shows the curves of the surface characteristic values against that of the cumulative relative frequency of the traffic volume of heavy vehicles at the time of the survey. Of the maintenance extension of about 17,000km of the asphalt pavement of the directly supervised national highways in the country except Okinawa, the sections of less than 1000 vehicles/day in one direction by the traffic volume of heavy vehicles accounted for about 34 percent, the sections of 1000∿3000 vehicles/day in one direction accounted for about 44 percent and the sections of 3000 or more vehicles/day in one direction accounted for about 22 percent, and the result of Fig.-1 agrees generally with such situation. Table-3 shows the actual conditions of the surface performance of a section of 1118km in Japan except for Okinawa, as measured during the period of from May 1978 to August 1979. There is no appreciable difference noted in the proportions by stratum of the cracking ratio and rutting.

Of 1808 sets of data, about 1100 sets of data were collected in and after 1977, comprising 210 sets of data under Overall Eavluation "A," 617 sets under "B," 248 sets under "C," 321 sets under "D" and 412 sets under "E." Thus, the data were the result of a survey carried out over as long a period as eleven years and were largely comprised of those obtained in recent years, and it was thought that there would be little deviation from area to area of investigation and due to traffic condition.

Fig.-2 shows the ratings of the overall evaluation as related to the surface characteristic values. As the surface characteristic values increase, the

proportion of data that are considered to be "A" or "B" in overall evaluation decreases, while that of the data considered to be "D" or "E" increases, and such trend is the largest in the case of the rutting. There are data of "D" or "E" in overall evaluation notwithstanding smaller surface characteristic values, and these are considered to be determined so because of other factors than the surface characteristic values.

1.3. Examination of Performance Evaluation Formula

1.3.1. Formulation

Using the said 1808 sets of data, formulation of the performance evaluation was examined. Here, the performance is more practically expressed in numerals than in categories, but for such purpose, it is necessary to express the results of overall evaluation in numerals. Then, the foregoing overall evaluation ratings were expressed in numerals as A=10, B=8 - - - - - E=2 as the performance would express, in a broad sense, the service level of road so that it would be convenient and readily understandable to decrease the value showing the performance with decreasing surface performance and that the indication in 10 steps would be adequate for expressing the size of such value.

For the performance evaluation formula, various functional forms are conceivable. But, a complex function is not practical. From the result of the AASHO road test, a functional form shown by formula (1) is provided, and the performance evaluation formulas proposed in Japan in the past are generally based on formula (1), with the coefficients only modified.

$$PSI = a_0 - a_1 \log\sigma - a_2 \sqrt{C} - a_3 D^2 \quad - - - - - - (1)$$

where PSI : Present Serviceability Index;

C : Cracking ratio (%);

D : Rutting (mm);

σ : Evenness (mm); and

a_i : Constant.

But, the formula is the one formulated about 20 years ago and is different in the method of evaluation from the surface maintenance survey, as shown in Table-4, so that an examination is necessary for direct use of the functional form, but it is not considered that the result of the examination will give a greatly different functional form from formula (1). Thus, with the data of measurement of the cracking, rutting and evenness converted exponentially and logarithmically, their relationship with the overall evaluation values was analyzed. First, the three characteristic values were

subject to exponential conversion in 10 kinds of multiplication of 0.2, 0.4 - - - - 2.0 by themselves respectively, and of 1000 cases obtained through combination of such values, multiple correlation coefficients with the overall evaluation values were obtained. As the result, the largest multiple correlation coefficients were obtained when the exponents of the cracking, rutting and evenness were 0.4, 0.6 and 0.2 respectively. Then, further changing the exponents by 0.05 within the ranges of 0.30∿0.50 for the cracking, 0.50∿0.70 for the rutting and 0.10∿0.50 for the vertical profile, multiple correlation coefficients were obtained. For the rutting and vertical profile, calculations were also made in the case of logarithmic conversion. But, for the cracking, the logarithmic conversion was not considered to be adequate in that the cracking ratio of the surface immediately after construction would be 0 percent and that it was known empirically that whether there was a cracking produced or not would have a great effect on the result of evaluation of the serviceability. On the other hand, the rutting and evenness immediately after construction would generally be 0∿3mm and 1.0∿2.0mm respectively so that it was considered that their relative changes after servicing would be significant if they were known to be 0mm or not. The multiple correlation coefficients thus obtained of 1563 cases are shown partly in Table-5. As seen, the largest multiple correlation coefficient was obtained when the exponent was 0.30 for the cracking, 0.70 for the rutting or 0.20 for the evenness. Then, as a formula for estimating the overall evaluation value upon the surface characteristic values, the following is considered:

$$H = 10 - a_1 C^{0.3} - a_2 D^{0.7} - a_3 \sigma^{0.2} \quad \cdots\cdots (2)$$

where H : Overall evaluation value; and

C, D, σ and a_i : Same as in formula (1).

The coefficient a_i in formula (2) should, therefore, be determined by multiple regression. But, as stated above, the evaluation of serviceability is apt to vary from the standpoint to standpoint of the evaluator, and such standpoints are classified largely into those of the road administrator, road users such as drivers and passengers, and roadside inhabitants and pedestrians, and a summary of the results of evaluation of the evaluators of the respective standpoints is considered to be a final performance evaluation. As the overall evaluation by the surface maintenance survey would be considered as an evaluation seen from the standpoint of the road administrator for determination whether the maintenance repair of the pavement

would be required or not, it was named, in order to distinguish from the final evaluation, as Maintenance Control Index. The regression formula of MCI is given as

$$MCI = 10 - 1.48C^{0.3} - 0.29D^{0.7}\ 0.47\sigma^{0.2} \quad ---- \quad (3)$$

where MCI = Maintenance Control Index; and

C, D and σ : Same as in formula (1).

The survey was made of various surfaces throughout the country, while no pre-training of the evaluators was made. Nevertheless, the multiple correlation coefficient of formula (3) was about 0.75. Thus, it was considered that the foregoing three characteristics were appropriately chosen to express the performance, that the method of numerical representation of the overall evaluation was adequate and that the MCI formula would be practically applicable.

Change of the MCI value with change of the respective surface characteristic values is shown in Fig.-3. As seen, the cracking ratio is the largest in the effect on MCI, and the evenness is not so much. From Fig.-1, the scope of application of the MCI formula is considered to be generally 40 percent or less for the cracking ratio, 35mm or less for the rutting or 5.0mm or less for the evenness.

For measurement of the surface characteristic values, a high speed measurement method has been established and data collection has been made in a large scale for the cracking and rutting, but not for the vertical profile so that it is often required to evaluate the serviceability of pavement upon two characteristics of cracking and rutting. Furthermore, on account of the traffics of larger and heavier vehicles in recent years, sections where only rutting is appreciable are noted increasingly. Formula (3) may be applied to such sections, but adequate evaluation is not always obtainable because of the other surface characteric values being small. For such case, a serviceability evaluation formula upon two or one characteristic values would be required. Thus, evaluation formulas on two characteristics and one characteristic were examined, and as a serviceability evaluation formula upon two characteristics of cracking index and rutting, formula (4) was obtained. Also, for the serviceability evaluation formulas upon the cracking ratio and rutting respectively, considering the effect of the other surface characteristic value to be constant from the overall evaluation value and by multiple regression, formulas (5) and (6) were obtained.

$$MCI_0 = 10 - 1.51C^{0.3} - 0.3D^{0.7} \quad - - - - - - - (4)$$

$$MCI_1 = 10 - 2.23C^{0.3} \quad - - - - - - - - - - (5)$$

$$MCI_2 = 10 - 0.54D^{0.7} \quad - - - - - - - - - - - (6)$$

where MCI_0 : Maintenance control index upon two characteristics;

MCI_1 : Maintenance control index obtained from the cracking ratio;

MCI_2 : Maintenance control index obtained from the rutting; and

C, D : Same as in formula (1).

To distinguish the values obtainable by formulas (4), (5) and (6) from the MCI stated above, they were represented by MCI_0, MCI_1, and MCI_2 respectively. Curves of MCO_0, MCI_1 and MCI_2 as a function of the respective surface characteristic values are shown in Figs.-4 and 5.

1.3.2. Discussion on MCI Formula

Formula (7) is the PSI formula shown in the Manual for Road Maintenance and Repair (1978 edition). It uses the functions obtained from the result of the AASHO road test for the surface characteristic values but has the coefficients modified as below:

$$PSI = 4.053 - 0.518 \log\sigma\, 0.371\sqrt{C} - 0.00174D^2 \quad - - - - (7)$$

where PSI : Present Serviceability Index; and

C, D and σ : Same as in formula (1).

Examining the relationship of MCI and PSI to the surface characteristic values, features of the MCI formula were considered. Fig.-3 shows the change of the evaluation value with change of the surface characteristic values in MCI and PSI formulas. MCI is of 10-step indication, while PSI of 5-step indication, so that the scales are adjusted, but in the greater part of the range which is considered to be of greater frequency of application from the results of Table-3, the change of the evaluation value by the respective surface characteristic values according to MCI formula is greater than that according to PSI formula. Particularly, when the surface characteristic values are smaller, the degree of change is greater, and as the surface characteristic values increase, the degree of change decreases. And when the value of cracking ratio, rutting or evenness exceeds about 32 percent, about 30mm or about 4.0mm respectively, the change by PSI formula becomes greater. Thus, compared with the PSI formula, the MCI formula is adapted to the cases where the surface characteristic values are small. In other words, the PSI formula is applicable to evaluation in good accuracy of the surfaces having cracking or rutting produced considerably, while the MCI is applicable to evaluation in good accuracy to the initial stage up

to the condition of PSI formula.

In the directly administered national highways, the average surface characteristic values of the pavement where the overlay is made are: cracking ratio, about 20 percent; rutting, about 20mm; and evenness, about 4.0mm. In such pavements, if the change of the evaluation value due to cracking is taken as unit, the changes of the evaluation value by the other two characteristics are 0.65 for rutting and 0.17 for evenness in the case of MCI formula but are 0.42 and 0.19 respectively in the case of PSI formula. Accordingly, the MCI formula has the effect of rutting enhanced relatively to the PSI formula, and when the drive safety against rutting, splashing to the sidewalk or other administrative problem is considered, it may be taken as an adequate evaluation method.

With respect to the evenness, both formulas are not much different from each other, and the effect on the evaluation value is smaller than the other two characteristic values are. Then, assuming the evenness as 4.0mm, the curves of MCI and PSI for the varying values of cracking ratio and rutting are shown in Fig.-6. The MCI value of the pavement of the cracking ratio at 15 percent and rutting at 0mm is about 6. Considering the case where this surface is degraded to an MCI of about 5, if the cracking only accounts for such degradation, the cracking index must be 37 percent or must increase by 22 percent, but for the rutting only, it must be great only by 6mm. Similarly, for the pavement of 0 percent and 20mm with MCI at about 7, if the rutting only accounts for the degradation, it must be 48mm or must increase by 28mm, but for the cracking ratio, it must be greater only by 3 percent. Such is because the MCI formula has importance attached to the cases where the surface characteristic values are small, as stated above, but is, at the same time, explained by the interaction taken into consideration in the formula. That is, when only one characteristic is considered, however great a value it shows, it is not taken that the performance is not good. But, in the case of two characteristics, if they show some appreciable values, the performance is considered to be bad. The interaction includes the synergetic and offset actions, and in the present case, it is apparently synergetic. But, in PSI, when the cracking ratio and rutting exceed about 5 percent and about 10mm respectively, the PSI equivalent curves are linear so that it may be said that the interaction is not much taken into consideration.

2. Conditions and Forecasts of Surface Performance

2.1. Actual Conditions of Surface Performance

The surface performance survey was conducted over an extension of 1,118km, and its classification by traffic and by category of serviceable years is as shown in Table-6.

Fig.-7 shows the distributions of the cracking ratio, rutting and MCI_0 by category of the traffic volume of heavy vehicles. In the distributions of the cracking ratio, no difference is observed between the distributions by category of the traffic volume of heavy vehicles, but in the case of the rutting, the proportion of 10~20mm is decreasing with increasing traffic volume of heavy vehicles. As the result, in the distributions of MCI_0, there is noted a trend that the sections of small MCI_0 increase with an increasing traffic volume of heavy vehicles.

Table-7 is a list of MCI_0 by the serviceable years. According to the table, the proportion of the data of MCI_0 being 3 or less is 6 percent. But, it was found that even if MCI_0 was greater than 3, the data of MCI_1 or MCI_2 being 3 or less were included for about 4 percent. Then, the proportion of the extension requiring repair immediately under the current condition is considered to be about 10 percent. Also, as the proportion of the data of MCI_0 being 4 or less is about 12 percent and that of the data of 5 or less is about 25 percent, it is considered that the extension requiring repair will increase by about twice for every increase by 1 of MCI as a standard of repair. Table-7 also shows the proportion by category of the serviceable years of MCI_0 being 3 or less. As seen, with elapse of time, the proportion increases, and upon an elapse of 5 years, more than 5 percent of the extension, and upon elapse of 10 years, more than 10 percent of the extention, degrades to 3 or less.

2.2. Forecasts of Surface Performance

2.2.1. When No Repair is Made

Predicting the cracking ratio and rutting by prediction formulas, the surface performance was forecasted until 10 years later. For the prediction, formulas (8) through (11) derived from the results of the surface maintenance survey were used:

$$C = 0.01Y^{3.25} \quad (Y \leq 10) \qquad \cdots\cdots(8)$$

$$C = 5(Y - 10) + 17.8 \quad (Y \geq 10) \qquad \cdots\cdots(9)$$

$$D = a_1 \cdot Y^{b_1} \quad (Y \leq 10) \qquad \cdots\cdots(10)$$

$$D = a_2(Y - 10) + b_2 \quad (Y \geq 10) \qquad \cdots\cdots(11)$$

$a_1 = 3.23\log T - 5.82$

$b_1 = 0.0168\log T + 0.673$

$a_2 = 1.17(\log T)^2 - 5.50\log T + 6.94$

$b_2 = 18.2\log T - 34.3$

where C : Cracking ratio (%);

D : Rutting (mm);

Y : Apparent serviceable years; and

T : Traffic volume of large vehicles (vehicles/day.one direction), provided, 5000 vehicles or more is taken as 5000 vehicles, and 250 behicles or less is taken as 250 vehicles.

The method of application of the prediction formulas is as follows. In Fig.-8, if the present cracking ratio is assumed to be 5 percent, then by formula (8), the apparent serviceable years is calculated to be 6.77 years. Then, the cracking ratio after one year is, if the apparent werviceable years is taken as (6.77 + 1) years, given from formula (8)as 7.8 percent. Distributions of the cracking ratio, rutting and MCI_0 until 10 years later are shown in Fig.-9.

In Fig.-9, the proportion of the data of the cracking ratio of 50.1 percent or more which is likely to give an MCI_0 of 3 or less is presently 1.0 percent. But, 3 years later, it increases to 2.8 percent, 5 years later, to 5.2 percent, and 10 years later, to 23.2 percent, initially at a rate of 0.5∿1.0 percent a year, and after 7 years, at a rate of 3∿4 percent a year. Similarly, the proportion of the data of the cracking ratio of 20.1 percent or more is presently 7.1 percent, but 3 years later, it increases to 17.7 percent, 5 years later, to 29.4 percent, and 10 years later, to 61.2 percent, initially at a rate of about 3 percent a year, and after 5 years, at a rate of about 8 percent a year.

For the rutting, the proportion of the data for rutting of 41mm or more which is likely to give an MCI_0 of 3 or less is presently 7.9 percent. But, after 3 years, it increases to 14.6 percent, after 5 years, to 21.0 percent, and after 10 years, to 45.2 percent, or at a rate of 3∿5 percent a year. Similarly, the proportion of the data of the rutting of 21mm or more is presently 54.1 percent, but after 3 years, it increases to 72.6 percent, after 5 years, to 85.4 percent, and after 10 years to 94.7 percent, at a rate of about 6 percent a year.

As the result, the proportion of the data of MCI_0 being 3 or less is presently 6 percent, but it increases by 3.3 percent average until 3 years

to 6 years later, and by 11.5 percent from 6 years later and after. The proportion of the data of MCI_0 being 4 or less is presently 12.4 percent, but it increases by 6.9 percent average until 3 years later, then by 11.3 percent from 3 years to 6 years later. The proportion of the data of MCI_0 being 5 or less is presently 24.8 percent, but it increases by 11 percent average until 6 years later.

Accordingly, assuming that the repair is made preferentially from the place where the MCI_0 value is small, in order to eliminate the places where MCI_0 is 3 or less, the proportion of the extension to be repaired until 3 years later will be 5.3 percent (= 6/3 + 3.3), that from 3 years to 6 years later be 8.1 percent, and that from 6 years later and after be 11.5 percent. Similarly, to eliminate the places where MCI_0 is 4 or less, the proportion of the extension to be repaired until 3 years later will be 11 percent (= 12.4/3 + 6.9), and thereafter be 11.3 percent, and to eliminate the places of MCI_0 5 or less, it will be about 15 percent (= 24.8/6 + 11).

2.2.2. Forecasts With the Current Level of Repair Made

It was assumed that the current level of repair would be made as shown below.

(i) The proportion of the repair extension is taken as 6.5 percent. Accordingly, of 11,180 sets of data for the extension of 1,118.0 km, repairs are made for 727 sets of data (11,180 × 0.065).

(ii) Sections where the serviceable years are less than 5 are not taken as the object of repair regardless of the size of any surface characteristic value.

(iii) The proportion of repair by cateogory of the serviceable years is as follows.

Serviceable years	5 ∿ 7	727 × 5% = 36 sets of data
	7 ∿10	727 × 15% =109 sets of data
	10 ∿15	727 × 45% =327 sets of data
	15 or more	727 × 35% =255 sets of data

(iv) Classification of repair by category of the serviceable years is as follows.

Serviceable years	5 ∿ 7	Data of MCI_0 4.5 or less.
	7 ∿10	Data of MCI_0 5.5 or less.
	10 ∿15	Data of MCI_0 6.0 or less.
	15 or more	Data of MCI_0 6.5 or less.

For the places to be repaired, weighting was made so that preference would be given to places of smaller MCI_0. Then, with the repairs carried out as

above, changes of the cracking ratio, rutting and evenness were examined. The results are shown in Fig.-10. In Fig.-10, the proportion of the places of $MCI_0 = 4$ or less is presently 12.4 percent, but 1 year after, it will be 17.3 percent. The diagram also shows that when the repair of the current level stated above is made after 1 year, MCI_0 decreases to 13.1 percent, and when another one year elapses, that is, after 2 years, it increases to 19.3 percent.

The proportion after repair of the data of the cracking ratio of 50.1 percent or more remains unchanged at 1.0 percent or less until 6 years later but thereafter increases slightly. The proportion after repair of the data of 20.1 percent or more is increasing, and the average rate of increase until 8 years later is about 1.4 percent.

The proportion after repair of the data of rutting of 41mm or more is increasing by 0.7 percent average until 9 years later. The proportion after repair of the data of 21mm or more is increasing by 1.8 percent average until 4 years later but is decreasing thereafter. Consequently, for MCI_0, the proportion after repair of the data of 3 or less is increasing by 2.3 percent average, that of 4 or less by 3.6 percent average, and that of 5 or less by 5.0 percent average.

Thus, when the repair of the current level is carried out, it has an effect, as compared with the case of no repairs made, of decreasing the proportion of the data of cracking ratio of 50.1 percent or more by about .1 percent a year and that of the data of 20.1 percent or more by 2~4 percent, and the proportion of the data of rutting of 41mm or more by about 3 percent and that of the data of 21mm or more by about 4 percent. Then, for MCI_0, the proportion of the data of 3 or less decreases by about 5 percent a year and both those of the data of 4 or less and 5 or less by about 6 percent, as compared with the case where no repair is made. Thus, it is considered that the repair of the current level, when it is executed, will not be able to maintain the present condition but has at least an effect of retarding the increase of the extension requiring repair.

2.2.3. Forecasts With the Repair Made of the Places of $MCI_0 = 4$ or Less

The places of repair are not chosen only by the surface characteristic values and performance despite the fact that the surface characteristic values and performance have a great influence on the selection of the places of repair, the maintenance and repair standards are subject to vary with the degree of importance of the route, roadside conditions, etc. Presently,

the places of repair are determined by the road administrator who considers the foregoing carefully, but it is scarcely considered that there is no room of improvement. Then, as an example, selecting the places of repair upon performance only, an examination was made of the changes of the surface characteristic values and performance with the repair extension rate changed so that the places of MCI_0 of 4 or less would be reduced to zero. The repair extension rate was increased or decreased stepwise upon the current base of about 6.5 percent. The results are shown in Fig.-11. Numerals in the diagram are the repair extension rates. By raising the repair extension rate to about 13 percent maximum and finally to about 10 percent, the places where MCI_0 is 4 or less may be reduced to zero.

3. Pavement Data Bank

3.1. Basic Concept

Photographing and other automatic methods have enabled the measurement of surface performance over a wide range recently. Also, through development of computers, it is now possible to process the data much more thoroughly. Presently, the Regional Construction Bureaus and Hokkaido Construction Bureau have respectively a data bank which is capable of providing integrated outputs and are using them.

3.1.1. Object and Positioning

The pavement data bank is intended to maintain the basic data on pavement in an adequate form by computer processing to obtain statistic data on pavement and provide data for formulation of maintenance plans, as required. For the road in general, not only the data bank on pavement but many other data banks are conceivable, as shown in Table-8, and some of them are already operated. In the national highways under the Government's direct control, it is the simplest and most practical way to associate these data with one another by means of route name and kilometer marker. Thus, in the pavement data bank, the method of associating the data with one another by means of route name and kilometer marker is employed.

3.1.2. Input Information

As the operation is still in the early stage, only those data which are necessary presently are taken. Such data are shown in Table-9.

3.1.3. Output Formats and Applications

The output is tentatively provided in the following five formats.

Format 1 Surface performance data list.

2 Performance level diagram.

3 Frequency distribution table.

4 Histogram.

5 Maintenance repair sections list.

Formats 1 and 2 are adapted for output by work office, detachment and route, and Formats 3 and 4 are adapted for output by surface type and item of investigation (cracking, mean rutting, evenness and MCI) in addition to the foregoing. Format 5 is enabled to set the guide line as desired.

The outputs in these formats are usable as data for grasping the conditions of the sections under control, examination of the construction methods and preparing data for budgetary appropriations at the regional construction bureaus, work offices and detachments.

It is also expected that the unique methods of application to the respective regional construction bureaus will be developed and further that all data are managed at the Road Bureau, etc. for use on a countrywide level.

As the data are accumulated increasingly, they will permit a long-ranging analysis to be used effectively for examination of the pavement structure. At the same time, it will be enabled to obtain all sorts of data easily through connection with the other data banks.

3.2. Outline of the System

The system is represented in flow charts shown in Figs.-12 and 13 and is outlined by job as below.

I. Job (1) procedure

(1) Check and conversion 1)

Input the control and characteristic data and work and repair data 1) and 2); check them if they are adequate; and for any error data, print out the details in a check list.

(2) Data bank updating 1)

(a) Upon the processing codes "N (Input)," "S (Modify)" and "K (Erase)," registration, alteration and deletion of the control and characteristic data and work and repair data 1) and 2) are made to the bank. At this time, KP (kilometer post) (from) and KP (to) in the bank are referred to those of the respective data, and inadequate data are printed out in an update list 1).

(b) Registered, altered or deleted contents are printed out in the update list 1) for proofing.

Note: In the case of an error in any of the data, the error data is corrected and is processed again according to (1) above.

II. Job (2) processing

(1) Check and conversion 2)

(a) Input the surface performance data; check their adequacy and for any error data, print out the details in a check list.

(b) For data without errors, prepare a KEY for data bank, prefix it to the data, and output the data to an intermediate file 2).

Note: If the data has an error, the data is corrected and processed again before proceeding to the subsequent step.

(2) Data bank updatins 2)

(a) The surface performance data in the bank have the routes and KP (from) - KP (to) ranges correctively or additionally registered in reference to the surface performance data in the intermediate file 2).

(b) The new and old data of the correctively or additionally registered ranges are printed out in the update list 2).

III. Job (3) procedure

(1) Document file processing

(a) Retrieving the codes conforming to the conditions of the output indicated parameters from various data in the bank, document file is prepared.

(b) Computation of the values of forecast is made simultaneously.

Note: It is intended to carry out the surface performance survey at a rate of once in every three years, with forecasts using the prediction formulas for the places not investigated.

(2) Document writing and plotting

(a) Of the output formats 1, 3 and 5, those specified by the output parameters are written.

(b) Of the output formats 2 and 4, that specified by the output parameters is written.

IV. Job (4) processing

(1) Verification list writing

Input data from the data bank specified by the classifying codes of the verification parameters, extract data of the same route number and within the range of KP (from) to KP (to), and output to the verification list.

V. Job (5) processing

(1) Data bank updating 3)

(a) Of those data in the bank which are specified by the classification codes of inbound or outbound, and lane number change card, all of the data equal in the route number, old inbound or outbound classification and old

lane number and within the range of KP (from) to KP (to) are substituted with new inbound or outbound classification or lane number.

(b) The alteration is output to the alteration list.

VI. Job (6) processing

(1) Data bank name information maintenance

(a) Name information of the banks is registered, altered or deleted according to the processing code "N," "S" or "K" on the maintenance card.

(b) Details of maintenance are listed and output.

VII. Job (7) processing

(1) Storage of pavement data bank

The data bank in the disc is stored on magnetic tape.

If the road administration is practicable in such form, the collected data can be stored effectively, and in the case of an increase of data to be collected later, the bank can be readily expanded. Also, when it is desired to connect to the other data banks which are completed later for an overall analysis, the connection will be made with relative ease to permit retrieval so that the work will be carried out as desired.

3.3. Forecast of Surface Performance

If it is possible to forecast the change with time of the surface performance by means of influential factors, it will be very useful for implementing the pavement maintenance and repair plan. As such factors, the subgrade condition, pavement construction, traffic condition, road environment, type of asphalt mixture, years of service, etc. are considered. But, such factors include those which are hardly quantified, while the surface condition is changing every minute under the combined influence of these factors.

As a nation-wide prediction formula, there is an exponential formula obtained of the average changes of the cracking ratio and other surface performance values from the years of service by the least square method. But, when this formula was tested in unit of the regional construction bureau, there were problems of accuracy. Then, upon the consideration that it would be convenient if forecasting be made within the range of practical accuracy in use of the currently available data, the following prediction formulas were provided. Prediction formulas of the cracking ratio in a certain regional construction bureau are shown below.

$C_{i+1}=0.28+1.13C_i$ (Asphalt pavement, normal area)

$C_{i+1}=0.03+1.30C_i$ (Asphalt pavement, snowy cold area)

$C_{i+1}=0.39+1.09C_i$ (Concrete pavement)

Here, Ci represents the cracking ratio in i-th year, and Ci+1 represents the cracking index in i+1-th year.
The factors used now mainly are the type of pavement (asphalt or concrete), areal classification (normal or snowy cold area), years of service, and the values of cracking ratio, evenness and rutting (maximum and mean values) in the previous survey. But, they are generally calculated by multiplying the results of the previous survey by some coefficients so that for the pavement installed newly or immediately after repair, it is required to set initial values.

3.4. Choice of the Maintenance and Repair Method

When the maintenance repair of pavement is made, it must be planned with consideration given to the change of performance after repair. For example, in the following cases A∿D, the life cycle of the pavement will be as shown in Fig.-14.
In a newly installed pavement, if it is assumed that the overlay is made twice before reconstruction is made, the combinations shown in Table-10 are considered.

Case A : At MCI = 5, overlay is made in a thickness of 25 percent of T_A at the time of initial installation.

Case B : At MCI = 5, overlay is made in a thickness of 10 percent of T_A at the time of initial installation.

Case C : At MCI = 3, overlay is made in a thickness of 25 percent of T_A at the time of initial installation.

Case D : At MCI = 3, overlay is made in a thickness of 10 percent of T_A at the time of initial installation.

Now, examining the total cost and performance of those 8 cases which are framed in these 16 cases, "BB" and "DD" are nearly the same in the total cost. But, for the level of serviceability during the period, the case "B" is able to secure an MCI level of 5 or higher, but the case "D" has MCI degraded to 3. Then, to take such difference into consideration, there was considered an index calculated by the formula

$$P = \Sigma(Mi \times Pi) \quad \text{-----------------} (12)$$

where P : Cost index;

Pi : Cost of maintenance and repair in a certain year (Yen/m^2); and

Mi : Mean degradation of MCI in a certain year.

The cost index calculated by formula (12) is a cumulative product of the mean of MCI degradation and that of the maintenance repair cost and is thus

an index enabling comparison of various life cycles from both aspects of total cost and serviceability level.

The results of calculation are shown in Fig.-15. From the diagram, it may be said that "BB" is the best with low cost index and high performance level. Accordingly, by an overlay made in a thin layer at an earlier time, the total cost is reduced, and the performance is maintained in a good level.

Summary

A performance evaluation formula (MCI) was provided, and upon the MCI formula, forecasting of the surface performance of national highway was made. Also, the data bank investigated simultaneously with the present study was described briefly. The study presented here is still in an intermediate stage, while the maintenance repair of the pavement is to be made repeatedly and is increasing in volume with installation of new pavements every year. It is expected that furtherance of the study will enable more exact control of pavements.

Table-1 Items of Observation

Items		Contents
Texture of the surface		Very Coarse
		Coarse
		Aggregates Appear
		Smooth
Uniformity of the Surface		Good
		Medium
		Bad
Asphalt Content		Much Flush
		Some Flush
		Possible to Flush
		Black Surface & No Flush
		Brown and Dry Surface
Skidness		Not Slippery
		Medium
		Slippery
Deformation and Rutting		Much
		Some
		No
Stripping and Pot Hole		Much
		Some
		No
Wearing		Much
		Some
		No
Cracking	Line type	Much
		Some
		No
	Net type	0%
		0- 5%
		5- 10%
		10- 20%
		20- 50%
		50-100%

Table-2 The Observations of Each District

(Times)

Kind of Survey / District	Basic Survey	General Survey	Total
Hokkaido	82	267	349
Tohoku	116	75	191
Kanto	83	88	171
Hokuriku	49	87	136
Chubu	119	45	164
Kinki	88	85	173
Chugoku	148	104	252
Shikoku	83	89	172
Kyushu	130	70	200
Total	898	910	1808

Table-3 Actual Road Condition in Japan

Rutting (mm) / Cracking (%)	0-10	11-20	21-30	31-40	41-50	51 or More	Total Length	Total Percentage (%)	Total Cumulation(%)
0	576	1605	1144	755	214	41	4335	38.8	38.8
0.1- 5	537	1770	1168	731	231	44	4481	40.1	78.9
5.1-10	131	300	243	133	42	8	857	7.7	86.5
10.1-20	103	255	176	123	49	18	724	6.5	93.0
20.1-30	45	100	101	85	28	6	365	3.3	96.3
30.1-40	8	47	60	46	24	8	193	1.7	98.0
40.1-50	10	26	30	31	11	3	111	1.0	99.0
50.1	6	22	33	43	9	1	114	1.0	100.0
Length	1416	4125	2955	1947	608	129	11180		
Percentage	12.7	36.9	26.4	17.4	5.4	1.2		100.0	
Cumulation	12.7	49.6	76.0	93.4	98.8	100.0			

Unit 0.1Km

Table-4 Method of Estimation of Road Surface Characteristics

	AASHO Road Test	Surface Maintenance Survey
Period	1958-1961	1969-1979
Persons Who Estimated the Characeristics	Road Engineers Material Producers Automobile Producers	Public Officers, the Ministry of Construction
Method	Estimation by Running Test	Estimation by Observation

Table-5 Multiple Correlation Coefficient

Cracking \ Evenness \ Rutting		$D^{0.6}$	$D^{0.7}$	D	$D^{2.0}$
$C^{0.2}$	log σ	0.740	0.732	0.739	0.723
	$\sigma^{0.1}$	0.739	0.733	0.735	0.723
	$\sigma^{0.2}$	0.739	0.732	0.738	0.725
	σ	0.738	0.732	0.737	0.725
$C^{0.3}$	log σ	0.747	0.742	0.743	0.733
	$\sigma^{0.1}$	0.7488	0.7490	0.744	0.733
	$\sigma^{0.2}$	0.7490	0.7492	0.744	0.733
	σ	0.7491	0.743	0.745	0.733
$C^{0.6}$	log σ	0.725	0.719	0.722	0.706
	$\sigma^{0.1}$	0.724	0.716	0.717	0.702
	$\sigma^{0.2}$	0.724	0.716	0.727	0.705
	σ	0.721	0.716	0.717	0.699
C	log σ	0.680	0.674	0.676	0.656
	$\sigma^{0.1}$	0.669	0.661	0.662	0.640
	$\sigma^{0.2}$	0.674	0.665	0.670	0.650
	σ	0.663	0.660	0.658	0.631

Table-6 Length of the Observed Roads

Years in Use / Traffic Classification	0-5	5-7	7-10	10-15	15-	Total Length	Total Percentage (%)
L,A,B	72.8	25.6	38.8	96.6	15.8	249.6	22.3
C	246.2	109.8	88.6	106.8	47.8	599.2	53.6
D	110.4	63.2	70.6	21.8	3.2	269.2	24.1
Total Length	429.4	198.6	198.0	225.2	66.8	1118.0	
Percentage	38.4	17.8	17.7	20.1	6.0		100.0

Unit Km

Table-7 Actual Condition of Performance of Road Surface

Years in Use / MCIo ranking	0-5	Cumu	5-7	Cumu	7-10	Cumu	10-15	Cumu	15-	Cumu	Total Length	Total Percentage	Total Cumulation
10.0-9.6	0	4294	0	1986	0	1980	0	2252	0	668	0	0.0	100.0
9.5-9.1	13	4294	1	1986	6	1980	4	2252	5	668	29	0.3	100.0
9.0-8.6	246	4281	36	1985	49	1974	41	2248	7	663	379	3.4	99.7
8.5-8.1	521	4035	108	1949	115	1925	134	2207	36	656	914	8.2	96.4
8.0-7.6	584	3514	146	1841	119	1810	135	2073	26	620	1010	9.0	88.2
7.5-7.1	649	2930	176	1695	148	1691	170	1938	25	594	1168	10.4	79.1
7.0-6.6	663	2281	250	1519	217	1543	214	1768	64	569	1408	12.6	68.7
6.5-6.1	512	1618	284	1269	274	1326	281	1554	68	505	1419	12.7	56.1
6.0-5.6	339	1106	225	985	227	1052	286	1273	59	437	1136	10.2	43.4
5.5-5.1	265	767	188	760	215	825	214	987	67	378	949	8.5	33.2
5.0-4.6	177	502	171	572	204	610	196	773	65	311	813	7.3	24.8
4.5-4.1	136	325	126	401	112	406	147	577	50	246	571	5.1	17.5
4.0-3.6	70	189	97	275	97	294	120	430	38	196	422	3.8	12.4
3.5-3.1	33	119	73	178	57	197	83	310	47	158	293	2.6	8.6
3.0-2.6	34	86	46	105	51	140	83	227	38	111	252	2.3	6.0
2.5-2.1	23	52	31	59	32	89	52	144	35	73	173	1.5	3.7
2.0-1.6	14	29	22	28	31	57	35	92	19	38	121	1.1	2.2
1.5-1.1	7	15	3	6	21	26	37	57	7	19	75	0.7	1.1
1.0-0.6	8	8	3	3	3	5	13	20	8	12	35	0.3	0.4
0.5-0.1	0	0	0	0	2	2	7	7	4	4	13	0.1	0.1
Total	4294		1986		1980		2252		668		11180	100.0	
A (%)	2.0		5.3		7.1		10.1		16.6		6.0		
B (%)	0.8		0.9		1.3		2.0		1.0		6.0		

Unit 0.1Km

A:Percentage of which MCIo is below 3 to the total length of each year calssification
B:Percentage of which MCIo is below 3 to the total length

Table-8 Classification of Data Bank

No.	Category	Input Information	Usage
1	Road Administration	Road Classification, Origin & Destination, Actual Length, Length of Each Structure, Length of Sidewalk, Date of Commencement of Service, Street Trees	Road Planning Road Statistics
2	Road Traffic	Geometric Design, Width, Traffic Volume	Partial Reconstruction, By-pass Highway Planning, Calculation of Road Improvement Level
3	Pavement	Pavement Structure, Traffic Volume, Road Surface Characteristics	Planning of Pavement Maintenance
4	Road Environment	Traffic Volume, Roadside Condition Noise, Vibration, Air Pollution	Countermeasures to Road Public Hazard
5	Traffic Safety	Geometric Condition, Intersection (Structure of Intersection), Rate of Accident, Safety Equipment, Traffic Sign	Planning for Road Safety Improvement
6	Bridge	-----	
7	Tunnel		

Table-9 List of Input Information

1 Data for Road Administration
1. Route No.
2. Name of Construction Branch Office
3. Adress
4. Distance Between Kilometer Marker

2 Peculiar Data of Each Highway
1. Number of Lines
2. Road Classification by Traffic Volume of Heavy Vehicles
3. CBR of Subgrade
4. TA (Equivalent Thickness to As. Layers)
5. Distinction of Areas (1)
6. Distinction of Areas (2)
7. Road Equipments
8. Date of Construction
9. Existence of Intersection

3 Data of Road Surface Characteristics
1. Cracking Ratio
2. Rutting
3. Evenness
4. Asphalt/Concrete

4 Construction and Maintenance Data
1. Construction Technique
2. Thickness of Pavement
3. Materials
4. Pavement Structure
5. Treatment on Subgrade
6. Measured CBR of Subgrade
7. Measured TA

Table-10 Combination of Pavement Life Cycle

The 2nd Overlay / The 1st Overlay	Case A	Case B	Case C	Case D
Case A	A A	A B	A C	A D
Case B	B A	B B	B C	B D
Case C	C A	C B	C C	C D
Case D	D A	D B	D C	D D

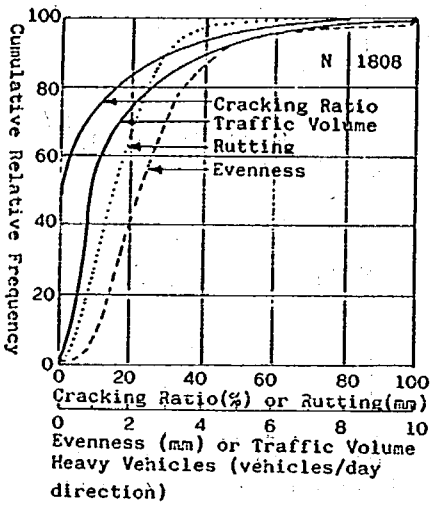


Figure-1 Cumulative Relative Frequency Distribution of Road Surface Characteristics

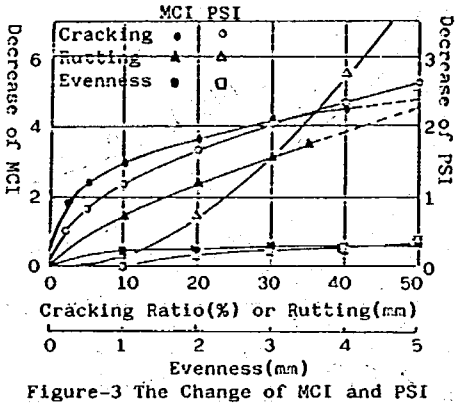


Figure-3 The Change of MCI and PSI by Road Surface Characteristics.

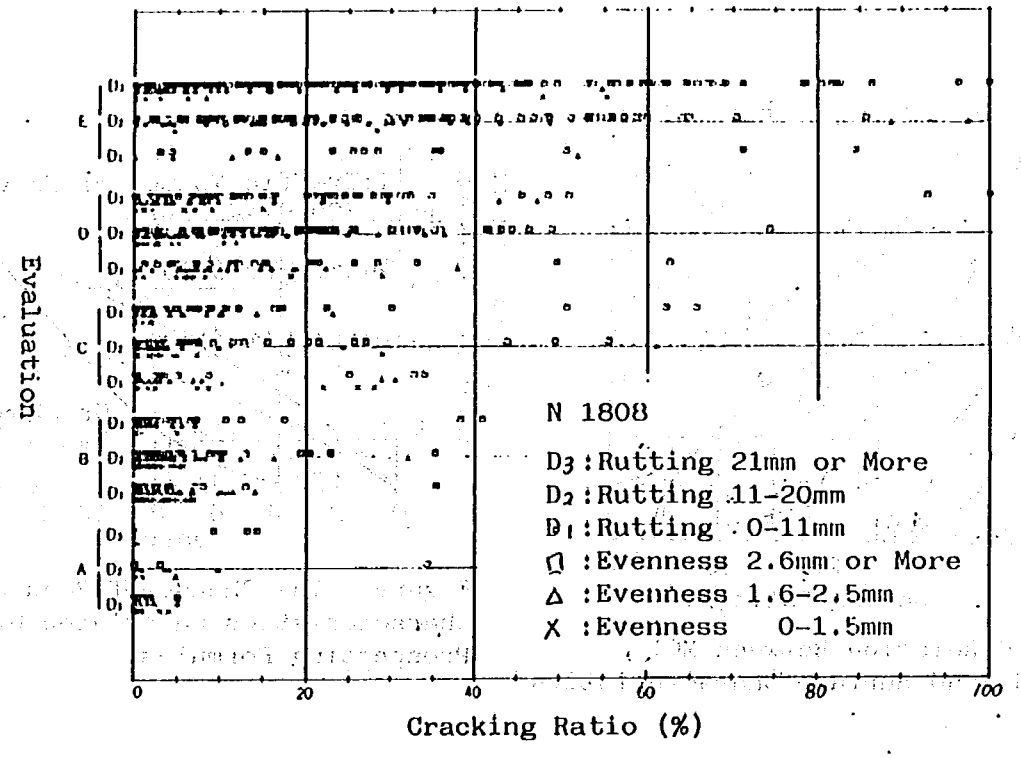


Figure-2 Relation Between Evaluation and Cracking Ratio

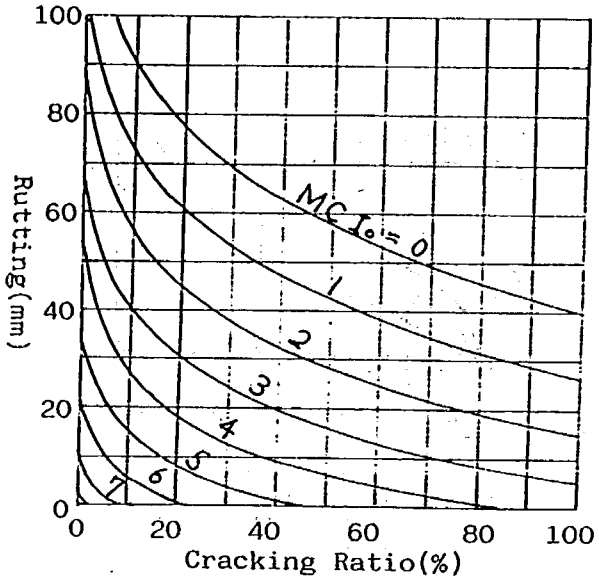


Figure-4 Relation Between MCI_0 and Road Surface Characteristics

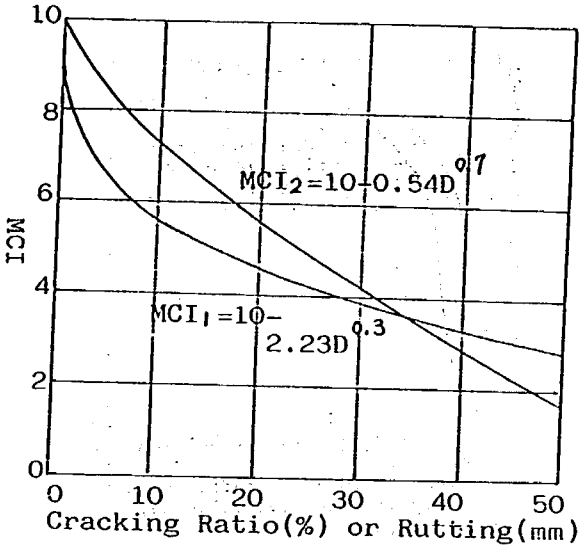


Figure-5 Relation Between MCI_1 or MCI_2 and Road Surface Characteristics

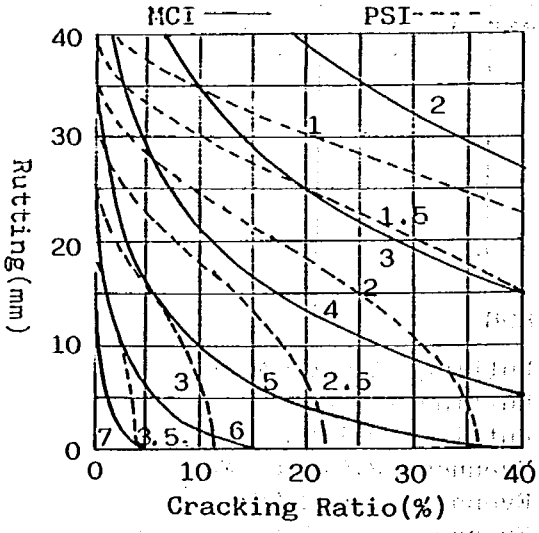


Figure-6 Relation Between MCI, PSI and Road Surface Characteristics

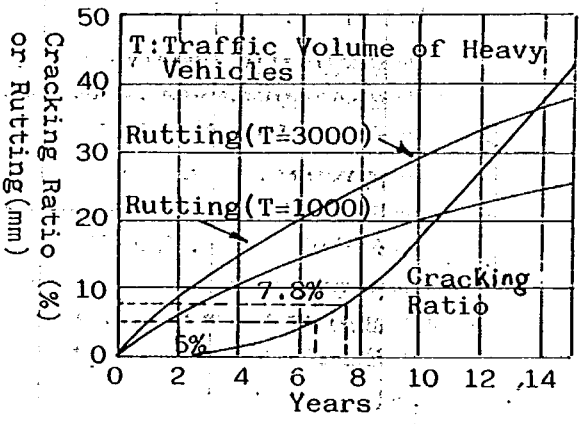


Figure-8 The Change of Road Surface Characteristics Calculated by Prospecting Formulas

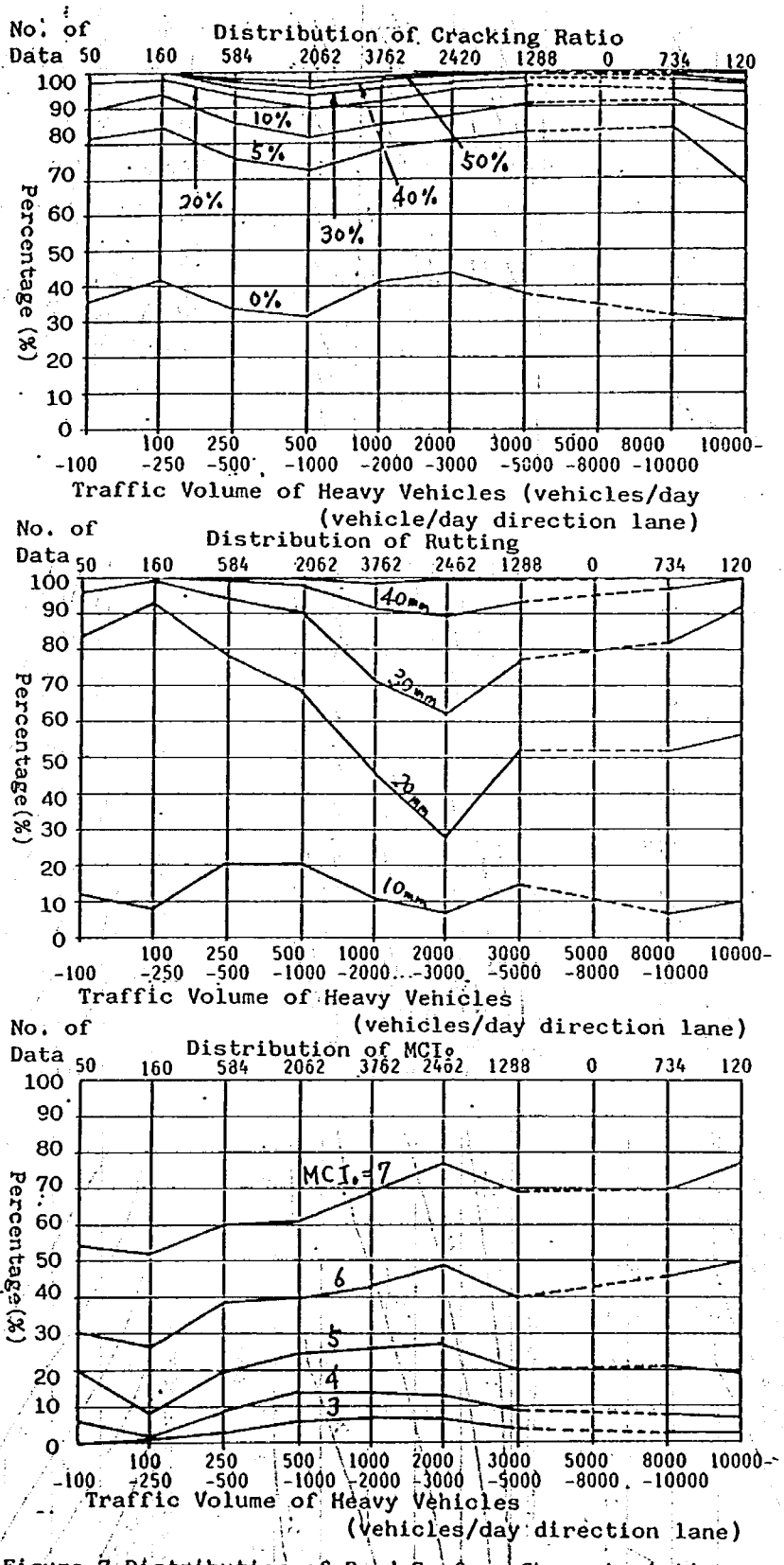
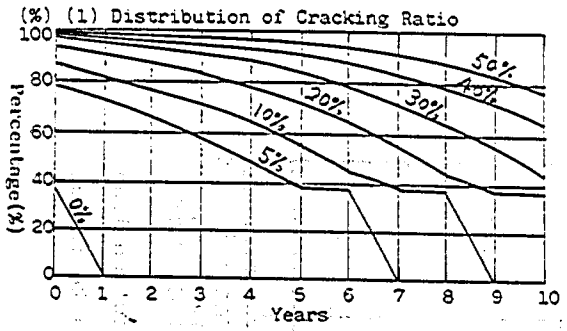
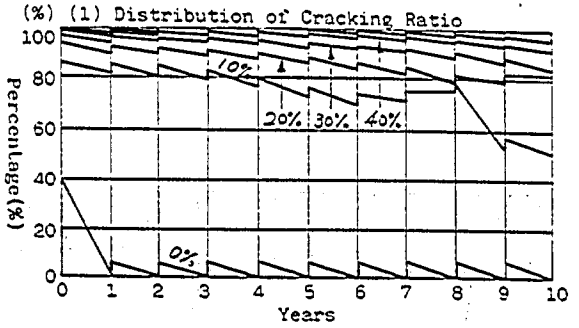
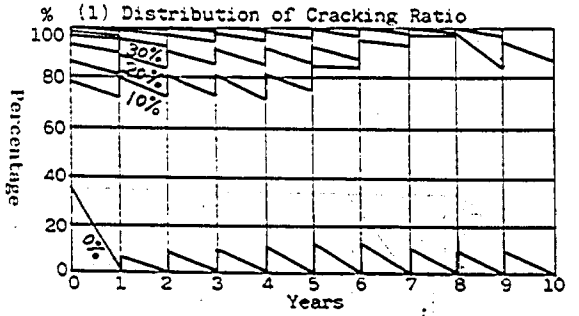
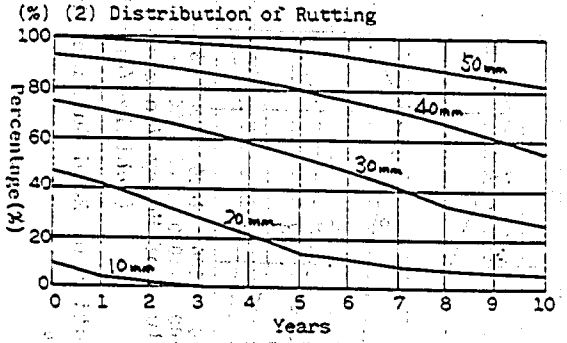


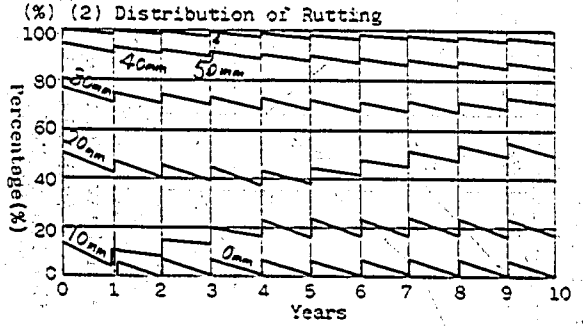
Figure-7 Distribution of Road Surface Characteristics and MCI_0

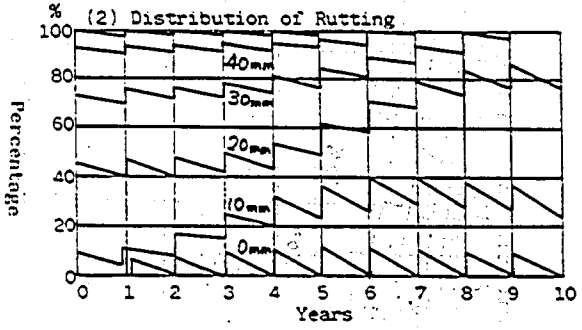












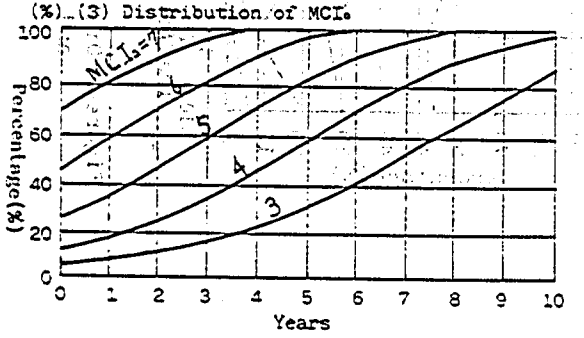


Figure-9 The Change of Road Surface Characteristics and MCI (No Repair)

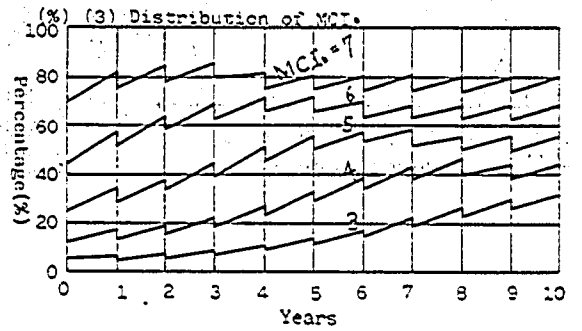


Figure-10 The Change of Road Surface Characteristics (Repair of Present Level)

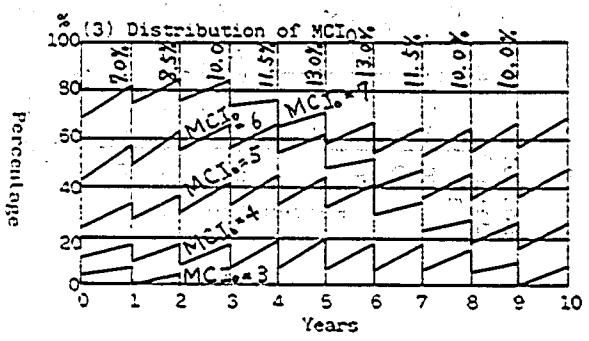


Figure-11 The Change of Road Surface Characteristics and MCI₀
(Roads of which MCI₀ are 4 or less are repaired)

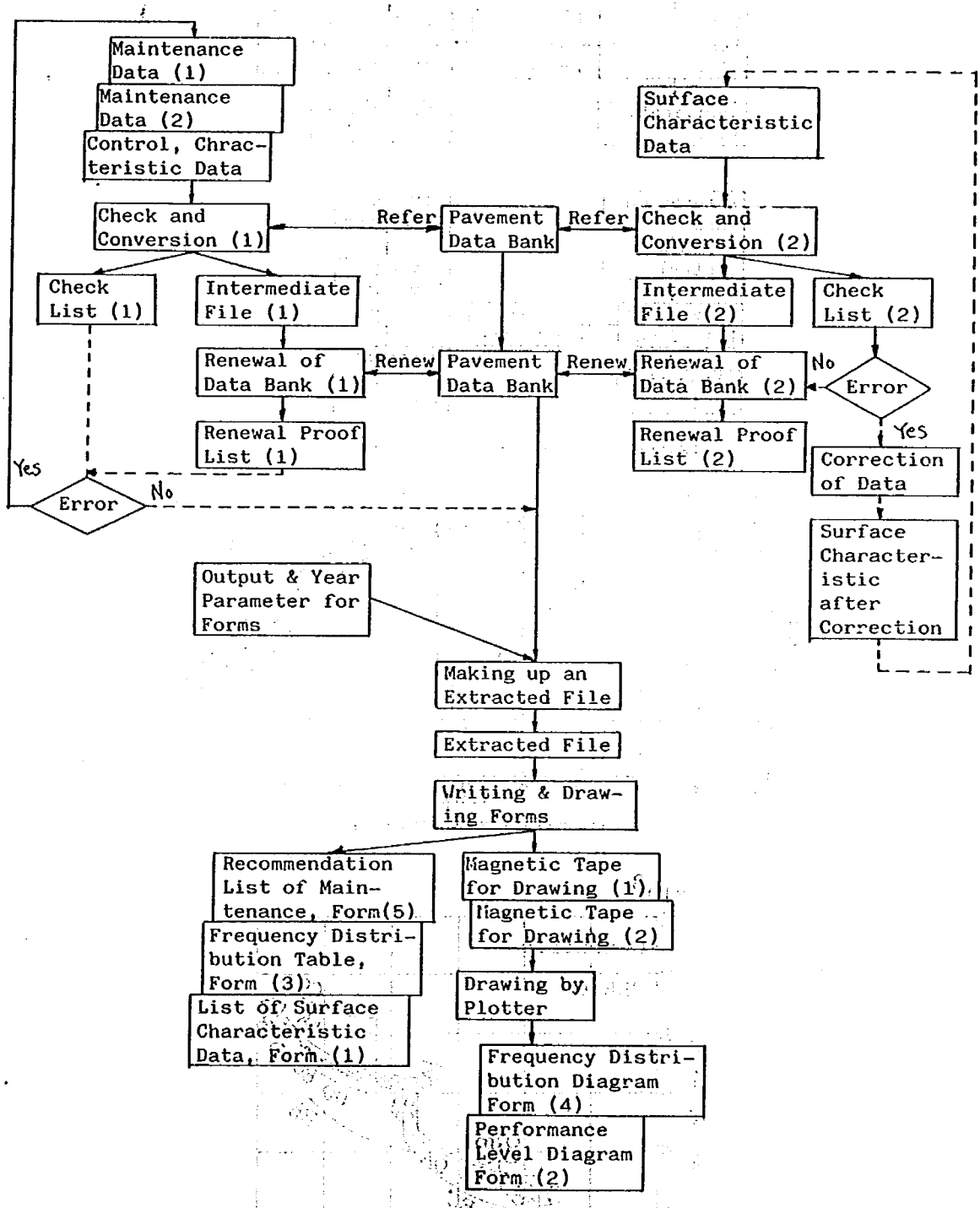


Figure-12 Main Flow of the System

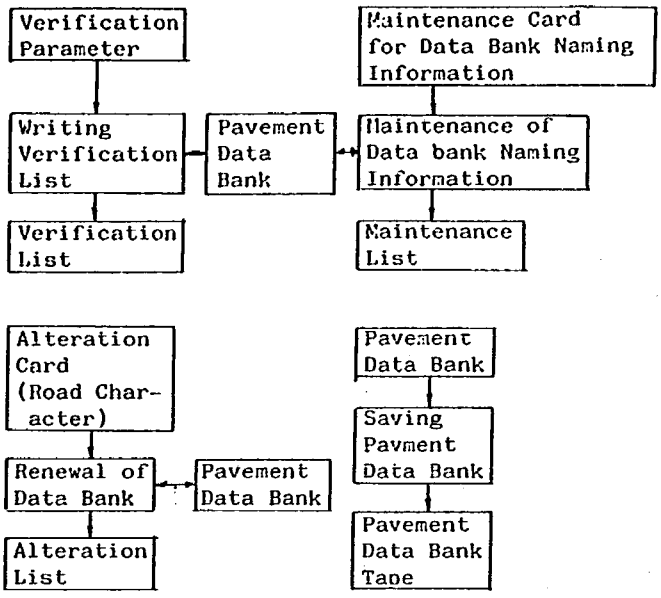


Figure-13 Sub-Flow of the Syetem

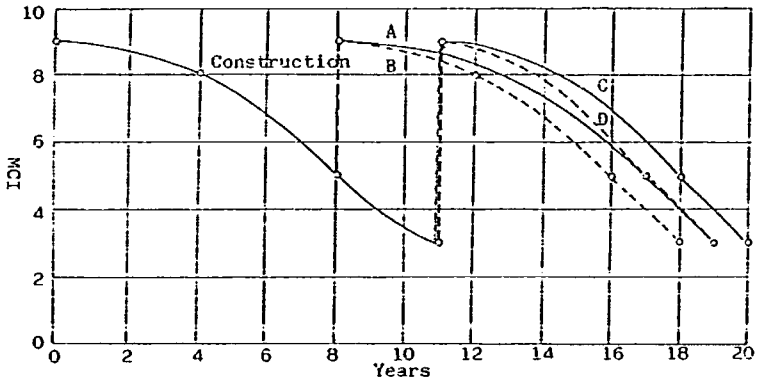


Figure-14 Pavement Life Cycle

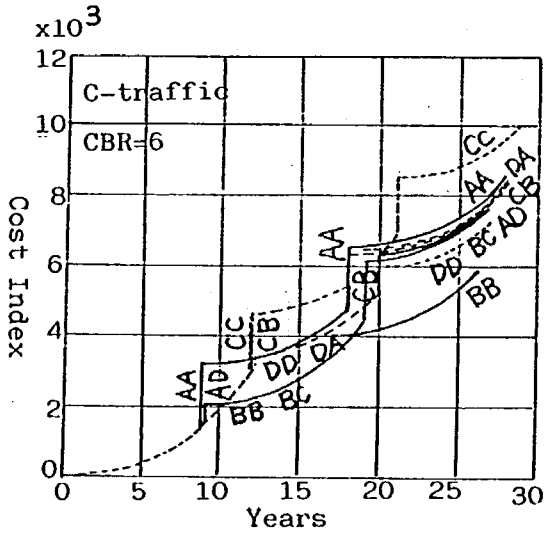


Figure-15 Comparison of Cost Index

Serviceability Evaluation and Rehabilitation Method for Street Pavements under Heavy Traffic

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ABSTRACT

In Tokyo Metropolis, The enormous city concentrated population and various industries, the street pavement under the heavy traffic is in need of keeping the serviceability and preventing outbreaks of several obstacles to roadsides being due to the automobile traffic.
Consequently, the serviceability evaluation of these pavements must be considered based upon two big requirements for the automobile traffic and the influence to roadside. Further, the works for evaluating and rehabilitating on these pavements must be performed rationally and rapidly during a short time at night decreasing automobile traffic.
Meanwhile, on considering the method of maintenance and repair, it is necessary that the pavement management system is established from a public standpoint.
In this paper, as the means of settling against these subjects, the actual conditions of the asphalt pavement is discussed, and the suggestions on the method of maintenance and repair is done based upon the experimental pavements and the outcomes of pursued investigations.

SERVICEABILITY EVALUATION AND REHABILITATION METHOD FOR STREET PAVEMENTS UNDER HEAVY TRAFFIC

by
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The Institute of Civil Engineering of
The Tokyo Metropolitan Government

1. Introduction

As the base of various city activities, the pavement of streets in our capital Tokyo, which is a gigantic city densely populated by both people and industry, is subjected to heavy traffic both day and night. These streets are not only used for traffic also from urban and besides contain such underground equipment as gas-supply, water-supply, drainage, electricity and telephone cable. They also function in many ways as public spaces in emergencies, while providing the place of living for the inhabitants at the roadsides.

Lately we have been receiving an increasing amount of claims arising from vibration and noise caused by vehicles running on uneven surfaces. This means that the maintenance and repair of pavements must be made for the purpose of maintaining the living condition of the roadsides and at the same time of providing safety and comfort for the vehicles running on the streets. Threfore, to evaluate pavement serviceability, it is necessary to use the evaluation method which reflects both safety and living condition. With these conditions,we will report the method of evaluation and that of maintenance and repair of asphalt pavement based on actual measurements.

2. Destraction and Evaluation Factors of the Street Pavements

Through more than ten years of various researchs, we have chosen three factors stated below as our evaluatuion factors of pavements.

* Evaluation of the pavement surface, which determines safety and comfort for running vehicles
* Evaluation of living condition of the roadside
* Evaluation of pavement structural capacity

Each factor is examined by the following items:

a) roughness and faulting

b) rutting

c) cracks and pot-holes

d) skid resistance

e) vibration for evaluation of living conditions of the roadside

f) deflection and deflection basin (as which we took Dynaflect deflection)

2.1 Evaluation of the Pavement Surface

2.1.1 Roughness and Faulting

Roughness works as a compulsory force to the running vehicles, reaches the riders as a vertical vibration to which it has been transfered through the underspring/downspring-mass system, and eventually causes the loss of the comfortable condition of the riders. This vehicle vibration not only works as an impact force to the pavement but also causes some damage to the houses and unpleasantness to the people at the roadside, as being propagated through the pavement-ground system. The relation between roughness and vibration is indicated in Figure 1. Though you may recognize some scattering in the figure. Nevertheless, it can be concluded that roughness is one of the main causes of vibration[1], and it follows that roughness is an important item of evaluating comfort of running vehicles and the living condition of the roadside.

As roughness is firstly determined by its figure and condition right after construction and is usually changed according to the influence of the weather and the age of the pavement[2] or even deteriorated by faulting and roughness made by imperfect underground reconstruction, it is most important to have a thorough and complete construction of the surface course and enough compaction on each layer.

2.1.2 Rutting

It is said that rutting has smaller effects on riding quality than roughness, but as it happens at the wheel trace of heavy vehicles (see Figure 2), they give instability and incecurity to the perform-

ance of the vehicles of smaller sizes and to the steering at the point of changing of lanes. When a rutting becomes bigger and makes a pool with water from the rain, it brings out such harm as splashes to both vehicles and the roadside. Figure 3 shows the relation between the numbers of passing heavy vehicle and rutting at the entrance of an intersection and at the road section of uninterrupted flow. You can see that we should be careful at the decrease of skid resistance caused by the pool at the entrance of intersections, where braking resistance is required to be big, since both the depth and alteration ratio are bigger there than at the road section of uninterrupted flow.

2.1.3 Cracks and Pot-holes

Cracks of small width without faultings will make almost no effect on performance or on vibration outbreak, as shocks are softened by the property of involution of tires. Nevertheless they happen to cause deterioration of performance or outbreak of vibration when invation of water from cracks makes further failure of the pavement. The degree of aggravation of cracks is the important item of evaluation of pavement bearing capacity.

2.1.4 Skid Resistance

Skid resistance of the pavement surface is the dominating factor in the braking effect of vehicles. It is also an important item in respect of safety for both pedestrians and running vehicles . According to a survey, the traffic accident rate at the entrance of an intersection with anti-skid pavement is reported to be decreased by 32% on a sunny day and by 86% on a rainy day; it also reports that the accident rate between pedestrians and vehicles at pedestrian crossings is reduced by 67%[3].

2.2 Evaluation of The Living Condition of The Roadside (Vibration)

To evaluate the living condition of the roadside, air pollution caused by the exhaust gas, is also considerable besides noise and vibration. However, we have chosen vibration because air

pollution and noise cause minor effects on the roadside human life compared with vibration. Traffic vibration is a very complex phenomenon: it determines its size according to the machanism and the speed of vehicles, to the structure and the condition of the pavement surface, and to the vibratility of foundations (see Figure 4); it changes its recievability according to the structure of the houses and to the sensibility of the inhabitants. Therefore it is a difficult task to predict the vibration intensity or to decrease it.

The necessary items to predict traffic vibration are indicated in Table 1. We have examined the relation between each item and vibration by the multiple regression analysis, and have chosen items whose correlation coefficient is large in order to get the predictive equation of the traffic at Kanto-loam upland[4]. Equation 1 means the equivalent vibration level (Lequ) at the predictive position of each lane, and L_{10} at the predictive position can be obtained from equation 2.

$$L_{equ}(k)=23.3\log\sigma-19.1\log T_A+18.6\log V+8.2\log Q_1 +1.7\log Q_2-5\log(R+1)-0.09R+27.9 \quad (1)$$

$$L_{10}=10\log\sum_{k=1}^{N}10^{\frac{Lequ(k)}{10}}-1.5 \quad (2)$$

where, k: lane
σ: roughness (mm)
T_A: relative thickness on the assumption of making every layer of the pavement with surface-course and binder-course hot-asphalt mixture (cm)
V: average running speed (km/h)
Q_1: traffic volume of heavy vehicles (/h)
Q_2: traffic volume of light vehicles (/h)
R: distance from the roadway side (m)

The correlation coefficient between the actually measured value at 39 positions and the predicted value from the equation was 0.85, and the standard deviation of the difference was 2.7 dB. Therefore to decrease vibration at roadside, we have to take away the roughness and faulting of pavement surface, to strengthen the pavement bearing capacity and to widen the sidewalk itself.

2.3 Pavement Structural Evaluation

Deflection and deflection basin are the items to know the load-carrying capability of the whole pavement including subgrade and the level of contribution of each layer. The change of deflection from the construction or reconstruction of pavements to another reconstruction can be described with the bath-tab curve line as indicated in Figure 5, taking the accumulated heavy comercial vehicles for the holizontal axis. This means that until a certain point of traffic service after the execution of construction or reconstruction, deflection decreases as the stability of pavement increases due to compression, high density and increased strength of each layer by the traffic load , and that after the point it keeps its level quite stably till the outbreak of failure , when the deflection increases again. Therefore we can judge the degree of pavement failure by taking deflection of pavement at the stable period for the base. Also as we can estimate elastic modulus, strain and the present condition of already constructed pavement (T_A) by applying the theory of elasticity and analysis from investigations of the past, this can be used as a measure to select the method of maintenance and repair.

3. Evaluation of Pavement through Visual Research and Paired Comparison Method

As we have seen in the previous chapter, it is necessary to take living condition , safety and dynamics of the street into consideration besides present serviceability index (PSI) used in AASHO road test and the road maintenance and repair manual[5]. We have tried to make an equation of evaluation through the visual research on the pavement surface and that of evaluation through the paired comparison method.

3.1 Evaluation through the Visual Research on the Pavement Surface

This research was done in order to make an equation of evaluation on the condition of the pavement surface for maintenance and repair of prefectural roads in Tokyo.[6] It was carried out in the form of questionnaire corncerning the condition of the

pavement surface and vibration. It was directed to twenty-one pavement engineers in the fifty sample sections, who had to grade the following items with A-C (A: very good, B: fair, C: no good).

Items:

a) roughness
b) rutting
c) crack
d) stripping and pot-hole
e) slipping
f) vibration

According to the evaluation of each item, we have further made a synthetic evaluation by grading the failure of the pavements with A: no need of repair, B: some need of repair, and C: a good need of repair. The results of the research is indicated in Table 2. The item of the biggest range is cracks, and it is followed by rutting, stripping and pot-hole,roughness, vibration and slipping. This order can be translated as that of contribution in the judgement of maintenance and repair. However, this may be due to the fact that the research was done in the day time on a sunny day, when vibration was not so large because of the traffic jam, and when the pavement surface was dry, which made it difficult to judge slipping. It also has to be remembered that all the researchers were from management offices.

In each sample section, cracking ratio,rutting, roughness and vibration were actually investigated to find out the relation between these measurements and the visual evaluation. The data are indicated in Table 3. You can see that the criteria from the visual evaluation is quite low and severe compaired with the target value of the road maintenance and repair manual.[5] As for vibration, we didn't have the clear evaluation data due to the difference in time of the executions of the visual research and of the vibration investigation,which caused us to be unable to get the correlation of the two. Therefore we don't have the criteria of the vibration.

3.2 Evaluation through Paired Comparison Method

Since we know that living condition and safety of the roadside must be considered as well as the performance of vehicles to evaluate serviceability of streets, we have chosen three items---performance (serviceability index), living condition (vibration) and safety (skid resistance)--- and tried to evaluate them through Guttman's paired comparison method. In this method of Guttman, a questionnaire is required to be completed about what can't be measured theoretically. The questionnaire should have been carried out by the administrator of road, users, and inhabitants of the roadsides, but as an attempt, we have taken 131 road engineers of the construction bureau of T.M.G. instead, and had them make personal (non-official) answers to the questionnaire. The items and categories of the questionnaire is shown in Figure 6 and the results of calculated evaluation value are indicated in Table 4. Equation (3)-(5) are the equations of evaluation of the items led by the method, and equation (6) indicates the evaluation index Φ.

$$R=2.60718Rx-0.15559Rx^2 \qquad (3)$$
$$0.2 \leqq Rx \leqq 4.42$$
$$E=8.44380+0.20031Ey-0.00327Ey^2 \qquad (4)$$
$$40 \leqq Ey \leqq 90$$
$$S=-9.78139+59.23510Sz-55.59543Sz^2=19.777397Sz^3 \qquad (5)$$
$$0.2 \leqq S \leqq 0.95$$
$$\Phi=R + E + S \qquad (6)$$

where, R: performance Rx: serviceability index
E: living condition Ey: peak of vibration (dB)
S: safety Sz: skid resistance

The evaluation index Φ caluculated with the actual measurements which were measured twice at a route is indicated in Figure 7. The mean of the evaluation index of the whole lanes was 25.770 for the first measurement and 24.917 for the second measurement which took place 10 months later. This means it made a decrease of 0.853 and moved from the high rank to the middle rank. However, even if the vibration at the border of the road site was changed into its maximum of 90 dB at the peak with same

performance and safety factors as those in Figure 7 (at the second measurement), the evaluation index would still belong to the middle rank as the arrow indicates. We must say this does not fit the claims and demands of the inhabitants of the road-side.Therefore we can conclude that it seems quite necessary to have the questionnaire involving the inhabitants, and also to improve the equation of evaluation in some ways.

4. Structural Evaluation through Dynaflect Deflection and Selection of the Method of Maintenance and Repair

Deflection is widely used as the method to evaluate the degree of failure . There are such things used in the investigation as Benkelman-beam, curvaturemeter, Dynaflect and falling weight deflectometer. We will report the method of evaluation and maintenance and repair through Dynaflect, as it has a high working mobility and it involves easy preparation. We took up the asphalt pavements without cementicious layers.

4.1 Estimate of Elastic Modulus through Swift's Equation and Its Practical Applications

Deflection on the surface of ashalt pavement is determined by its magnitude of the load, its loading time ,the temperature, the pavement structure and the subgrade condition etc. Among these, the magnitude of the load and the loading time are constant in Dynaflect. The temperature can also be ignored since T_A of pavements we focus on are usually of more than thirty centimeters, and the temparature does not effect this type of pavements, as indicated in Figure 8. Thus the elements we take up are the pavement structure and the subgrade condition. We investigated these elements by using Swift's equation and estimated their elastic modulus of pavement (Ep) and that of subgrade (Es). We have acquired Ep and Es by the Moore's method[7] (the method of estimating Ep and Es which minimize calculated errors for deflection,w1-w5, through Dynafleet).

4.1.1 Normal strain on the top of subgrade

After estimating Ep and Es of before and after the pavement repair by Swift's equation, we tried to get the normal strain ε_z

of the top of subgrade by using BISAR (BItumen Structure Analysis in Roads:Shell International Petroleum Company Ltd.). The relation of T_A and ε_z befor/after repair is a negative correlation as indicated in Figure 9. Concerning ε_z of repaired pavements, there is a clear division around 2.8×10^{-3}, where you can see the effect of repair.

4.1.2 The relation of Ep, Es and DMD

Figure 10(a) shows the relation of Ep and DMD, with Es as the parameter. In this figure we have taken the case in which H (the thickness of pavement) equals seventy centimeters. The curve line shifts to the left as the thickness increases. Table 5 shows the condition of cracks and mean, and standard deviation of Ep and Es. Ep's decrease is recognizable as the cracks get worse.

4.1.3 The relation between Ep and T_A

T_A is the total of the thickness of the hot-mixtured asphalt concrete which is used to form surface-course and binder-course and into which each layer of the pavement was converted. The coefficient of the relative strength of the materials of all the layers of the pavement (a) is the base of this calculation, but it does not define the strength in a strict sense. Therefore, in Figure 10(b) you can find some scattering but at the same time recognize a highly significant positive correlation between T_A and Ep. Though Figure 10(b) offers the relation in a good condition of pavement after repair, we can get T_A of the present pavement condition ($T_{A.P}$) by inputting DMD and Es from the Dynaflect investigation at the intended repair positions. For example, in the case that the thickness of pavement is 70 cm, DMD befor repair is 39×10^{-3}mm and Es is 300 kg/cm^2, it can be calculated $T_{A.P}$ equals 39 cm; and when T_A of a good condition or necessary T_A ($T_{A.N}$) was 44 cm, T_A can be judged to have decreased 5cm ($T_{A.N}-T_{A.P}=5$) by failure of the pavement. However, this method is not practical since it needs to make figures according to every thickness of pavement and needs the use of a computer. To make it more practical and simple, we have made the multiple regression analysis concerning all the elements of Figure 10.

4.2 Multiple Regression Analysis through DMD, w5, H and T_A

The elements to estimate $T_{A.P}$ are DMD, Es and H in Figure 10. Since we have to use the computer to get Es, it is better to find another element to replace Es. In Utah, BCI(w4-w5) is used to indicate the strength of the bottom part of the pavement. Thus we paid attention to w5. We investigated Es at every measuring position of each route that were measured after repair, and tried to find the relation of Es and w5 at 120 measuring positions selected at random. This is indicated in Figure 11, and it shows their very strong correlation as you can see. This means we can use w5 instead of Es. Therefore we used DMD, H and w5 as our predictor variables and made multiple regression analysis on the data of respective routes of a good condition after repair. The following is the result:

$$T_A = 18.804 - 0.319\,DMD + 0.356\,w5 + 0.316\,H \qquad (7)$$

(multiple correlation coefficient R=0.974)

where, DMD: deflection of first sensor ($\times 10^{-3}$mm)

w5 : deflection of fifth sensor ($\times 10^{-3}$mm)

H : the thickness of pavement (cm)

The standard deviation between T_A obtained from this equation and design T_A is only 2.242 cm and this is judged to make no practical difference. Indicated in Table 6 is the relation of $T_{A.P}$ of respective routes befor repair, design T_A and the method of repair. From this table, we can see that below 1.3 cm, we use the surface layer reconstruction (the method which takes away the pavement surface and covers it with new asphalt), 0.8 -11.9 cm the partial layer reconstruction method, and over 5.8 cm the full layer reconstruction. Nevertheless, since some in the table used unusual methods under their particular conditions, we think we can simply conclude, taking possible errors drawn from equation (7) into consideration, that below 3 cm we should take the surface reconstruction or the surface treatment, 3-10 cm the partial reconstruction, and over 10 cm the full reconstruction. However, when it is possible to make the surface formation higher by ($T_{A.N} - T_{A.P}$), the overlay method should be used instead. The flow chart to select the method of structural evaluation and repair with these mentioned methods is indicated

in Figure 12. In Table 6, we made calculations with mean value of all measurements, but the value used for calculation are usually determined according to what is considered to be the management level. The claims and demands of the inhabitants of the roadsides and economy must be considered as well.

5. Conclusions

In the maintenance and repair of the pavements of streets, we have to take the living condition and safety elements into consideration as well as its serviceability. Therefore, we are going to develop a complete equation of synthetic evaluation which will satisfy all of the administrators, users and inhabitants.

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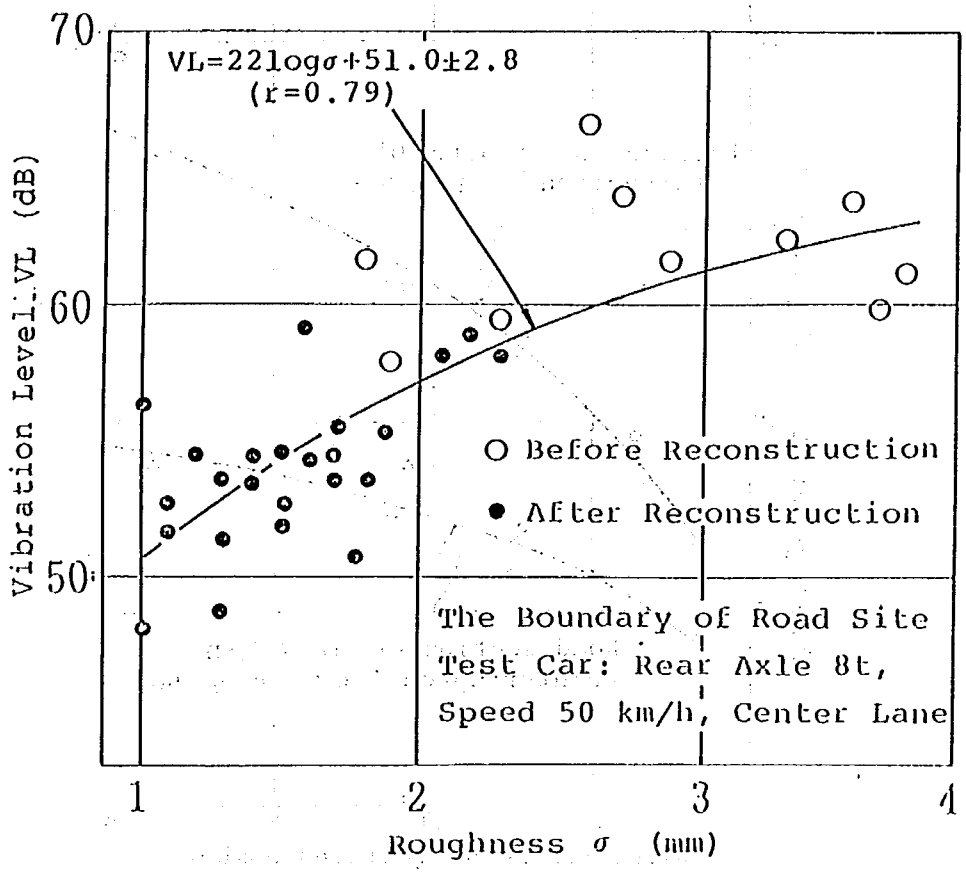

Figure 1. Relation between roughness and vibration

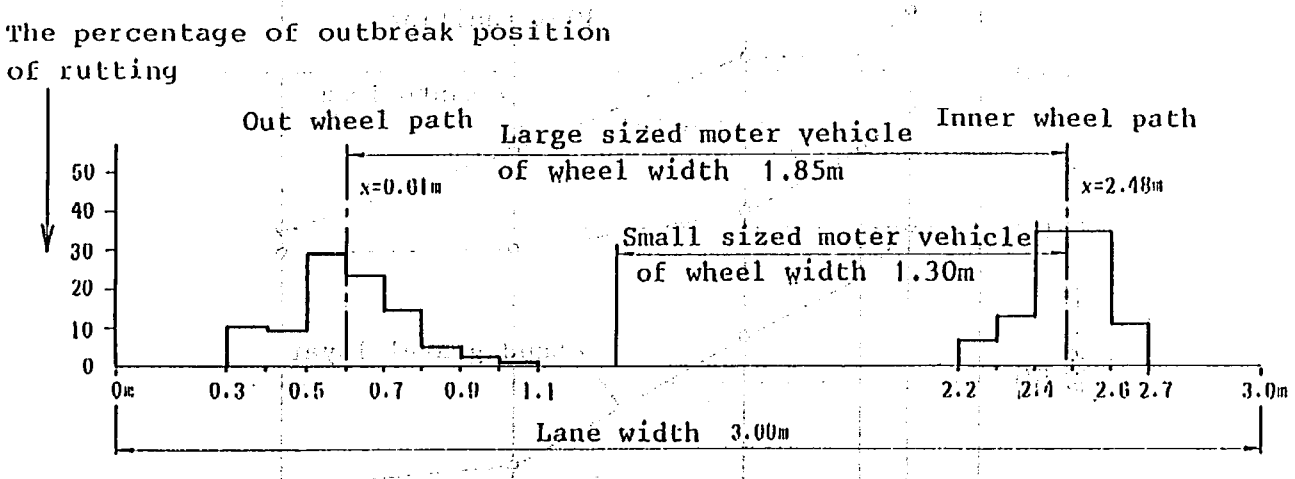

Figure 2. The outbreak position of rutting

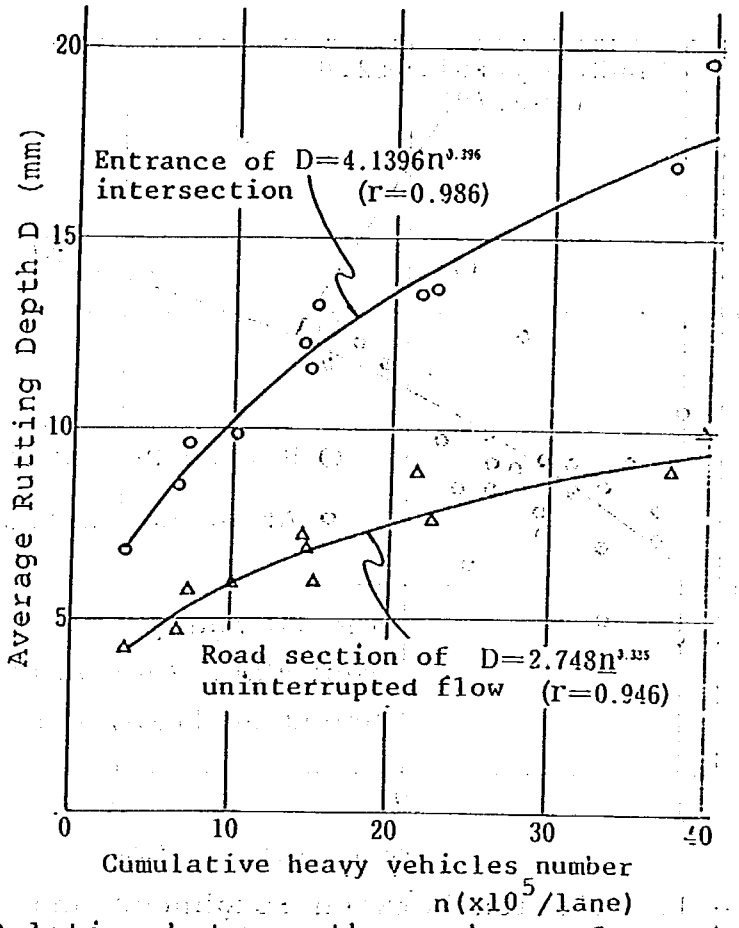

Figure 3. Relation between the numbers of passing heavy commercial vehicles and rutting

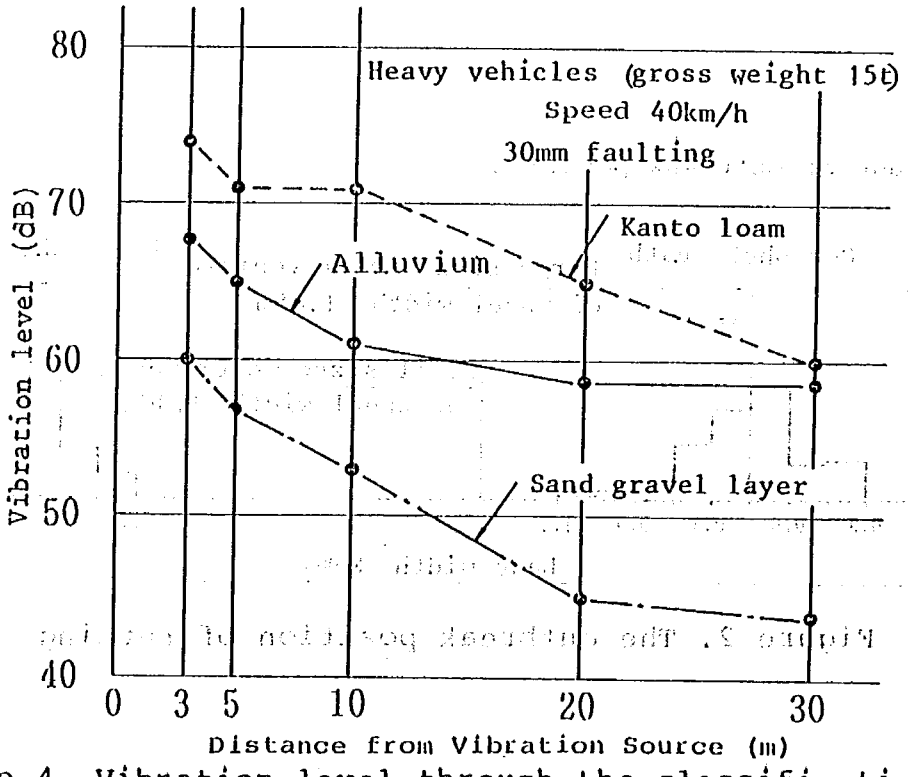

Figure 4. Vibration level through the classification of foundations

Table 1. Items for the predictive equation

Traffic	* Vehicle Classification / Lane * Average Speed / Lane * Interval of Traffic Control Signal
Vibration Measurement	* Distance from Vibration Source * Distance from Roadway Side
Foundation	* Classification of Foundation * Damping Factor
Condition of Road	* Lane Width * Surface Loughness * T_A

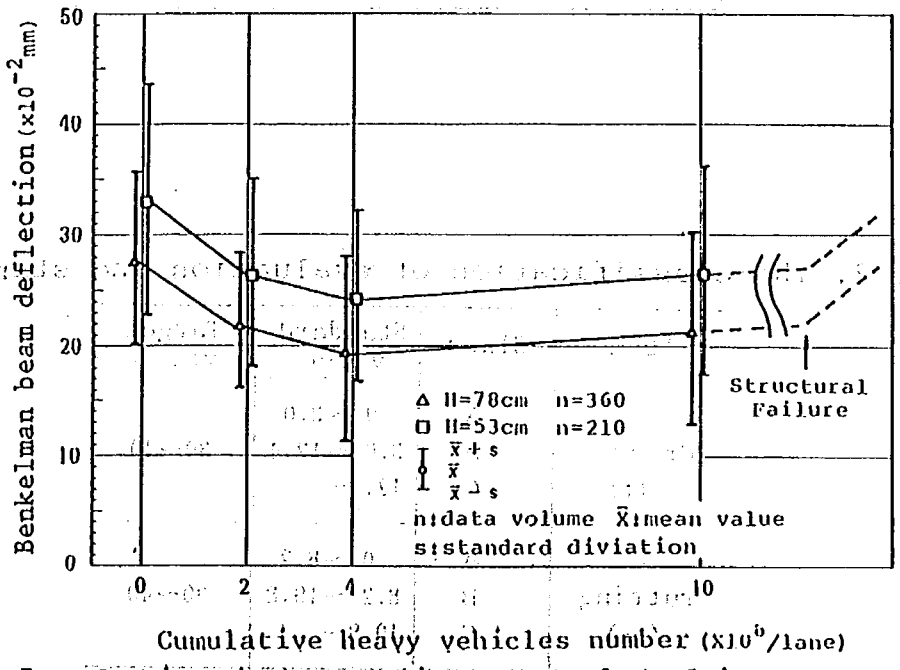

Figure 5.Relation between the accumulated heavy commercial vehicles and Benkelman deflection

Table 2. The results of the research

Item	Rank	Synthesis rating by three grade	
		Point	Range
Crack	A	− 0.290	1.422
	B	− 0.222	
	C	1.132	
Deformation and rutting	A	− 0.166	1.154
	B	− 0.156	
	C	0.988	
roughness	A	− 0.218	0.887
	B	0.246	
	C	0.669	
Vibration	A	− 0.140	0.689
	B	0.255	
	C	0.549	
slipping	A	− 0.240	0.457
	B	0.094	
	C	0.217	
Stripping and pot hole	A	− 0.053	1.108
	B	0.255	
	C	1.050	

Table 3. The classification of evaluation and standard value

Item	Class	Standard value	Target value
Crack (%)	A	0 ~3.0	30~40
	B	3.0 ~12.4	
	C	12.4~	
rutting (mm)	A	0 ~8.2	30~40
	B	8.2 ~19.8	
	C	19.8~	
roughness (mm)	A	0 ~1.7	4.0 ~5.0
	B・C	1.7 ~	

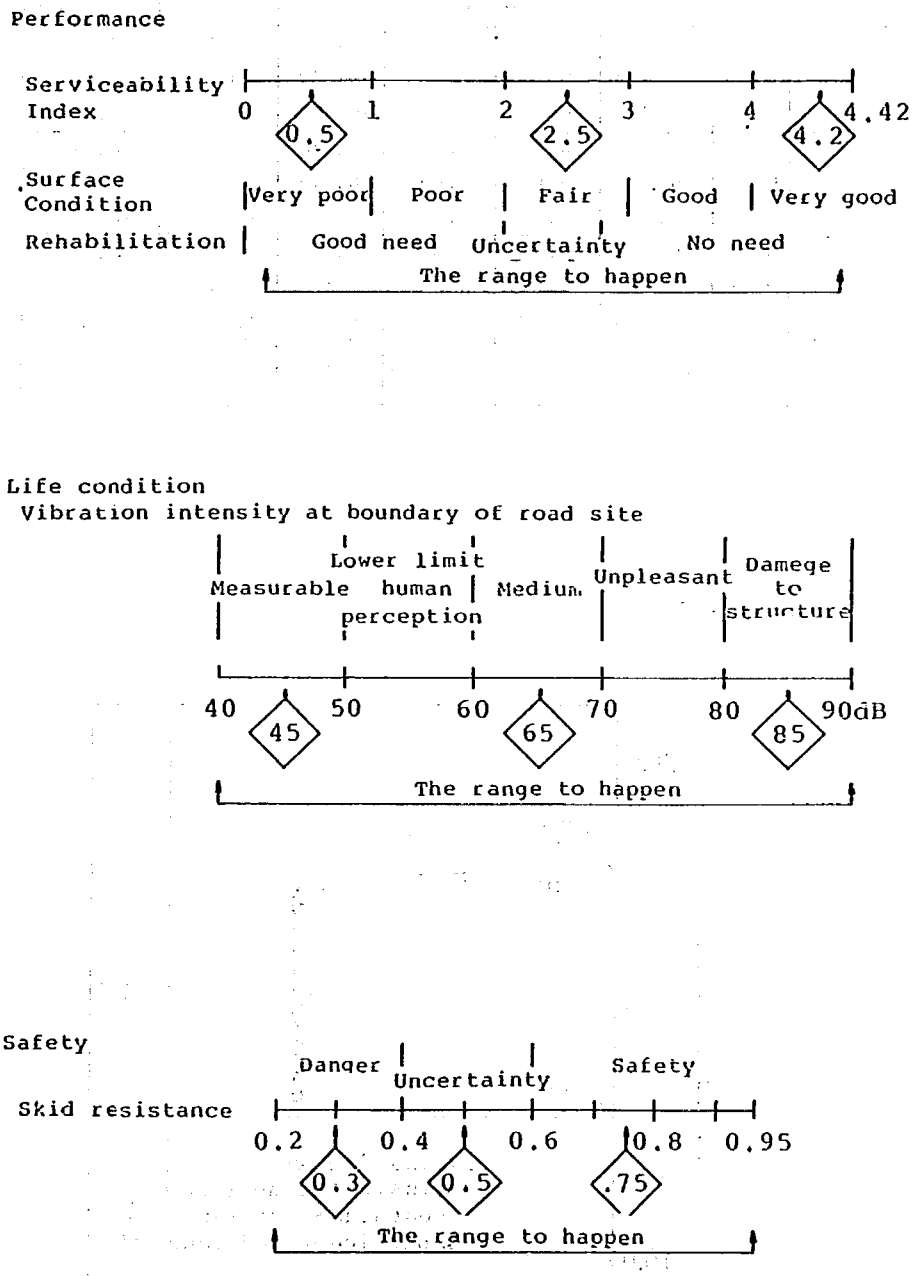

Figure 6. The items and categories of the questionnaire

Table 4. The results of calculated evaluation value

Item		All the members	Home post		Driving license		Experience	
			Head office	Local office	have	haven't	more than 12 years	less than 12 years
Performance	4.2	0.320	0.327	0.313	0.319	0.322	-0.334	0.297
	2.5	0.054	0.054	0.054	0.055	0.051	0.055	0.052
	0.5	-0.374	-0.381	-0.367	-0.375	-0.373	-0.390	-0.349
Living condition	45	0.406	0.402	0.409	0.402	0.412	0.401	0.413
	65	0.087	0.093	0.081	0.086	0.090	0.085	0.091
	85	-0.493	-0.494	-0.491	-0.487	-0.502	-0.485	-0.504
Safety	0.75	0.383	0.377	0.389	0.387	0.376	0.380	0.387
	0.50	0.053	0.052	0.054	0.054	0.049	0.047	0.061
	0.30	-0.436	-0.430	-0.443	-0.442	-0.425	-0.428	-0.448
Correlation ratio		0.60441	0.60474	0.60424	0.60449	0.60445	0.60199	0.60900

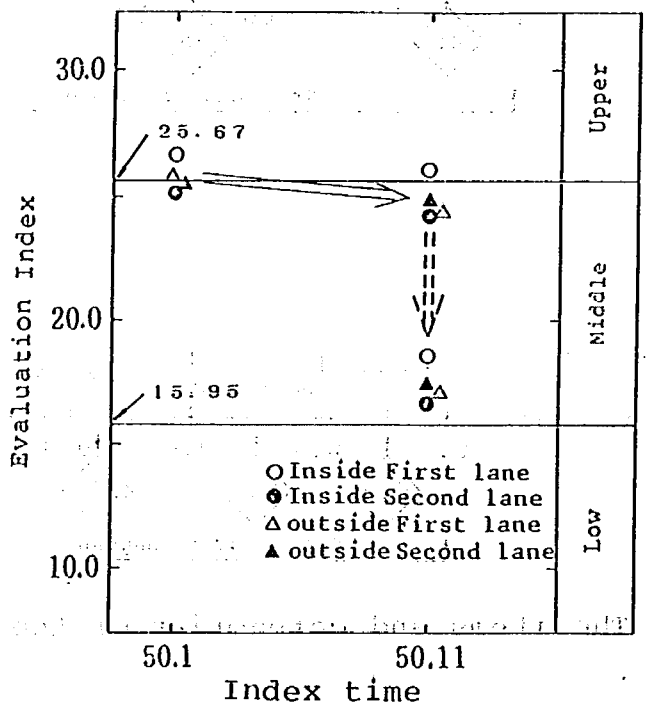

Figure 7. Progressive developement of the evaluation index

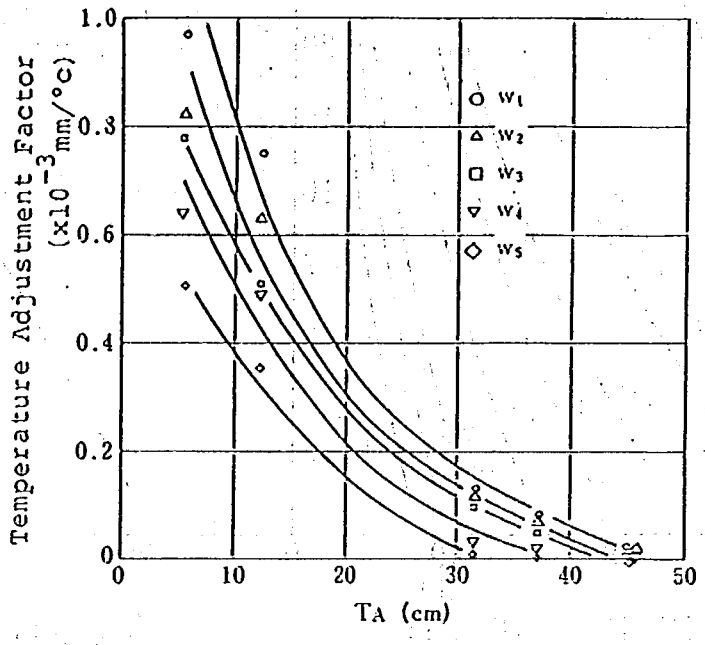

Figure 8. Relation between T_A and temperature adjustment factor

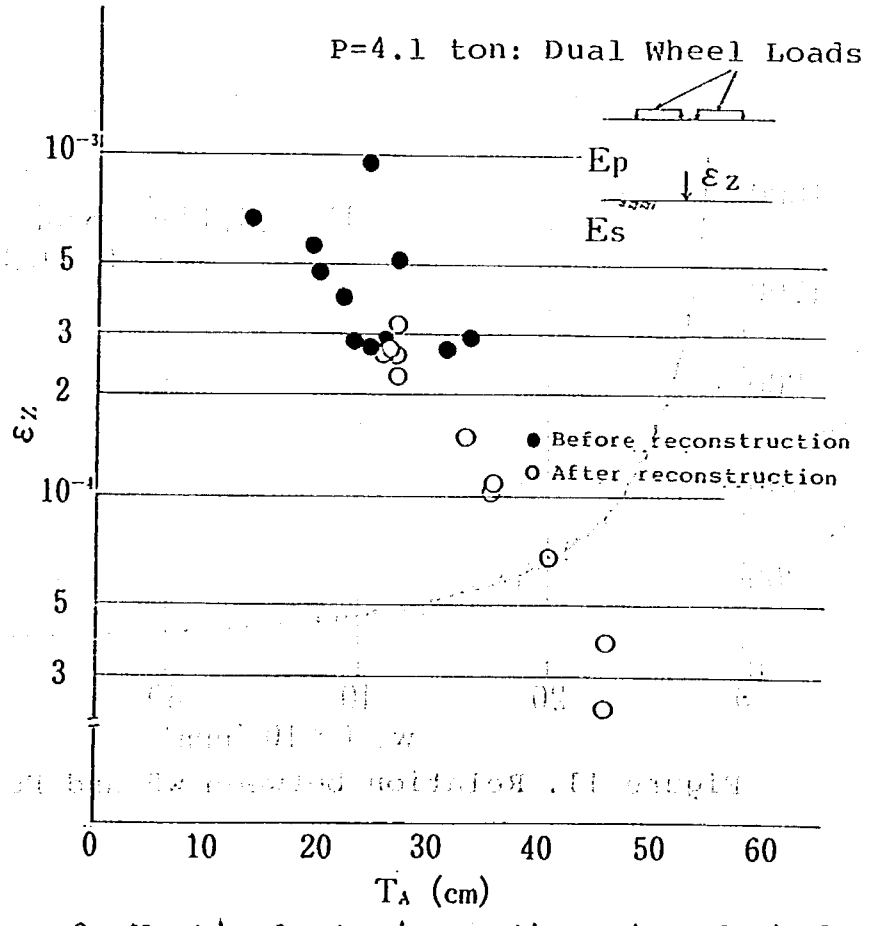

Figure 9. Vertical strain on the subgrade before and after reconstruction

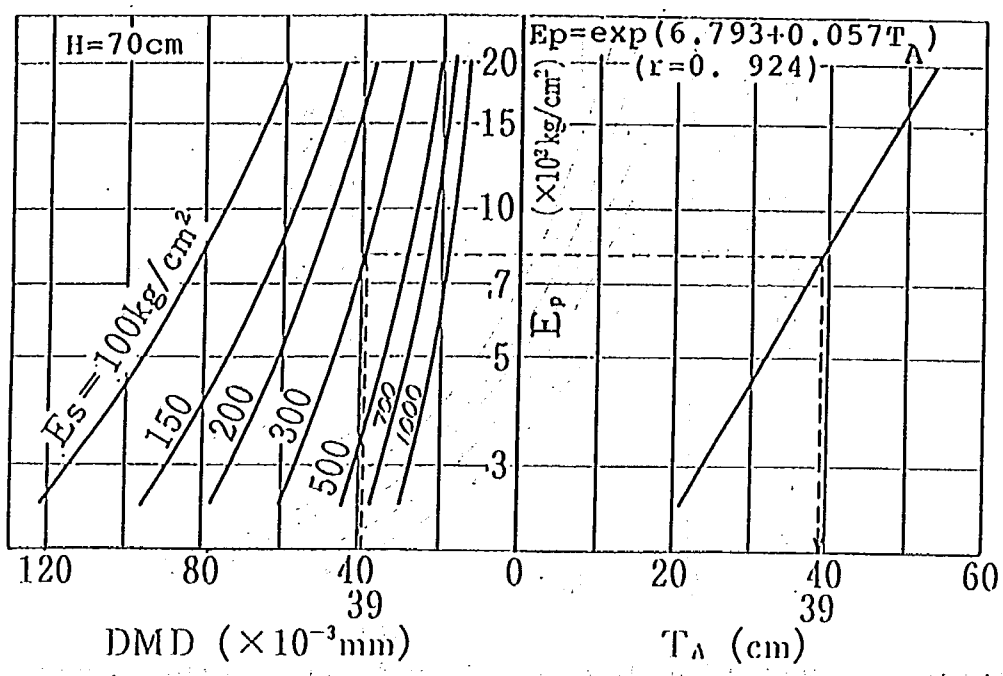

Figure 10. Relation of DMD, Es, Ep and T_A

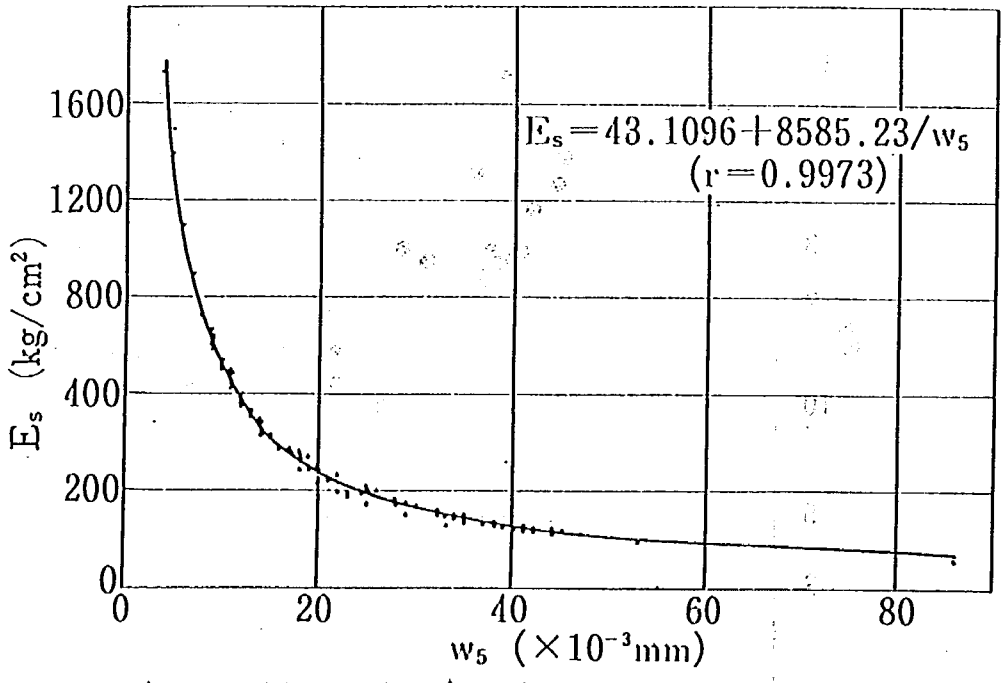

Figure 11. Relation between w5 and Es

Table 5. Ep, Es and the type of crack

Item / Type of crack	Ep.av kg/cm	s kg/cm	Es.av kg/cm	s kg/cm
Alligator	1500	320	700	170
Linear	2700	460	850	190
No crack	3900	760	900	220

Table 6. $(T_{A.N}-T_{A.P})$ and rehabilitation methods

Section No	TA.p (cm)	TA.N (cm)	TA.N - TA.p (cm)	Rehabilitation Method
1	25.7	27.0	1.3	A
2	11.0	27.0	16.0	C
3	18.2	27.0	8.8	B
4	24.9	35.2	10.3	C
5	7.9	25.5	17.6	C
6	27.5	35.3	7.8	C
7	25.5	26.3	0.8	B
8	17.6	29.5	11.9	B
9	29.7	40.5	10.8	B
10	34.3	35.5	1.2	A
11	34.0	34.0	0.0	A
12	27.9	33.7	5.8	C
13	19.6	35.2	15.6	C
14	33.6	35.2	1.6	C

A Surface Layer Reconstruction

B Partial Reconstruction

C Full Reconstruction

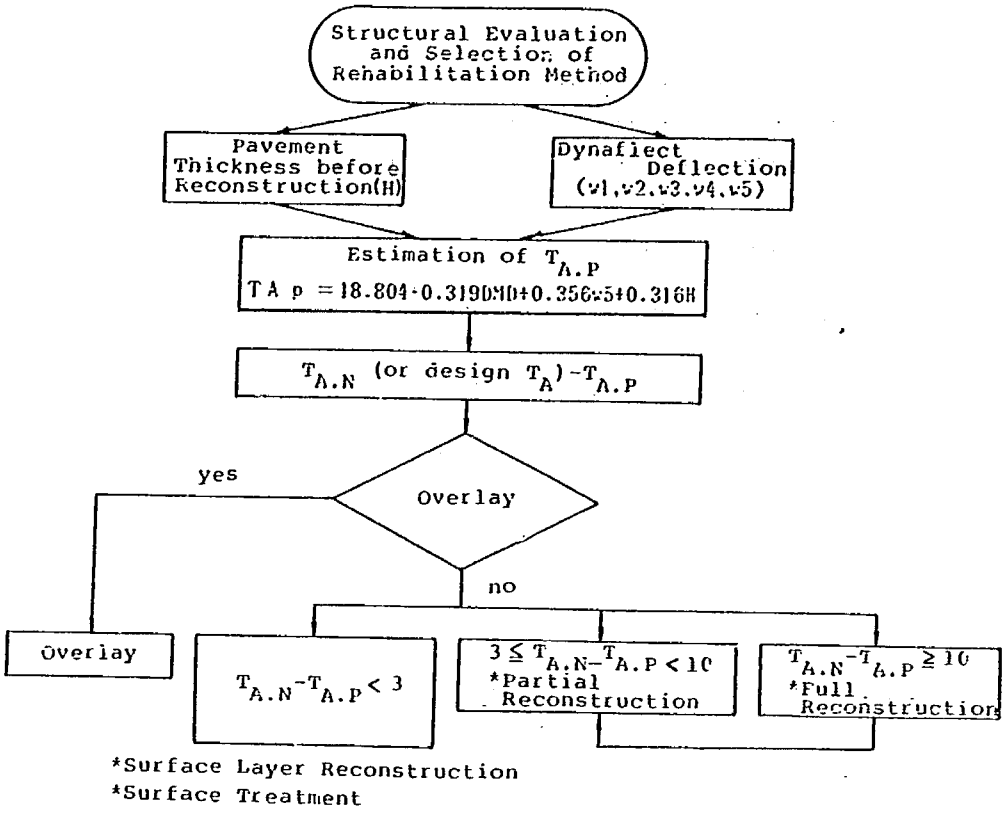

Figure 12. Flow-chart for the structural evaluation and the selection of rehabilitation methods

Improved Data in Ontario's Pavement Management Processes

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Ontario Ministry of Transportation and
Communications

IMPROVED DATA IN ONTARIO'S PAVEMENT MANAGEMENT PROCESSES

by

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J.J. Hajek, Research Officer
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ABSTRACT

Improvements in pavement management visualized for Ontario lie in two principal areas. The first is in the utilization of a pavement condition rating measure, Pavement Condition Index (PCI), which is more objective than the currently utilized Pavement Condition Rating (PCR). PCI uses measured pavement roughness and weighted distress values, and is compatible with past PCR records because of the high degree of correlation of CI with PCR.

The second is in the development of an integrated and co-ordinated five-year maintenance and rehabilitation action plan. Since there is limited data available for developing an adequate spectrum of prediction equations required for the integrated plan, a process to capture the experience of pavement design, evaluation, and maintenance engineers, and to translate this experience into performance prediction of alternative strategies, is proposed instead.

Pavement management data base requirements, based on highway sections with uniform pavement performance, as well as the development of the integrated action plans, and the organizational interplay in the overall pavement management process relevant to these improvements are discussed. Need for minor change to accommodate the improvements is outlined.

IMPROVED DATA IN ONTARIO'S PAVEMENT MANAGEMENT PROCESSES

W.A. Phang, J.J. Hajek, G.A. Wrong

1/ INTRODUCTION

Road transportation in Canada is a responsibility of each provincial government. In the year ending March 1983, the Province of Ontario spent about $540 million on the provincial King's Highways (approx. 25 300 km), and about $484 million on subsidies for municipal roads (approx. 131 300 km). Within the Ministry of Transportation and Communications (MTC), the Transportation Capital Branch is responsible for program planning, development, priority setting, expenditure control, and monitoring and evaluation for the provincial highway program. One office of this branch, the Highway Program Planning Office, develops specific five-year highway program targets within given and estimated funding constraints and strategic planning guidelines, while another office of the branch, the Highway Program Administration Office, develops the operational plan for the first few years of the five-year program. Both of these offices work closely with the five regional offices, who carry out the program, in developing program targets and operational plans.

In the past few years, the importance of maintenance and rehabilitation activities has been emphasized as a result of spending restraints. As a result, determined efforts have been made to improve rehabilitation planning through the introduction of PARS (Program Analysis of Rehabilitation Strategies) [1].

More recently, a task force has been working on the integration of pavement management system concepts into current management processes for both maintenance and rehabilitation.

This report outlines some of the concepts and processes which are currently being considered for a more effective MTC pavement management system (PMS).

2/ WHERE DOES PMS FIT?

Where do pavement management systems (network and project) fit into the current MTC management processes? Some insight into this question was gained from an 1983 article by Finn [2].

Figure 1 (adapted from Finn's article) shows that 'Network Management' in MTC is within the functional areas for 'Planning' and for 'Programming', and that 'Network Optimization' is addressed by the Ministry's computer program, PARS (Program Analysis of Rehabilitation Strategies) [3]. 'Project Management' is within the Pavement Design and Regional Geotechnical areas and is addressed for new pavement construction, by the MTC computer program called OPAC (Ontario Pavement Analysis of Costs) [4]. For maintenance and rehabilitation there is currently no Ministry-wide optimization process so this aspect is addressed later in this report. Central to all of these PMS processes is a pavement management data base. At present, however, the MTC Highway Inventory is not capable of providing all of the necessary pavement management data (this is discussed in the following section).

Figure 2 shows the inputs required for network optimization in developing five-year planning target actions, costs, and system performance, while Figure 3, based on Reference 2, illustrates how project optimization translates these targets into specific job designs and specifications which are later worked-up into contract documents.

The 'players' in MTC who already carry out some of the functions described above have been identified, and the additional functions and processes needed for an integrated PMS were noted in a previous pilot study report [5]. In the present report, the changes which are necessary for implementation of the improved PMS, are outlined.

3/ DATA BASE

The Highway Inventory in MTC is maintained by the Highway Program Planning Office, and it is based on data obtained from the Ministry's five Regions.

The major data items collected for the Highway Inventory are principally related to geometric characteristics such as pavement widths and to traffic operations, for example, Annual Average Daily Traffic (AADT). The inventory sections therefore begin and end at features which relate to highway geometry and traffic, i.e., at changes in widths, at intersections, etc.

For pavement management purposes, the data base sections should conceptually begin or end wherever changes in pavement performance can be expected. It is logical to expect these changes to take place at beginning and end points of construction contracts, and at significant subgrade soil changes, i.e., pavement structure and subgrade support parameters should define PMS data base sections.

The criteria for sectioning the highway system for different purposes may or may not be compatible. For example, criteria for evaluating level of service of traffic operations may be quite different to those for evaluating serviceability of highway pavements. However, the Linear Highway Reference System (LHRS) based on a system of numbered nodes and an offset to define a point on a highway [6], can be used to tie different data bases together and to avoid duplication of data.

For network management purposes, a composite data base will need to be created using pavement performance data for PMS data base sections annexed to whatever highway inventory data are relevant. Such a composite data base will, of course, contain historical profiles of the pavement structure and performance as well as current conditions including truck traffic for pavement design and evaluation purposes.

The Highway Inventory and the Pavement Performance data bases are currently on an IBM 370 mainframe computer. For improved convenience and reduced operating costs, it is proposed that the new composite data base be formulated for an appropriate 'personal' computer which will also be available in each of the five Regions. At this time, an IBM-PC running the FOCUS program is being used for pilot studies. The data base thus produced is uploaded onto the mainframe and can be accessed by Regions through remote terminals. Ultimately, each Region would have it's own microcomputer.

4/ PAVEMENT CONDITION RATING (PCR) AND PAVEMENT CONDITION INDEX (PCI)

In the past, MTC has evaluated pavement performance through the pavement condition rating (PCR) taken at least every third year (annually in one Region). The PCR is a purely subjective rating assigned to a road section by one or more experienced pavement engineering staff based on a set of word and picture guidelines [7]. PCR values trigger certain actions, such as placing a section in the five-year pavement rehabilitation plan.

Because subjective PCR's create inconsistencies in planning, and since a single PCR does not give clues as to future performance and may be as much as three years out of date, there is need to replace PCR as the planning parameter.

The measure proposed for replacing PCR is the Pavement Condition Index (PCI). Over the past few years, an investigation of how PCR is assigned has shown that PCR is essentially comprised of two elements -- riding quality, and pavement distresses which significantly affect ride quality [8]. In this investigation, PCR was correlated against a calculated PCI in such a way as to minimize scatter around a 45° line, by assigning appropriate weights to both riding quality (RCR) and to the various distresses which comprise the sum of distress manifestations (DM). These findings were summarized as follows:

$$PCR = PCI = 100\,(0.1 \cdot RCR)^{1/2} \cdot \frac{320 - DM}{320} \qquad (1)$$

where: RCR, the Ride Comfort Rating, is obtained either as a subjective rating or, preferably, is derived from a correlation between RCR and roughness measured with mechanical devices such as PURD (an accelerometer mounted on a trailer axle) or the Mays meter (excursions of a car rear suspension system), and DM is calculated as follows from the check-off sheet (see Figure 4):

$$DM = \sum_{i=1}^{27} C_i (S_i + D_i) \qquad (2)$$

C_i = weighting value for a particular type of crack or other form of pavement distress (see Table 1).

S_i = weighting value for severity of crack or other form of distress (see Table 2).

D_i = weighting value for density of occurrence of the particular crack type or other form of distress (see Table 2).

A certain amount of historical data on PCRs is available over about 10 years, but detailed data used in the development of PCI is more limited and is only available for the last five years.

5/ PERFORMANCE PREDICTION

A requirement for evaluating the cost-effectiveness of rehabilitation alternatives, and for evaluating consequences of different spending levels, is a knowledge of the future performance of pavements and their maintenance and rehabilitation treatments. Available PCR data were analysed by Cook, Bacchus, and Kher using multiple regression techniques [3] and the results were summarized in the following equation:

$$P = P_o - K_1 A^{K2} \cdot M^{K3} \cdot T^{K4} \qquad (3)$$

where: P = PCR

P_o = initial PCR

A = age in years since last pavement overlay

T = AADT (Annual Average Daily Traffic Volume)

K = constants which change with the Region

These prediction equations are currently being used in the development of five-year targets by the Highway Program Planning Office. However, it is clear that because of their limited scope, a great deal of future development is needed. Also, the regression route to the development of an expanded scope of these prediction equations may not be feasible because of a significant lack of data. A way in which the prediction data may be obtained in future is discussed in the following section.

Another facet of the prediction of pavement performance is the immediate effect of the proposed treatment on the PCR or PCI. For example, sealing cracks as a preventive treatment is unlikely to immediately change the present condition rating, but it ought to retard the future rate of change in PCI. Extensive hot-mix patching done as a corrective measure could immediately improve the PCR of the section, but not change the overall rate of deterioration of the section in future years. Thus in predicting the effects of applying a treatment, the immediate PCR change or jump, as well as the future rate of change in PCR need to be given agreed values.

6/ CO-ORDINATED MAINTENANCE AND REHABILITATION

Funds for corrective maintenance done by the maintenance patrols are included in funds which are under Maintenance Management control. On the other hand, non-patrol maintenance of contract size is often carried out using funds administered by the Highway Program Administration Office. As a result, there may be a lack of co-ordination between these expenditure streams and it is possible that a road which has just had extensive maintenance could be resurfaced within a year or two. This does not happen often but it is clearly indicative of the need to assess intended actions from both areas at the same time.

One of the objectives of improved pavement management is the integration of maintenance and rehabilitation to avoid the possibility of such injudicious spending. The integration of maintenance and rehabilitation in planning vastly increases the scope of the programming needed to structure and analyse alternative strategies. However, this can be offset by restricting the alternative treatments considered to those which are practical and/or preferred for the local conditions. As well, since only limited data exist for developing prediction models for all of these alternative maintenance treatments (also the case for rehabilitation treatments), the only viable solution to these problems at this stage is to rely on predictions based on the best available "engineering judgement" and knowledge of local conditions. At some later stage when a significant population of "judgements" and historical pavement performance data has been collected for analysis,

then more conventional prediction models may be built and included in the PMS.

7/ ENGINEERING JUDGEMENT IN PREDICTION

During the development of a manual for choosing cost-effective corrective maintenance treatments [9], one of the major items quantified was the "expected effective life" of various maintenance treatments. These predictions of expected effective life, illustrated in Figure 5, really represent historical data since they were acquired by interview of over 50 persons, each of whom had at least 20 years of pavement maintenance-related experience [1]. The results of this work can be used as one set of information on maintenance which can form a basis for the engineering judgements needed to develop predictions of performance for a co-ordinated maintenance and rehabilitation strategy.

One systematic way in which engineering judgement can be harnessed to provide predictions is the estimation of PCI_3, i.e., the prediction of the change in PCI visualized at the end of a three-year period. It was the opinion of some pavement design engineers who had long experience in evaluating pavement performance that it was difficult to give a 'judgement' on change in PCR in one or even two years, but, given historical PCR data and knowledge of local conditions, it should be possible to estimate change in PCR over three years.

In the case where rehabilitation is envisioned within the five-year period, an additional way in which this judgement may be harnessed is to obtain answers to questions such as:

- How long would the treatment delay the rehabilitation?
- What terminal PCI condition do you believe is desirable?

Another area in which engineering judgement is needed in performance prediction is estimating the immediate effect on the PCR or PCI of carrying out a treatment. Some before-and-after treatment data are available for overlays and these were used to develop the diagram in Figure 6 [1]. This diagram quantifies the PCR jump or immediate change in PCR based on the

prior, or PCR_B, condition. However, a judgement needs to be made on the expected quality of workmanship, so that the effectiveness factor, e, can be quantified. Analysis of a small amount of data shows that the majority of contracts have an effectiveness factor of about 0.8. This same approach can be used to quantify PCR or PCI 'jumps' for other maintenance and rehabilitation treatments. However, prior to acquiring useful data for developing this approach, the PCI jump for a given treatment, or an estimate of the effects of a treatment, will need to be assigned by an experienced pavement evaluation engineer. For example, a question which will need to be answered in specific terms may be:

"How close to a PCI of 96 (the average condition of newly completed rehabilitation treatments) do you expect the pavement condition to be after application of the recommended rehabilitation treatment?"

8/ CO-ORDINATED STRATEGIES

Information from the previously outlined PMS computerized composite data base will be available to be printed out in all of the regional offices for any section within the given Region. There are somewhat less than 2000 sections which encompass the paved major highway system in Ontario.

As currently conceptualized, the upper part of the printout documents inventory and design data. The central part of the printout has two diagrams -- a plot of PCI vs. Years, and a plot of Normal Pavement Maintenance Costs in $/km, vs. Years. The maintenance cost data are not readily obtainable from the maintenance management costing system. However, the figures can be obtained by special request, once per year, for the specific pavement maintenance items which affect pavement performance for the particular section of road specified. Responsibility for acquiring these maintenance costs should rest with the custodian of the PMS data base. Figure 7 illustrates a possible format of this printout.

The lower part of the printout contains a number of blank spaces which are to be filled out by the Regional Geotechnical (Pavement Design and Evaluation) engineering staff in consultation with District maintenance personnel. The spaces are organized to describe alternative strategies:

1/ the 'do-nothing' option in which no treatment is proposed, but the consequences in terms of PCI_3 and costs are estimated,

2/ the desired strategy with appropriate timing, costs, and consequences, for each treatment envisioned over a five-year planning period, and

3/ the 'fall-back' strategy which shows, for example, what is envisioned if option 2 is delayed for say two years.

There is additional space if other strategies are being considered.

The completed action plan fact sheet for each section is sent on to the Highway Program Planning and Programming Offices where these and other computer devised strategies will be analysed, aggregated, and optimized, consequences of different spending levels evaluated, and five-year target actions, costs and performance formulated. These targets will be discussed with Head Office Operations and Maintenance and needed adjustments made (see Figure 8). The targets are then sent back to the Regional Geotechnical and Maintenance personnel for finalization of the five-year action plan, which will then be funded through the Highway Program Administration Office.

9/ TRAINING PROGRAMS

The success of this 'engineering judgement' approach to performance prediction requires some consistency in application of knowledge which already exists in this area. There is therefore a need to synthesize the available information in some form of guidelines, with examples of use.

A method for capturing all of the 'engineering judgement' for future analysis is also needed, and must be developed.

10/ TIME FOR DEVELOPING AND UPDATING ACTION PLANS

It is expected that 20-30% of all of the sections in the highway system will need to have co-ordinated action plans available for analysis in any given year. That is, about 400-500 action plan fact sheets (see Figure 7), or about 80-100 sheets per region will need to be prepared each year.

Normally, the activities in Year 1 will have been funded and programmed, perhaps in categories of preventive maintenance, integrated normal and extraordinary corrective maintenance, and rehabilitation. (Reconstruction activities would not be included directly in the pavement management processes because these are triggered by deficiencies other than pavement performance, such as safety and capacity.) Activities planned for Year 2, will need to be re-evaluated in Year 1, and adjustments made to the action plans. Activities planned for Year 3 and beyond should be re-evaluated in Year 2, and the five-year action plan should be updated in Year 3.

11/ CONCLUDING REMARKS

Improvements in the current pavement management processes in Ontario's MTC, appear to lie in two major areas:
1/ improved pavement condition ratings for use in financial planning,
2/ improved co-ordination of maintenance and rehabilitation activities.

The institutional changes needed amount to the strengthening of the Head Office Pavement Evaluation function to accommodate the carrying out of periodic pavement roughness measurements of the highway system, and to act as custodian of the pavement management data base. As well, there is a need to recognize the necessary additional Regional Geotechnical input and the development of appropriate training courses.

Certain adjustments to the management processes will be also necessary to enable the co-ordinated maintenance and rehabilitation activities to operate smoothly. Finally, the Highway Program Planning, and the Highway Program Administration Offices need to actively participate in the development of these management process adjustments.

The Task Force report containing estimated costs for implementation of these recommended changes is expected shortly.

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TABLE 1/ Distress Weighting Factors and Description Codes

Pavement Distress Manifestations				
	Description		Code	Weight (C)
Surface Defects	1. Coarse Aggregate Loss		CA	3.0
	2. Ravelling		RA	3.0
	3. Flushing		FL	0.5
Surface Deformation	4. Rippling		RI	0.5
	5. Shoving		SH	0.5
	6. Wheel-Track Rutting		WR	3.0
	7. Distortion		DT	3.0
Cracking	8. Longitudinal Wheel Track	- single	S/LWT	1.0
	9.	- multiple	M/LWT	1.5
	10.	- alligator	A/LWT	3.0
	11. Mid-Lane	- single	S/ML	0.5
	12.	- multiple	M/ML	1.0
	13. Centre-Lane	- single	S/CL	0.5
	15.	- alligator	A/CL	2.0
	16. Meander	- single	S/ME	0.5
	17.	- multiple	M/ME	1.0
	18. Pavement Edge	- single	S/PE	0.5
	19.	- multiple	M/PE	1.0
	20.	- alligator	A/PE	1.5
	21. Transverse	- partial	P/TRV	0.5
	22.	- half	H/TRV	0.5
	23.	- full	F/TRV	0.5
	24.	- multiple	M/TRV	2.0
	25.	- alligator	A/TRV	3.0
	26. Random		RD	0.5
	27. Slippage		SL	0.5

TABLE 2/ Severity and Density Weights and Description Codes

Weight	Severity				Density		
D or S	Description	Code	Rut Depth (cm)	Crack* Width (cm)	Description	Code	%Length or Area
0	Very slight	0	<0.5	<0.7	Few	5	<10
1	Slight	1	0.5-1	0.7-1	Intermittent	6	10-20
2	Moderate	2	1-2	1-2	Frequent	7	20-50
3	Severe	3	2-5	2-3	Extensive	8	50-80
4	Very severe	4	>5	>3	Throughout	9	>80

*For cement-treated base, use 0.3 instead of 0.7.

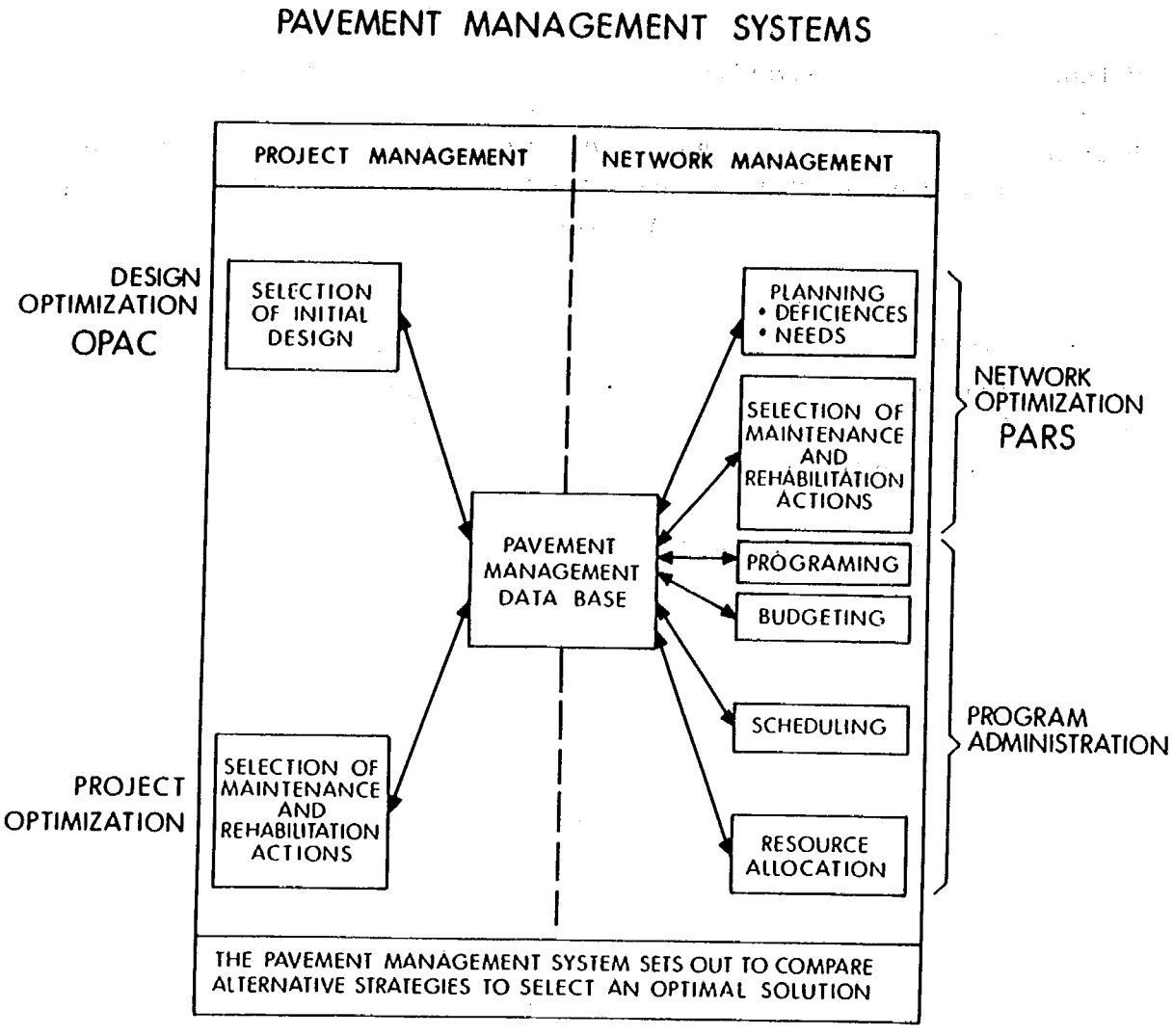

Figure 1/ Illustrating Network and Project Management Functions in MTC

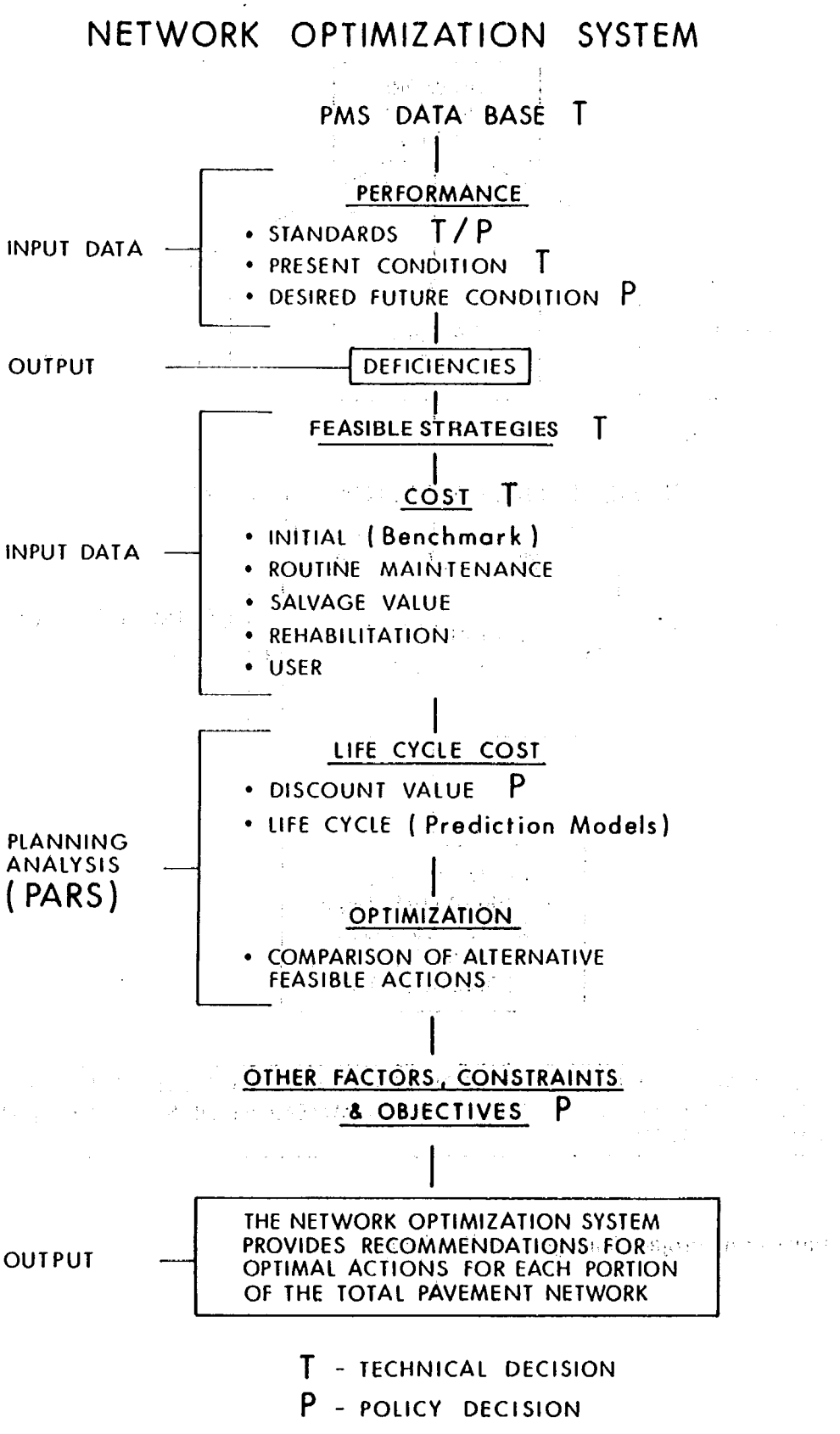

Figure 2/ Illustrating Network Management Products

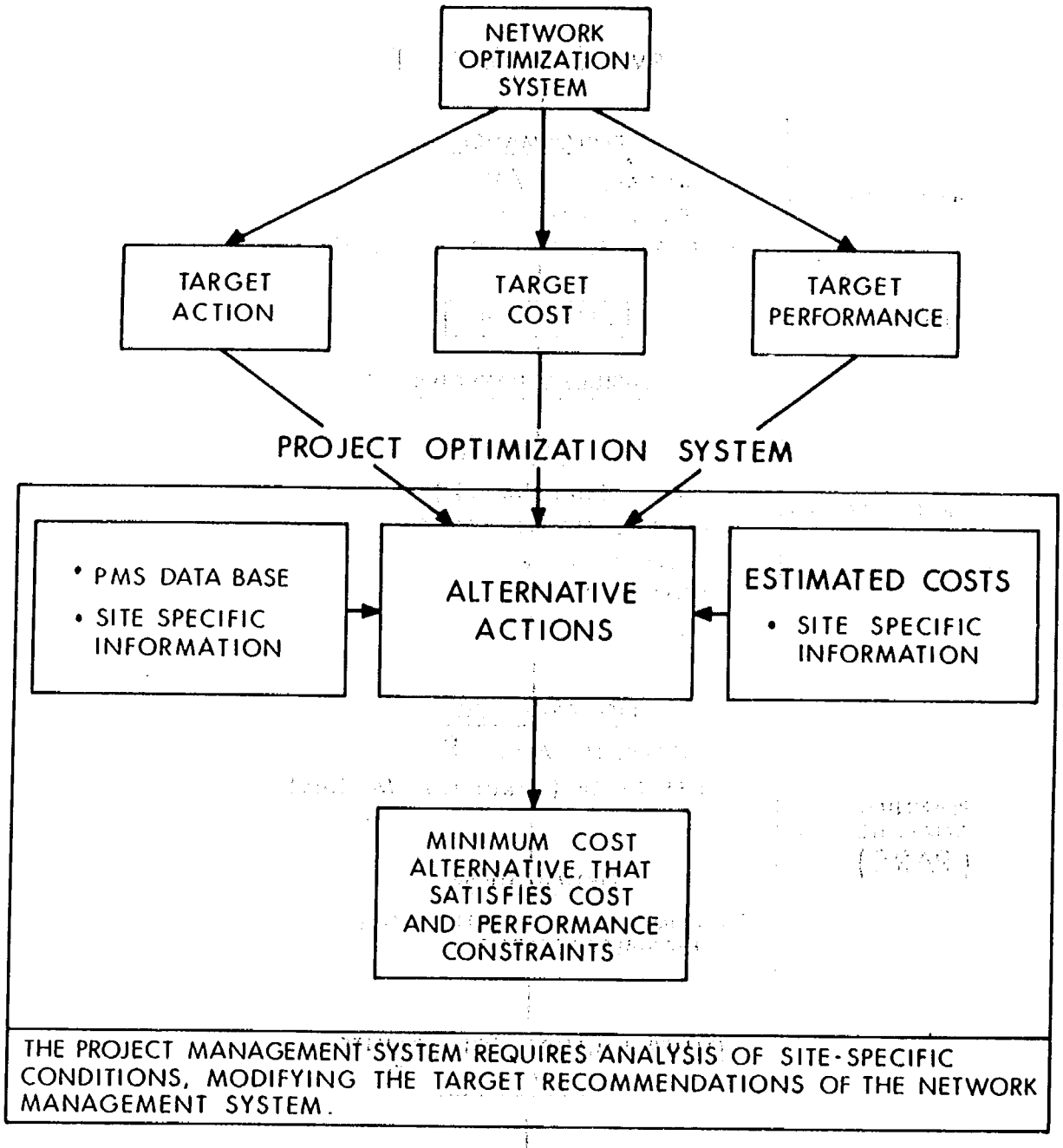

Figure 3/ Illustrating Project Management Product

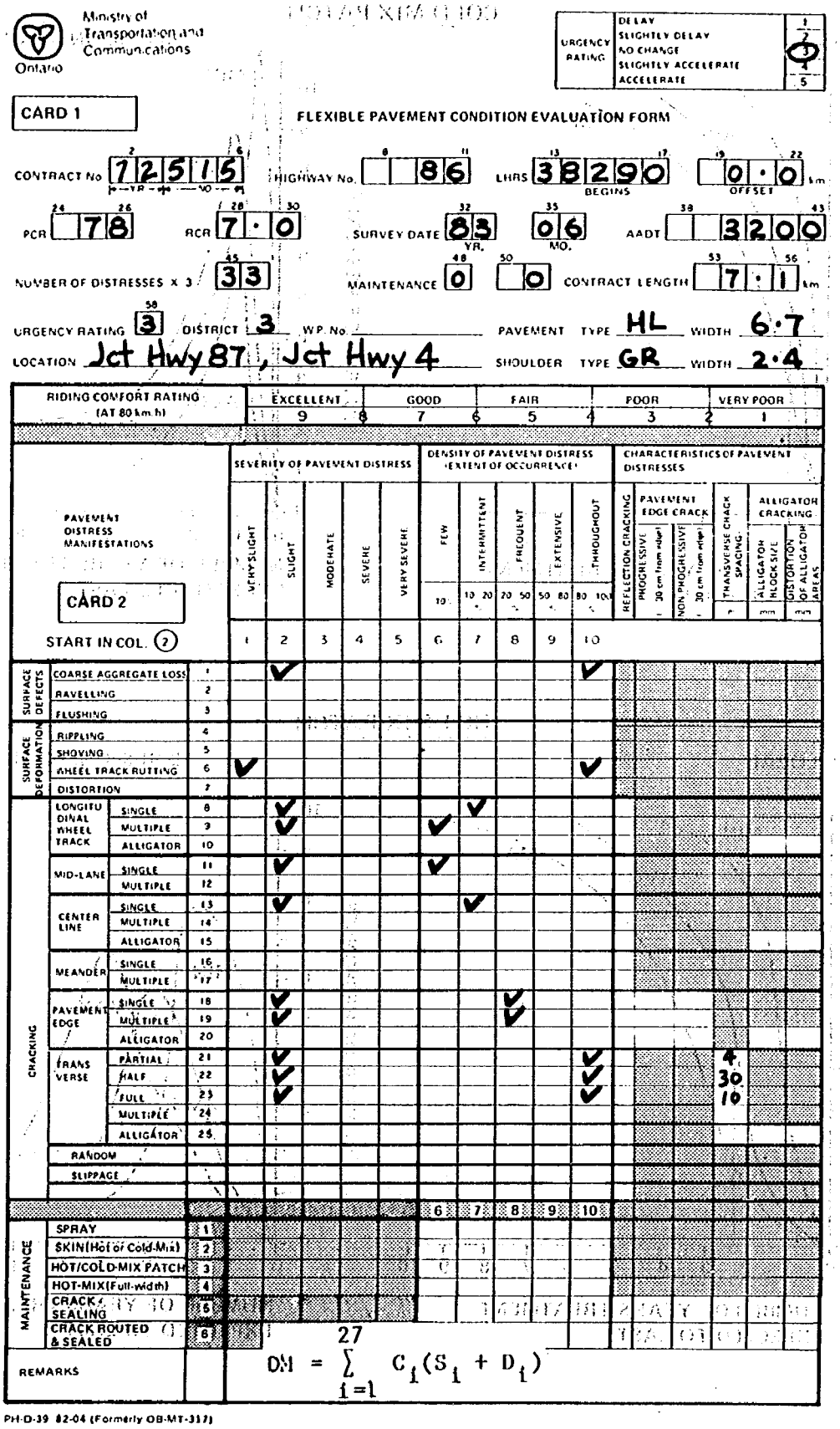

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Figure 4/ Flexible Pavement Condition Evaluation Form

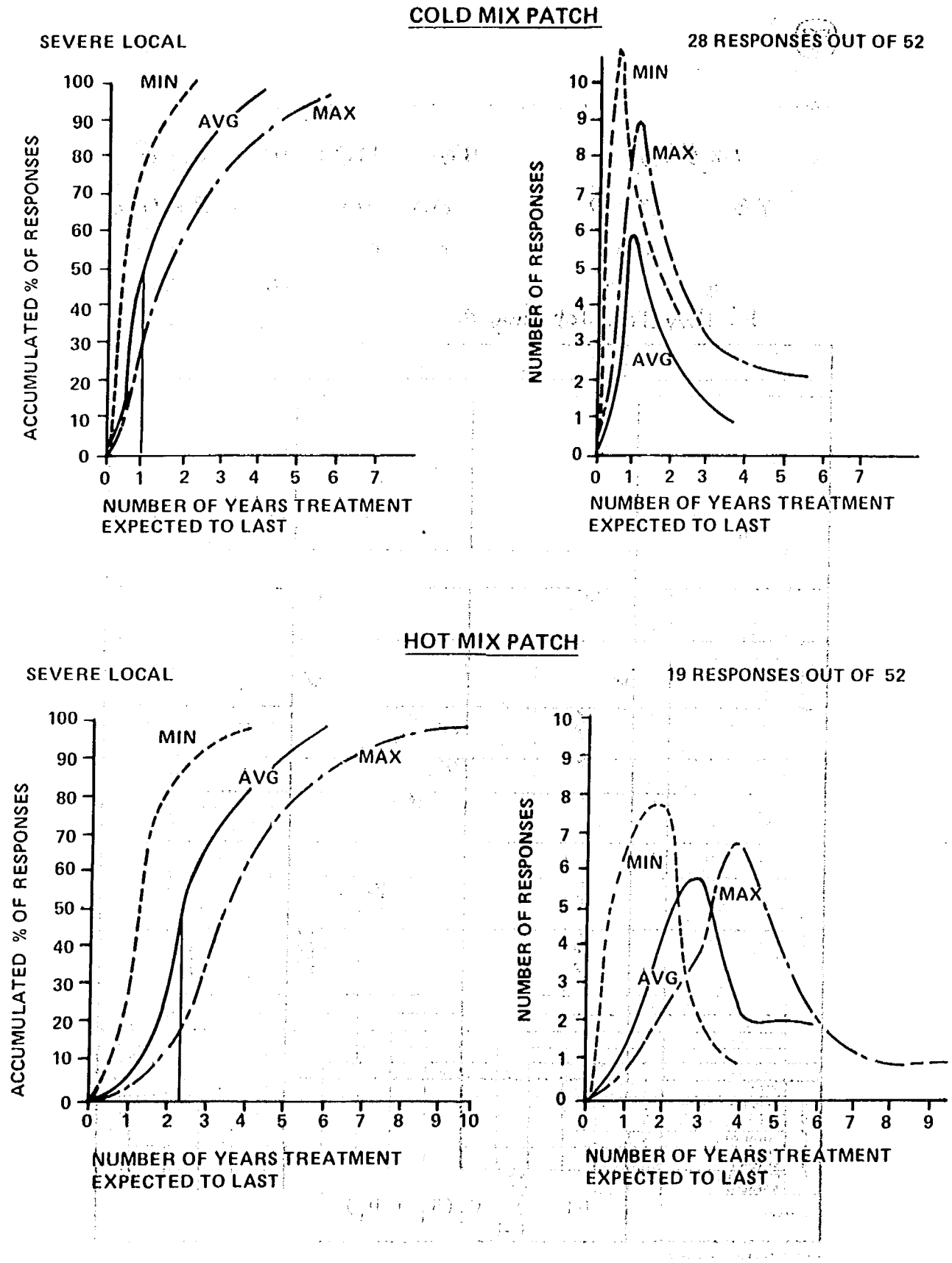


Figure 5/ "Expected Effective Life" of Maintenance for Distortion of Severe/Local Condition

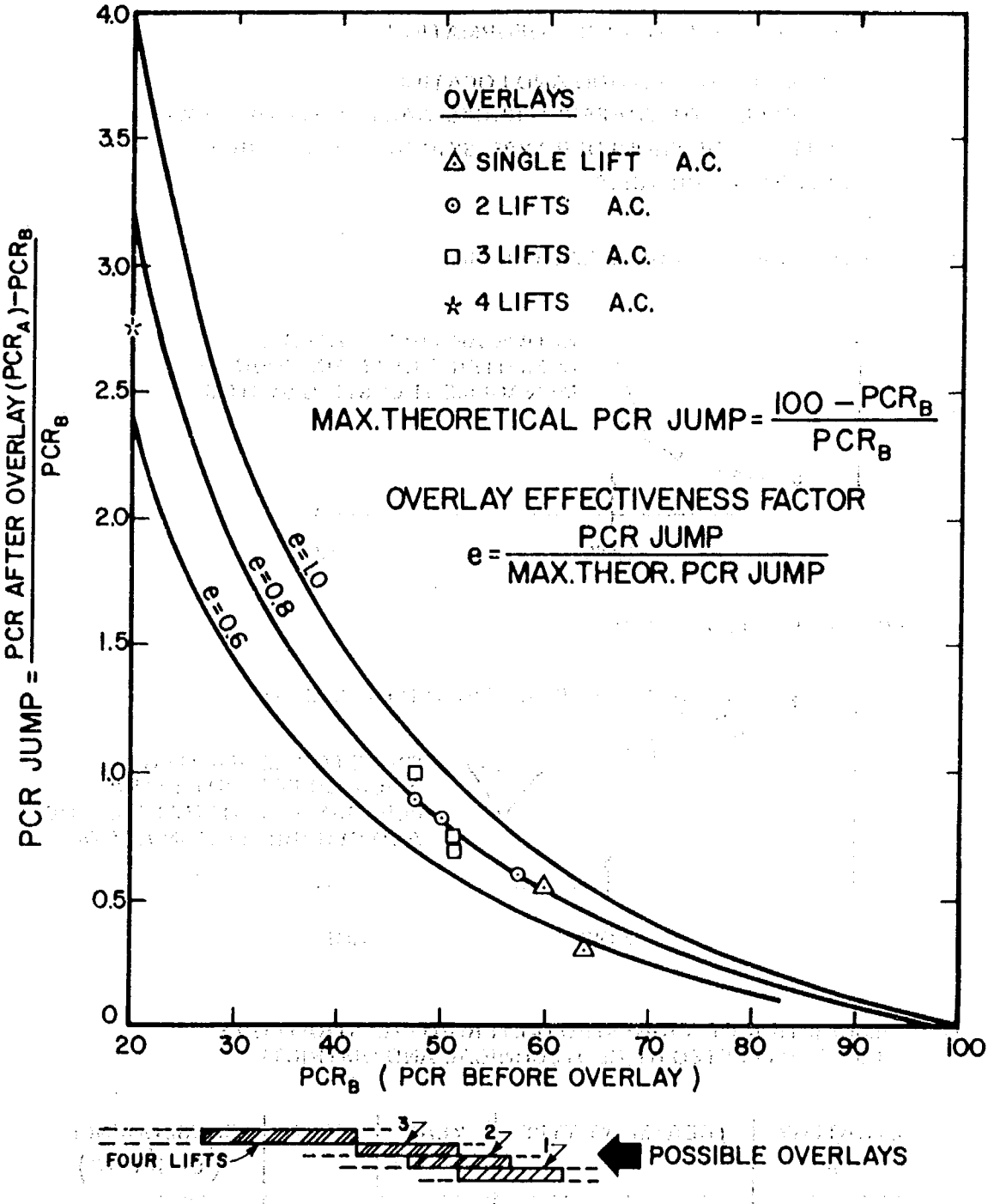


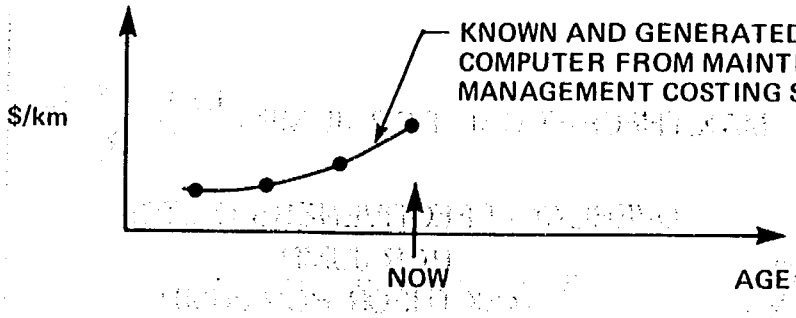
Figure 6/ Diagram Illustrating Effectiveness e of Overlay in Restoring Riding Quality (PCR).

POSSIBLE FORMAT OF ACTION PLAN FACT SHEET

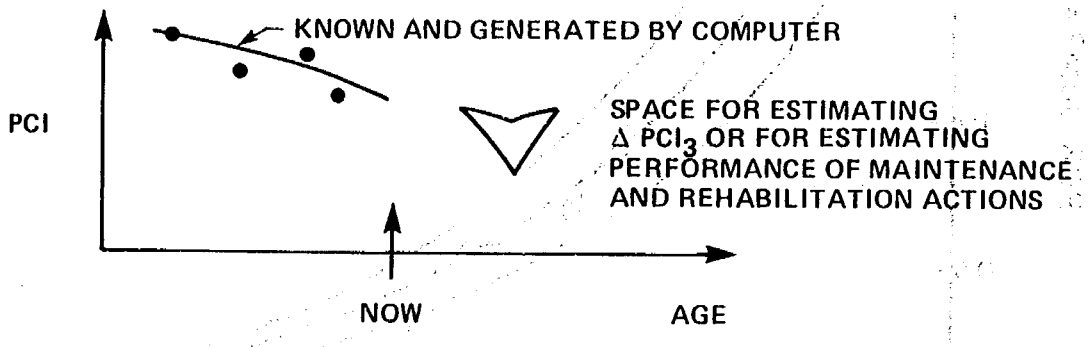
A – GENERAL AND BACKGROUND INFORMATION

- SECTION IDENTIFICATION AND LOCATION
- SECTION LENGTH, TRAFFIC (TRUCKS, AADT), RATE OF GROWTH
- PAVEMENT AND SHOULDER TYPE, SKID DATA IF POSSIBLE
- EXISTING DEFICIENCIES
- ETC

B – MAINTENANCE $ – AGE RELATIONSHIP



C – PCI – AGE RELATIONSHIP



D – DESCRIPTION OF SUGGESTED (ALTERNATIVE) STRATEGIES (TO BE COMPLETED BY GEOTECHNICAL AND DISTRICT)

STRATEGY No.	TREATMENT TYPE	YEAR	COST	CONSEQUENCES (BENEFITS)

Figure 7/ Possible Format of Action Plan Fact Sheet

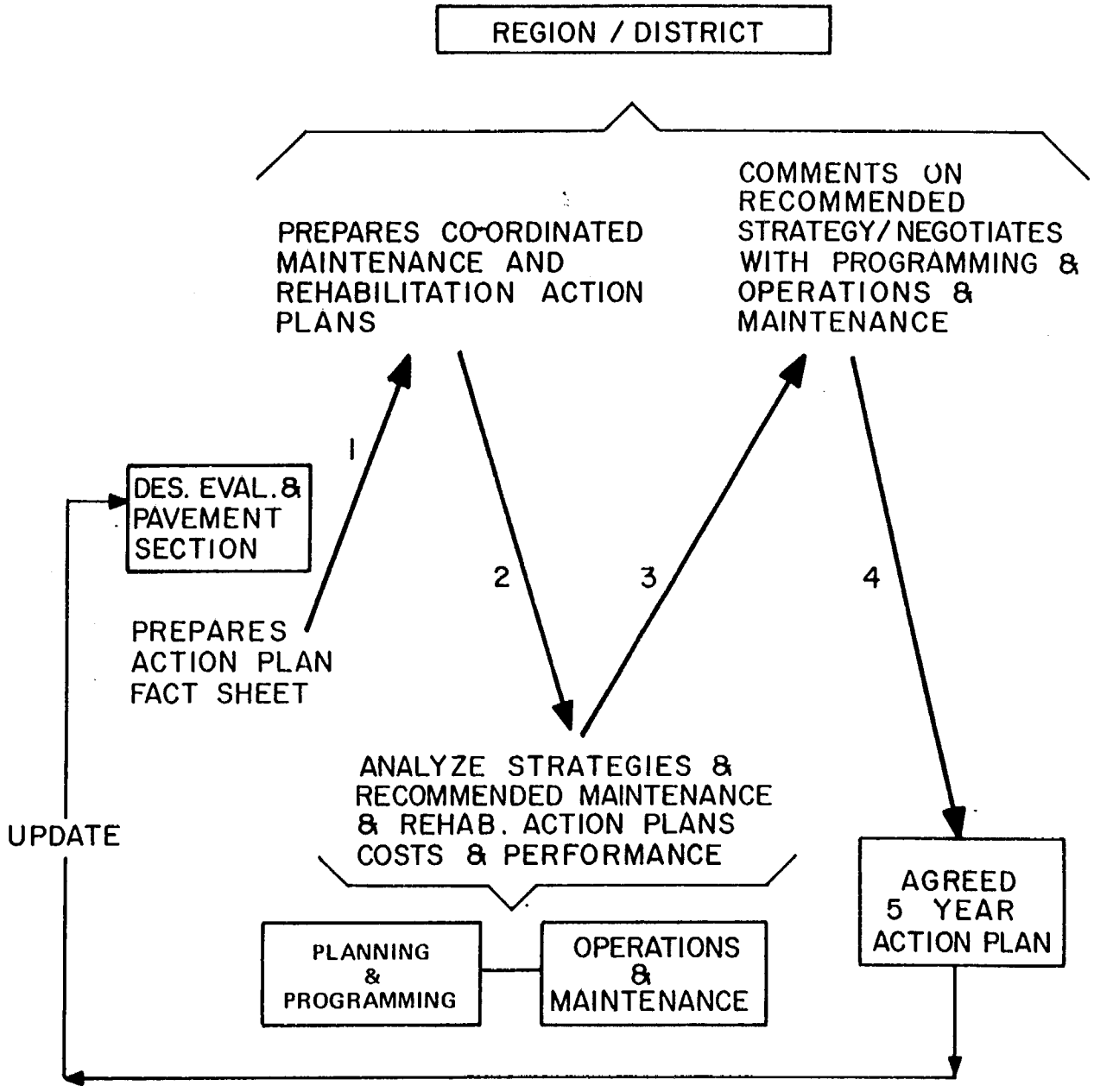


Figure 8/ Regional and District Role in PMS

Development of Pavement Management Systems for Expressways in Japan

Mr. M. Onodera, Mr. Y. Kawashima, Mr. I. Fukushima
Japan Highway Public Corporation

ABSTRACT

It is very important that pavement information generated from the past construction and maintenance activities be stored and utilized for later construction and maintenance pavement activities. Besides conventional technical considerations, effective methods of construction and maintenance of pavement with long-term economic consideration are demanded. In these circumstances, Nihon Doro Kodan (The Japan Highway Public Corporation) is developing computerized pavement management systems since 1980 in order to operate the present pavement construction and maintenance systems more effectively.

The pavement management systems consist of data bank, design subsystem and maintenance subsystem. The data bank stores on design, construction and rehabilitation of pavement, surface conditions, traffic conditions, etc.. The design subsystem offers information on thickness design of asphalt pavement in construction stage. The maintenance subsystem offers information on pavement maintenance for making maintenance program more effectively and reasonably.

The data bank and a part of the design subsystem and the maintenance subsystem have been developed. As a result, the followings can be conducted.

(1) Analyses of pavement conditions with statistical methods.
(2) Thickness design of asphalt pavement based on the Design Manual of Nihon Doro Kodan.
(3) Estimate of future needs for pavement rehabilitation with prediction method of rut depth progress.

DEVELOPMENT OF PAVEMENT MANAGEMENT SYSTEMS FOR EXPRESSWAYS IN JAPAN

by

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Nihon Doro Kodan

(Japan Highway Public Corporation)

1. PREFACE

All of the national expressways employ the toll system in Japan, and their structural standards are very high as understood from the divided highway structure with full access control. Construction and operation of national expressways is the responsibility of Nihon Doro Kodan, which was established in 1956 under the supervision of the Minister of Construction. Ever since a section of the Meishin Expressway (Nagoya-Kobe) was opened to traffic in 1963 as the first expressway in Japan, the total length of the expressways in operation has increased to reach 3,435 km as of March 1984. According to the national expressway network construction program, it is planned to build expressways totaling about 7,600 km in the future. So approximate 45 % of the planned length is in operation. On the other hand, 21 years have passed since the first expressway was opend to traffic.

After oil-crisis in 1973, conservation of resources and energy-saving have been strongly demanded socially in Japan and it is forecasted that these demands will become stronger. Length of expressways with short-lived maintenance cycles are increasing on account of deterioration of road and abrasion of pavement by studded tires and chained tires. As a result, maintenance cost for pavement is increasing rapidly [1]. For example, the pavement maintenance budget for the fiscal year 1983-84 was approximately 8 billion yen and it accounted for 11 % of the total maintenance budget. Since it is clear that the pavement maintenance cost increases drastically hereafter, more economical rehabilitation methods and more efficient invest-

ment methods of maintenance budget are demanded to develop urgently.

In this paper, we report on the general plan and present condition of the computerized pavement management systems which we are developing since 1980 in order to operate the present pavement construction and maintenance systems more efficiently. Hereupon, we describe systems concerning asphalt pavement, because the asphalt pavement is used almost exclusively for expressway on account of low initial cost, ease of rehabilitation, etc..

2. ASPHALT PAVEMENT ON EXPRESSWAYS

2-1 Flow of Pavement Activities

In a life-cycle of pavement, the pavement activities are divided into six main stages : planning, design, construction, surface conditions survey, programming and implementation of maintenance and rehabilitation. Fig.-1 shows the flow of the pavement activities.

Fig.-2 shows the organization of Nihon Doro Kodan. Expressways are constructed by a construction office and a construction bureau which supervises these on-site offices. On the other hand, expressways are maintained and operated by an operation office which takes charge of a section of an expressway of 50 to 60 km in length and an operation bureau which supervises 7 to 10 operation offices.

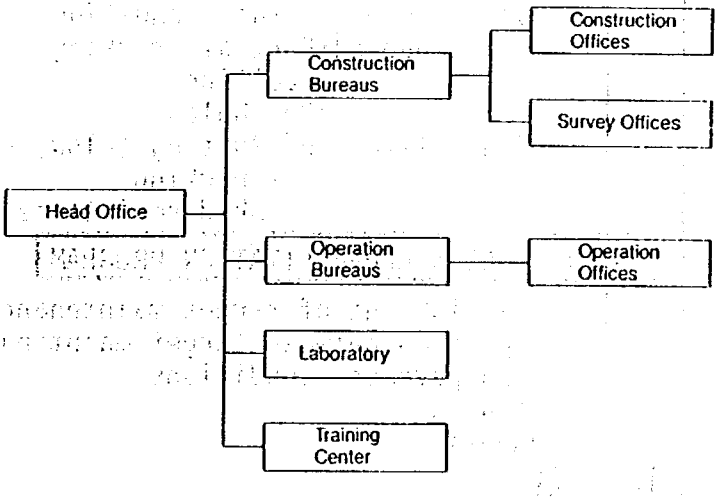


Fig.-2 General Organization Chart

The system of pavement technology in Nihon Doro Kodan is illustrated in Fig.-3. Pavement technology is systematized into two groups. The first is in the construction stage, and the other is in the operation stage. The former is mainly systematized in the Design Manual, the Execution Control Manual, etc. and the latter is in the Maintenance and Rehabilitation Manual, the

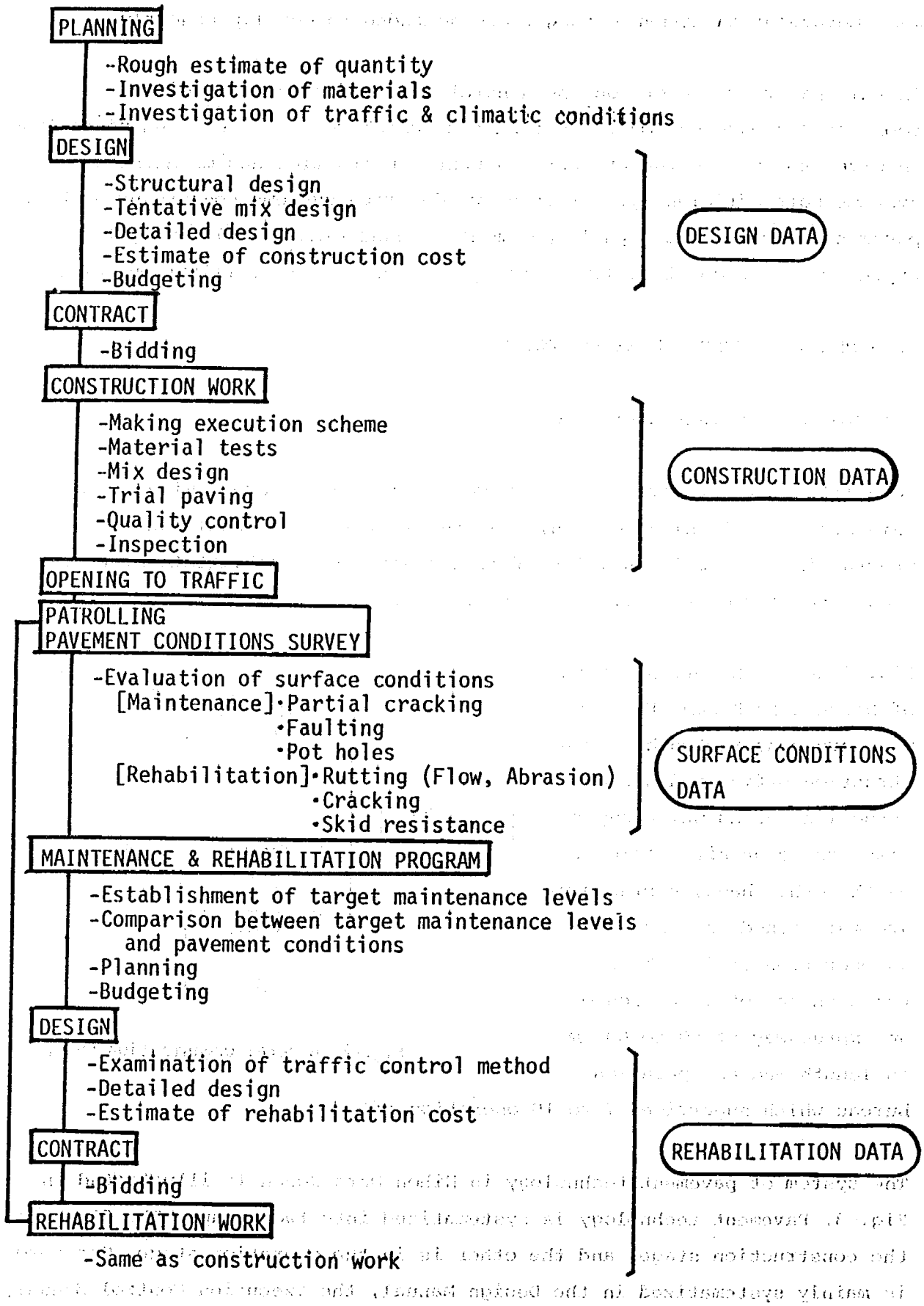


Fig.-1 Flow of Pavement Activities

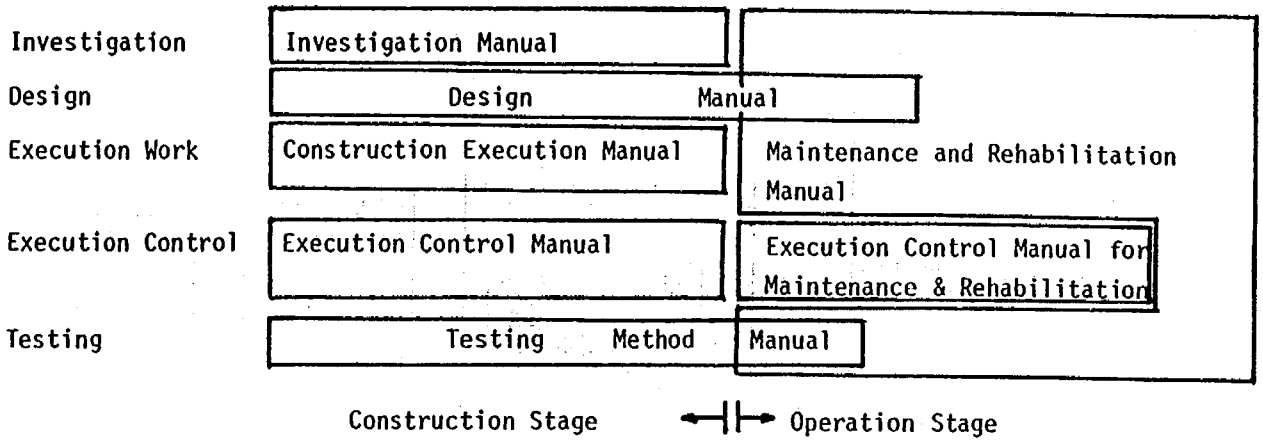


Fig.-3 System of Pavement Technology

Execution Control Manual for Maintenance and Rehabilitation, etc..

Main technology in the pavement activities are thickness design, mix design and quality control in the construction stage, and evaluation of surface condition, establishment of target maintenance levels and ordering priority in the maintenance stage. The important items will be briefly described in the following sections.

2-2 Construction Stage

2-2-1 Thickness Design of Asphalt Pavement

In the thickness design of asphalt pavement, the necessary pavement thickness is obtained from the total number of equivalent 10-ton axle loads in one direction during a 10-year period and the design CBR value of the subgrade. This method has been developed from the results of the AASHO Road Test with modifications made in the light of actual experience in Japan. The flow chart of design procedure is shown in Fig.-4.

Hereupon, with expressways in areas susceptible to frost action, if the frost penetration is greater than the total thickness of the pavement, the materials of the subgrade within the depth of frost penetration are to be replaced with materials unsusceptible to frost.

2-2-2 Mix Design of Asphalt Mixture

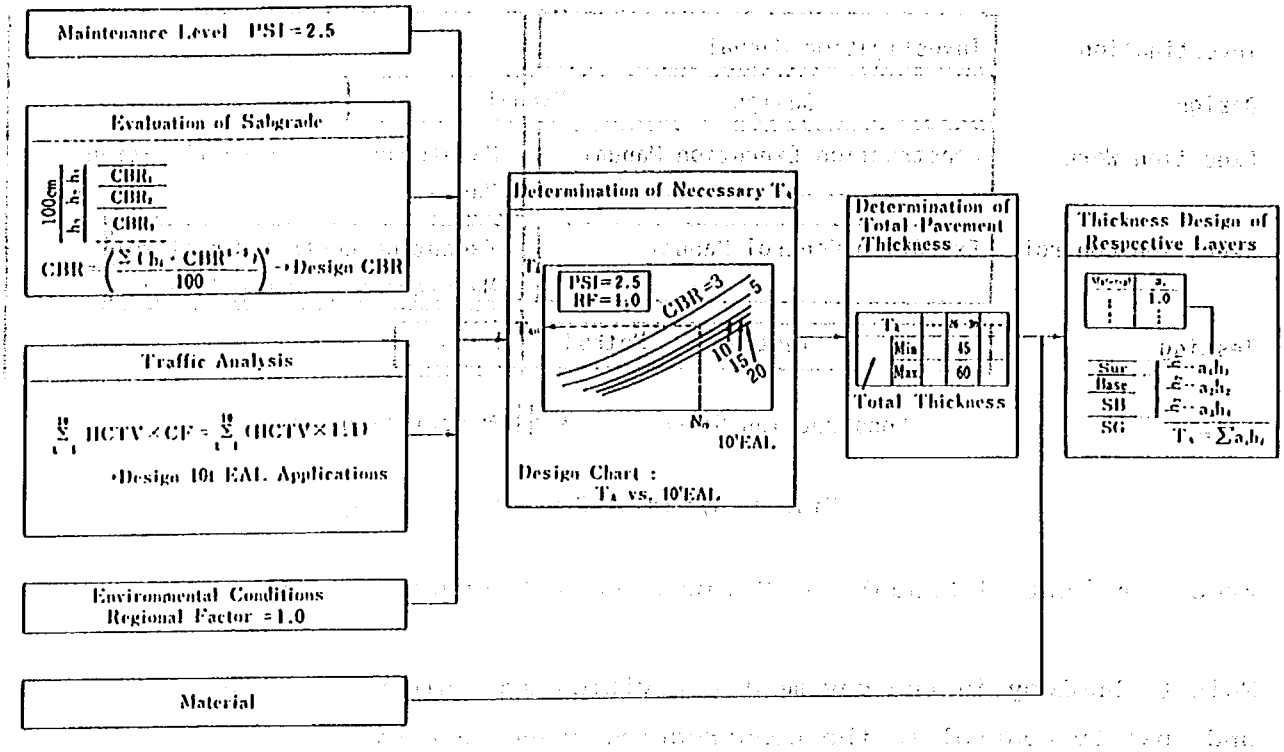


Fig.-4 Structural Design Procedure for Asphalt Pavements

Japan is an elongated country from north to south and the climatic conditions vary markedly from place to place. Traffic conditions, such as the travelling speed, axle load and percent of large-sized vehicles to the total traffic volume, also vary from place to place. These factors are given full consideration in selecting the types of surface course mixture. In the northern area where there is much snowfall, the surface course must be highly resistant to abrasion, while in the central and southern areas where the climate is generally mild to warm, the surface mixture must be highly resistant to plastic flow. With regards to the mix design of surface course in snowy and cold areas, reference [2] is available.

2-3 Maintenance and Rehabilitation Stage

2-3-1 Evaluation of Pavement Conditions

Evaluation of pavement conditions is the most fundamental activity for the reasonable and systematic maintenance of pavement. The pavement characteristics are divided into two groups :

a) pot-hole, faulting, localized cracking and roughness

b) rutting, cracking and skid resistance

The former is usually evaluated, judged and repaired by the individual operation office. For evaluation of the former characteristics, patrolling or visual inspection is the practical method because these characteristics are local and it is difficult to measure quantitatively and automatically. The latter, on the other hand, is usually observed on a large scale and needs major rehabilitation. An operation bureau evaluates, judges and plans the repairs. Repair work itself is done by an appropriate operation office even in this case. For evaluation of the latter characteristics, new techniques suitable for the inventory study are adopted aiming at quantitative and high speed measuring.

The outline of measurement methods, used for evaluation of pavement conditions in expressways, is described in the following.

(1) Rutting

Rut depth is defined as the vertical distance from the highest point to the lowest point within a lane as shown in Fig.-5.

Measurement methods of rut depth are roughly divided into two groups. In the case of localized extraordinary inspection or periodical survey on the expressways where rutting does not much develop, 3.6 m straight edge or transverse profilometer is mainly used. On the other hand, in the case of periodical survey on the expressways where the rutting develops well, the Rut Depth Photographic Recorder is usually used. This is composed of an illumination unit with hair line projector, camera unit and pulse signal transmitter which actuates the photographical interval. Rut depth is usually measured at every 100 m. With regards to the measuring principle, reference [2] is available.

Though it is prescribed in the Manual that measurement should be conducted every year in principle, it is very difficult because of extension of expressways and limited budget. Since rut depth can be predicted to some degree by introducing pavement management systems, mentioned later, we are examining the frequency of measurement according to importance of road and progress speed of rut depth.

(2) Cracking

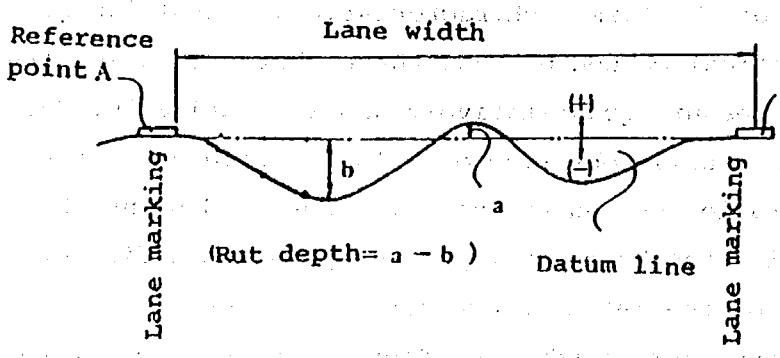
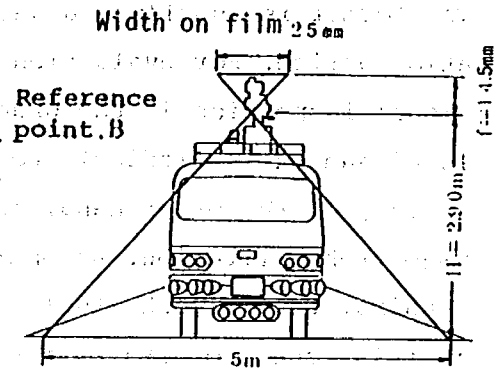


Fig.-5 Definition of Rut Depth



Photographing width on a road

Fig.-6 Road Surface Photographic Recorder

To express the cracked asphalt pavement condition quantitatively, crack ratio defined by Eq.-1 is adopted in Japan.

$$\text{Crack ratio} = \frac{\text{cracking areas + patching areas}}{\text{measured areas}} \times 100 \qquad (1)$$

Measurement method of cracking are roughly divided into two groups. The conventional method for cracked pavement is inspection and sketching by the pavement engineer. A new method is conducted by using Road Surface Photographic Recorder (see Fig.-6) which can record cracking continuously. It consists of a camera unit, illumination unit, power source unit and signal transmitter.

In determination of the cracked and patched areas, one-quarter lane width (= 0.875 m) x 1.0 m mesh plate is overlapped on the film or sketch, and the number of the meshes covering the cracks and patching is counted. The areas of alligator cracking or patching is determined by the number of meshes counted multiplied by the individual mesh area (0.875 m^2) and the area of longitudinal or transverse cracking is determined by the number of meshes multiplied by 0.875 x 0.3 = 0.263 m^2. The crack ratio is calculated every 100 m in principle.

(3) Deflection

Since the structural evaluation of pavement is necessary for selection of rehabilitation method and thickness design of overlay on the cracked pavement, surface deflection is measured at the same time when cracking is measured. The most popular measurement method for deflection is a method

using the Benkelman Beam on expressways.

(4) Skid Resistance

Skid resistance between the pavement and a tire is measured by a locked wheel skid test vehicle. The general view of skid test vehicle is shown in Fig.-7. A test vehicle travels at a constant speed with rolling the test wheel. When it is desired to measure the skid resistance, the test wheel is completely locked and the generated bracking force, F, and the wheel load,P, are simultaneously measured. Then skid number, SN, is determined by Eq.-2.

$$SN = \frac{F}{P} \times 100 \qquad (2)$$

Measuring conditions are as follows.

a) measuring speed : 80 km/h. Besides, 40 and 60 km/h at typical section.
b) measuring interval : 200 m
c) water film : 0.5 - 1.0 mm

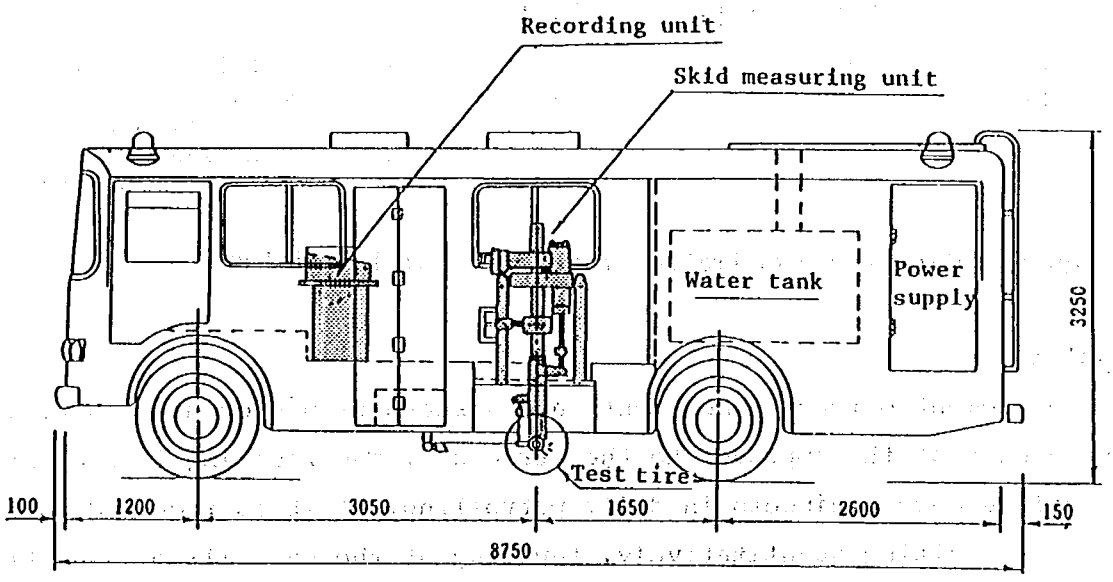


Fig.-7 General View of Skid Test Vehicle

(5) Longitudinal Surface Roughness

Longitudinal surface roughness is evaluated by riding comfort in daily patrolling. Roughness hardly become a great problem on expressways. However, when it become a problem especially, profilometer with wheel base length of 8 m, termed as 8 m-profilometer shown in Fig.-8, is used for measuring the roughness. Measured roughness is quantitatively expressed in Profile Index, PrI, as shown in Fig.-9.

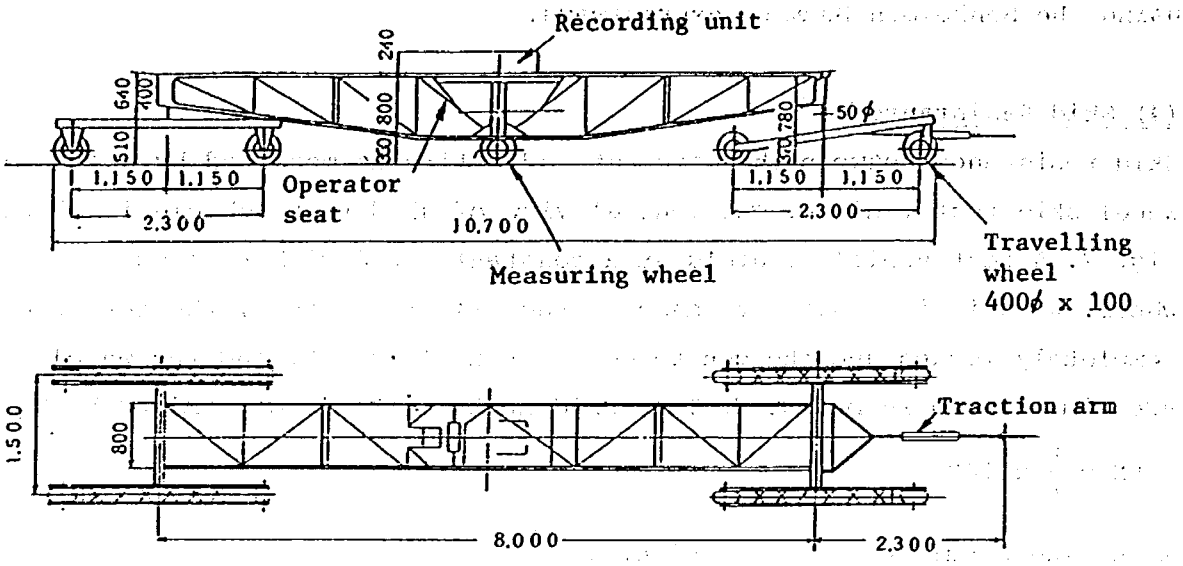
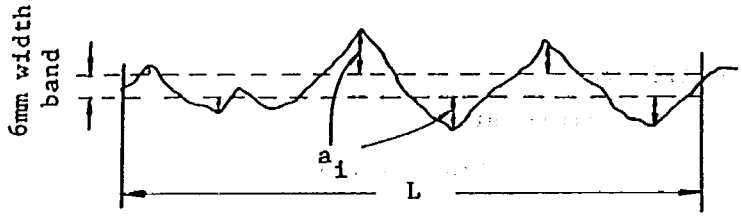


Fig.-8 General View of 8 m-Profilometer



$$PRI = \frac{\Sigma a_i}{L}$$

where
PRI = Profile index (cm/km)
a_i = Deviation in excess of 6mm band (cm)
L = Total length investigated (km)

Fig.-9 Determination of Profile Index, PrI, from Recorded Profile

(6) Faulting

Uneven settlement between embankments and structures has occured mostly, causing a bump at the approach to the structure. The faulting is evaluated by the maintenance engineers in daily patrolling. If it is required to express the faulting quantitatively, faulting at the backfill of the structure is measured by the string and rule. The string 15 m long is stretched along outer wheel path from the expansion joint of the abutment. Depth of the pavement surface from the base line string is measured and the maximum depth is named the amount of faulting.

(7) Pot Holes

Pot holes are evaluated by inspection and sketching as the case of cracking. Area and size of pot holes, degree of asphalt stripping, uniformity of mixture gradation, possible hardening of asphalt and other related conditions are carefully examined.

2-3-2 Establishment of Target Maintenance Levels

After pavement conditions are measured, in order to judge whether maintenance or rehabilitation is necessary and to make a systematic and reasonable maintenance program, it is necessary to set the target maintenance levels. The target maintenance levels for expressways are set as in Table-1.

Table-1 Target Maintenance Levels for Expressways

Pavement characteristics	Classification	Level-A *1	Level-B *2
Rut Depth (mm)	Through lane	15	25
	Track climbing lane	20	40
Crack Ratio (%)	All site	20	40
Skid Number	Standard site	30	25
	Important site	35	30
Faulting	Backfill of abutment	-	20
	Backfill of box culvert	-	30
	Expansion joint	-	15

*1 Level-A : Pre-maintenance level in which immediate rehabilitation is not required but examination of distress causes and planning of rehabilitation should be considered.

*2 Level-B : Target level, or rehabilitationis desired to be done up to this level.

2-3-3 Ordering of Priority

The figures in Table-1 are set for individual pavement characteristics. However, different types and degrees of distresses exist on actual road surface and, if it is necessary to decide the priority of rehabilitation on such a pavement, a synthetic evaluation is made by summing up each grade shown in Table-2 corresponding to the pavement condition value. Evaluation

Table-2 Evaluation Scales for Surface Conditions

Evaluated Grades		Level-A	1	2	3	4	5	Level-B
Priority			5	4	3	2	1	
Rut Depth	Through lane	15 mm	~17	~19	~21	~23	~25	25 mm
	Track climbing lane	20 mm	~24	~28	~32	~36	~40	40 mm
Skid Number	Standard site	30	~29	~28	~27	~26	~25	25
	Important site	35	~34	~33	~32	~31	~30	30
Crack Ratio		20 %	~24	~28	~32	~36	~40	40 %

scales of pavement shown in Table-2 are set on the assumption that weight for each distress is equivalent. However, regional conditions, traffic conditions and social demands are actually considered when maintenance programs are examined.

2-3-4 Selection of Execution Method for Maintenance and Rehabilitation

Adequate execution method for maintenance and rehabilitation is selected in due consideration of road class, type and degree of distress and its cause. Execution methods are mainly classified into two classes on expressways.

a) Method for maintenance
   i) Method as emergency measures --- patching, filling pot holes, leveling a faulting at backfill of abutment, etc.
   ii) Method which needs timely measures --- surface treatment, surface milling, treatment for decrease of skid resistance or localized cracking, etc.

b) Method for rehabilitation
   i) Method adopted from long-term point of view --- overlay, overlay with surface milling, etc.

3. OUTLINE OF PAVEMENT MANAGEMENT SYSTEMS

3-1 Purpose

As mentioned in Chapter 2, pavement technology on design, construction and maintenance are systematized in outline in Nihon Doro Kodan. On the other hand, as the length of the expressways in operation increases, the amount of pavement information generated from the past construction and maintenance activities is becoming too vast to handle by the hands. Then, Nihon Doro Kodan started developing the computerized pavement management systems in 1980. The purposes of development of the pavement management systems are

1) to process, store and utilize the pavement information generated from the past construction and maintenance activities for later activities,
2) to offer information for selecting more economical pavement structure and
3) to offer information for making more efficient and reasonable maintenance program.

3-2 General Plan

The activities which will be processed by using computer in the pavement activities shown in Fig.-1 are :

1) thickness design of asphalt pavement in design stage,
2) analyses of pavement conditions, prediction of future pavement conditions, selection of rehabilitation alternatives (maintenance level, execution method, etc.) and economic analysis in operation stage, and
3) storage and retrieval of pavement information.

General plan of the computerized pavement management systems is illustrated in Fig.-10 and Fig.-11. The computerized pavement management systems consist of design subsystem, maintenance subsystem and data bank. We intend to conduct pavement design and maintenance more reasonably and efficiently with the help of the pavement management systems based on data which is stored in the data bank.

3-3 Data Bank

Pavement information generated from the past activities is collected and stored in the data bank. The data bank offers data to the maintenance subsystem and the design subsystem. The data bank consists of data storing subsystem and analyzing subsystem. The data analyzing subsystem outputs statistical charts and surface conditions charts shown in Table-4. The data is automatically checked whether it is reasonable in the light of experience.

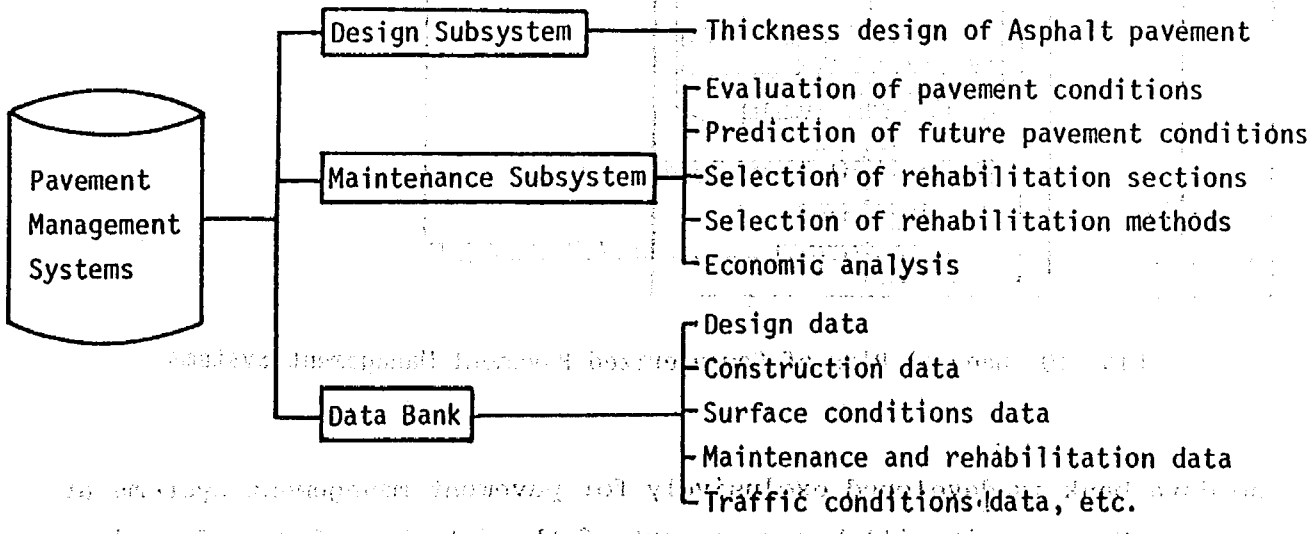


Fig.-11 Pavement Management Systems

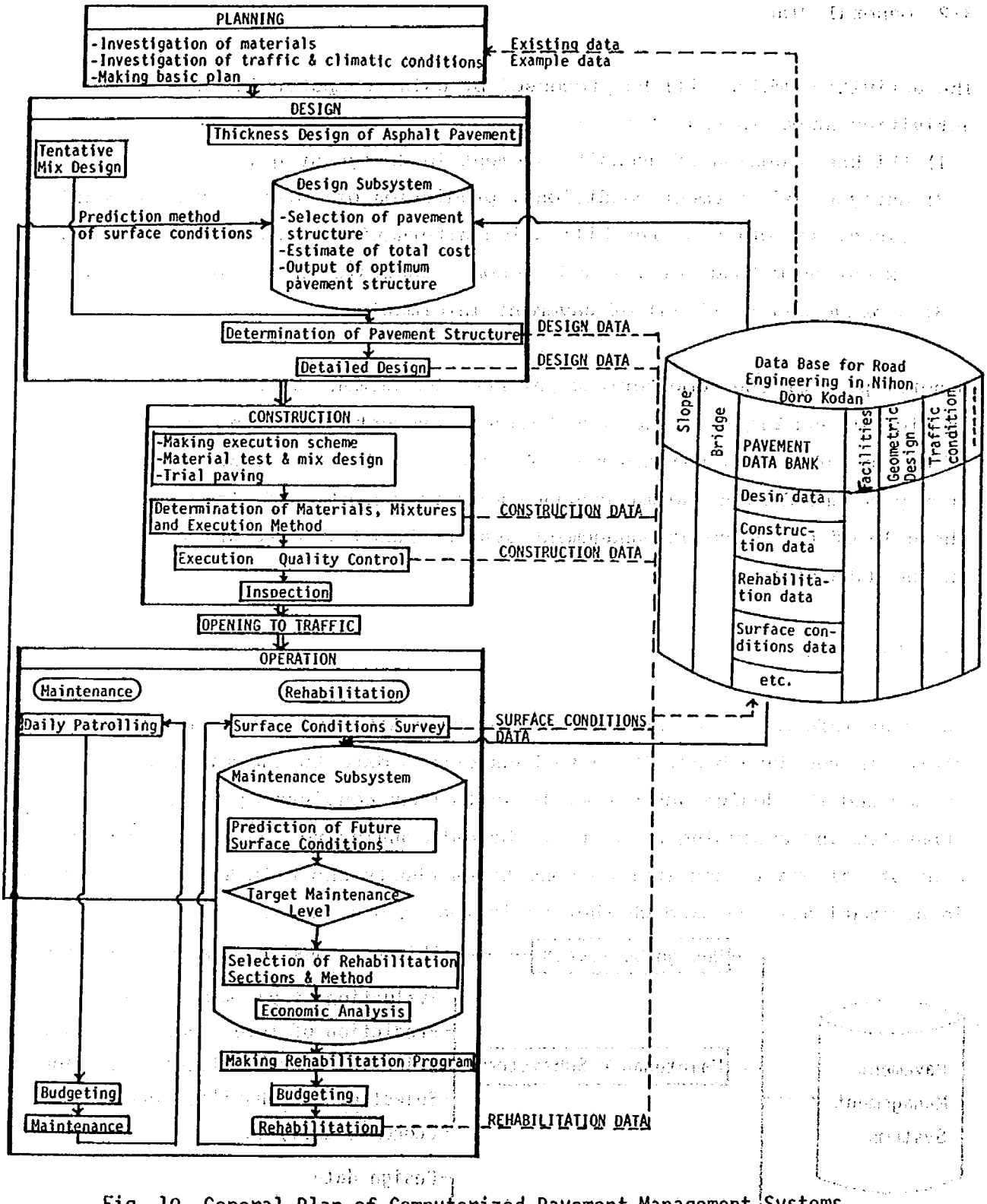


Fig.-10 General Plan of Computerized Pavement Management Systems

The data bank is developed exclusively for pavement management systems at present. However, it will become a part of the data base for road engineering in Nihon Doro Kodan shown in Fig.-10 in the near future.

3-3-1 Input Data

Input data are divided into six large groups shown in Table-3.

Table-3 Input Data

Design Data	Subgrade condition
	Pavement structure
	Geometric design data
Construction Data	Mixture data (Su., Bi., A.T.B.,C.T.SB.,G.SB.)
	Material data (Su., Bi., A.T.B.,C.T.SB.,G.SB.)
Rehabilitation Data	Purpose, Section, Date, Cost, Method, etc.
	Mixture data (Su., Bi.)
	Material data (Su., Bi.)
Surface Condition Data	Rut depth
	Crack ratio
	Skid number
Traffic Conditions Data, etc.	Traffic conditions (Total and large-sized vehicle traffic volumes)
	Location data (Major facilities)

Remarks : Su = Surface course
Bi = Binder course
A.T.B. = Asphalt treated base course
C.T.SB. = Cement treated subbase course
G.SB. = Granular subbase course

(1) Design Data

Design data are divided into subgrade condition data, pavement structure data and geometric design data as shown in Table-3. Subgrade condition data include subgrade composition, materials, etc.. Pavement structure data include design conditions such as design CBR, design traffic volume and T_A, and pavement composition such as total thickness and thickness of respective layers. Geometric design data include number of traffic lanes, vertical alignment, etc..

Data before March 1983 (length of expressways in operation is 3,232 km) are inputted and the input ratio (ratio of the inputted length to the length in operation) is approximately 90 %.

(2) Construction Data

Construction data are divided into mixture data and material data as shown in Table-3. Mixture data include mix proportion, gradation and characteristics of mixture such as Marshall stability, unconfined compression strength,

etc. for surface course, binder course, asphalt treated base, granular and cement treated subbase. Material data include properties of aggregate, screenings, filler and asphalt. Date of opening to traffic, contractor, etc. are also inputted.

Data before March 1983 are inputted and the input ratio is approximately 90 %.

(3) Rehabilitation Data

Rehabilitation data are divided into purpose and other data, mixture data and material data. Purpose and other data include purpose, section, quantity, cost, date, method, cause, contractor, etc..

Purpose and other data before 1983 are inputted except a part of expressways.

(4) Surface Conditions Data

There are many characteristics of surface conditions such as rutting, cracking, skid resistance, roughness, faulting, pot holes, etc.. However, we store rutting, cracking and skid resistance data as surface conditions data in the data bank, because these distresses occupy most part of rehabilitation causes on expressways and can be measured automatically at high speed. In principle, rut depth and crack ratio data are inputted every 100 m and skid number is every 200 m.

Measurement of rutting and cracking are being conducted since 1975 and these data are entirely inputted. Hereupon, recent measurement ratio (ratio of the measured lane length to the lane length in operation) of rutting is approximately 40 % and of cracking is approximately 15 %. Input ratio (ratio of the inputted lane length to the lane length in operation) of skid number is approximately 20 %.

(5)Traffic Conditions Data

Total and large-sized vehicles traffic volumes are inputted.
Data before March 1983 are almost inputted.

3-3-2 Example of Output

Example of output from the data bank is listed in Table-4 and is illustrated in Fig.-12.

Table-4 Examples of Output

Statistical Charts on Pavement Design & Construction	
-Subgrade Condition-------	Thickness, Design CBR, etc.
-Pavement Structure-------	Thickness & T_A for respective layers
-Properties of Mixtures	
Subbase----------	Type, Mix proportion, Gradation, Modified CBR
Base (A.T.B.)--------	Mix proportion, Gradation, Asphalt content, Marshall test value, etc.
Surface, Binder------	Mix proportion, Gradation, Asphalt content, Marshall test value, etc.
-Properties of Materials	
Asphalt----------	Penetration, Softening point, etc.
Aggregate----------	Physical properties for respective kinds of rock (Abrasion loss, Soundness, Amount of water absorption, etc.)
[Items of Output]------	Max., Min., $\bar{x}$, σ, n, Distribution charts

Surface Conditions Chart
See Fig.-13

Statistical Charts on Surface Conditions & Rehabilitation	
-Rutting----------	Measured length, n, Lane length & % of RD>x mm, $\bar{x}$, σ, Relative frequency, Date of measurement ——totalized by respective sections between two successive interchanges, Route, Structure type (Bridge, Tunnel)
-Cracking----------	Measured length, Section of CR>x % ; Relative frequency, n, length of CR>x %, $\bar{x}$, and σ for the data with CR>1 % ——totalized by route
-Rehabilitation----------	Sections, Total length & %, Length of every rehabilitated times, Relative frequency of years before the first rehabitation ——totalized by respective route, execution method, causes
-Traffic Conditions-------	Cumulative traffic volume of total & large-sized vehicles

Remarks : RD = Rut Depth, CR = Crack Ratio, A.T.B. = Asphalt Treated Base

3-4 Maintenance Subsystem

The maintenance subsystem is at the core of the computerized pavement management systems and its purpose is to offer information on pavement rehabilitation in the future in order to invest rehabilitation budget efficiently and to make effective and reasonable rehabilitation program.

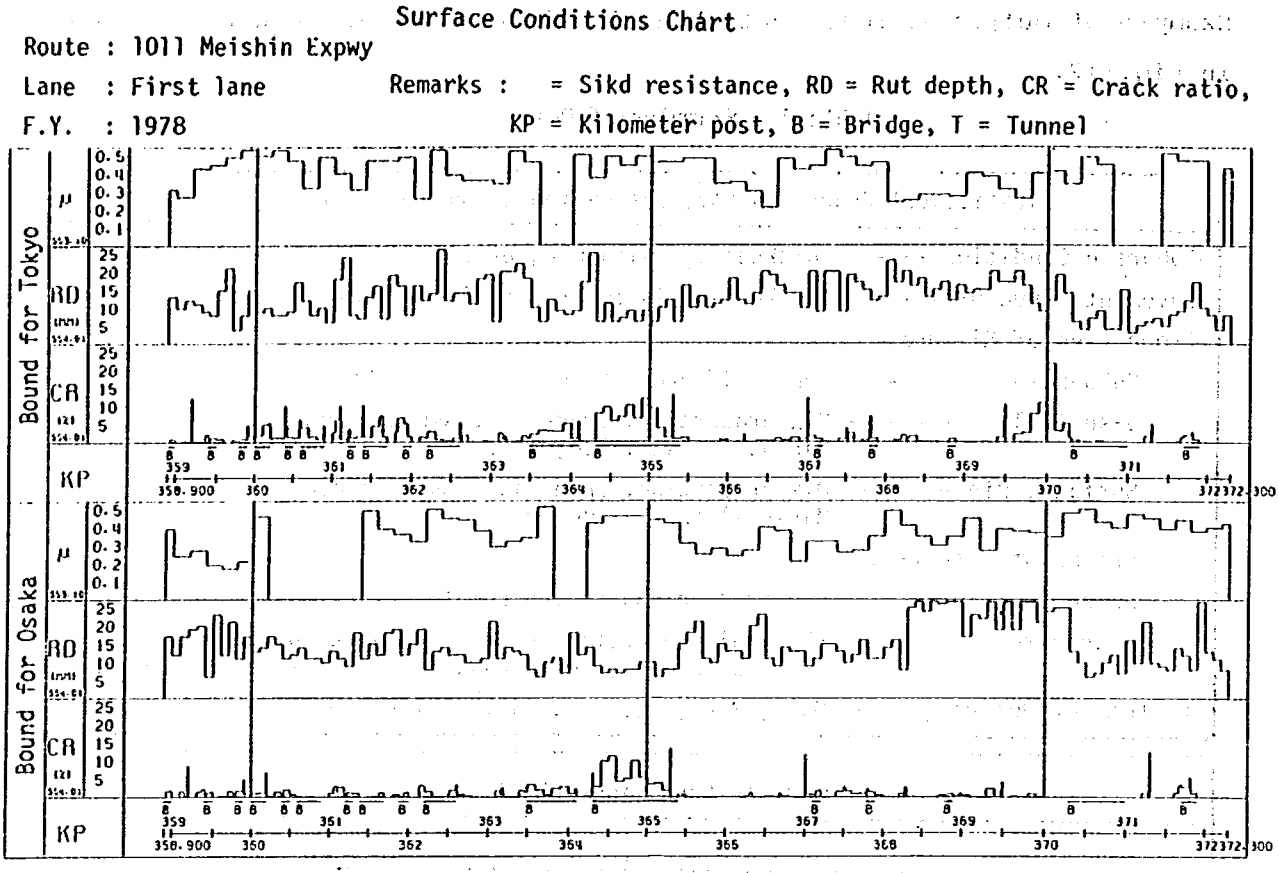


Fig.-12 Example of Surface Conditions Chart

Characteristics of surface conditions handling in the maintenance subsystem are limited to rutting and cracking at present. The reason for it is as follows :

a) Rutting and cracking occupy most part of rehabilitation causes on expressways in Japan.

b) When skid resistance becomes a problem, some measures are taken regardless of other surface conditions.

c) Longitudinal roughness hardly becomes a great problem on expressways. If it is judged that the riding comfort at the backfill of abutment is bad in daily patrolling, the measures are conducted in maintenance works.

Fig.-13 shows the flow chart of the maintenance subsystem and it is divided into the following five steps.

a) Analyses of surface conditions

b) Prediction of future surface conditions

c) Selection of rehabilitation sections

d) Selection of rehabilitation method

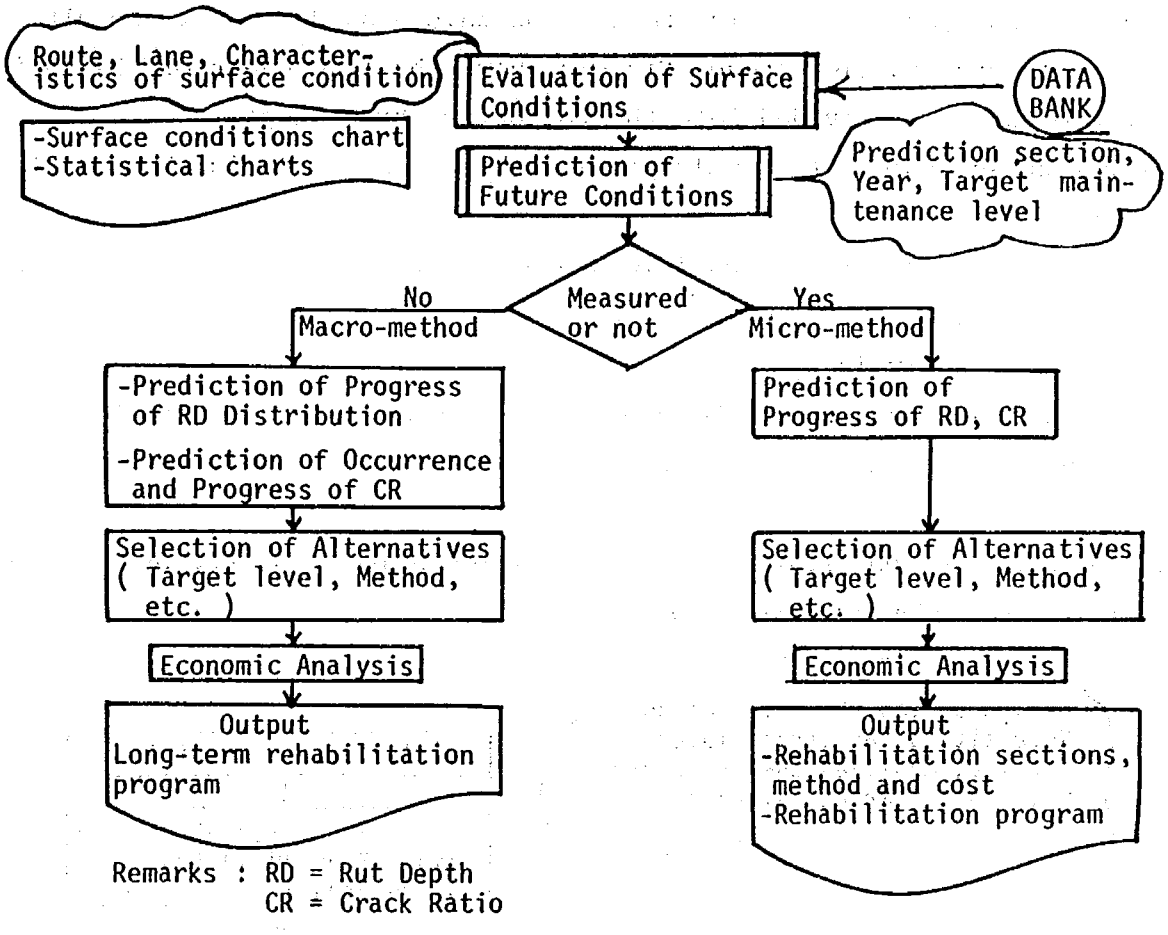


Fig.-13 Maintenance Subsystem

e) Economic analysis

a) is evaluations and analyses of actual surface conditions and their yearly changes. b) and c) will be described in detail in Chapter 4. d) is selection of adequate rehabilitation method considering surface conditions, rehabilitation records, regional conditions, etc.. e) is economic analysis for alternatives selected in c) and d). Rehabilitation cycles for respective alternatives are presumed and cost and benefit are estimated for given periods of time.

The maintenance subsystem on rutting is developed to the extent of the above-mentioned " c) selection of rehabilitation sections " and the rest of the subsystem is sheduled to complete by March 1985. As for cracking, evaluation method is being studied.

3-5. Design Subsystem

The design subsystem offers information on thickness design of asphalt pavement in construction stage. Fig.-14 shows the flow chart of the design subsystem and it is divided into the following three steps.

a) Thickness design of asphalt pavement based on the Design Manual of Nihon Doro Kodan

b) Prediction of surface conditions and maintenance cycle after opening to traffic

c) Economic analysis

Methods in the maintenance subsystem will be adopted to the step b) and c). So it can be said that the step peculiar to the design subsystem is a).

The thickness design program of asphalt pavement based on the Design Manual of Nihon Doro Kodan is developed at present.

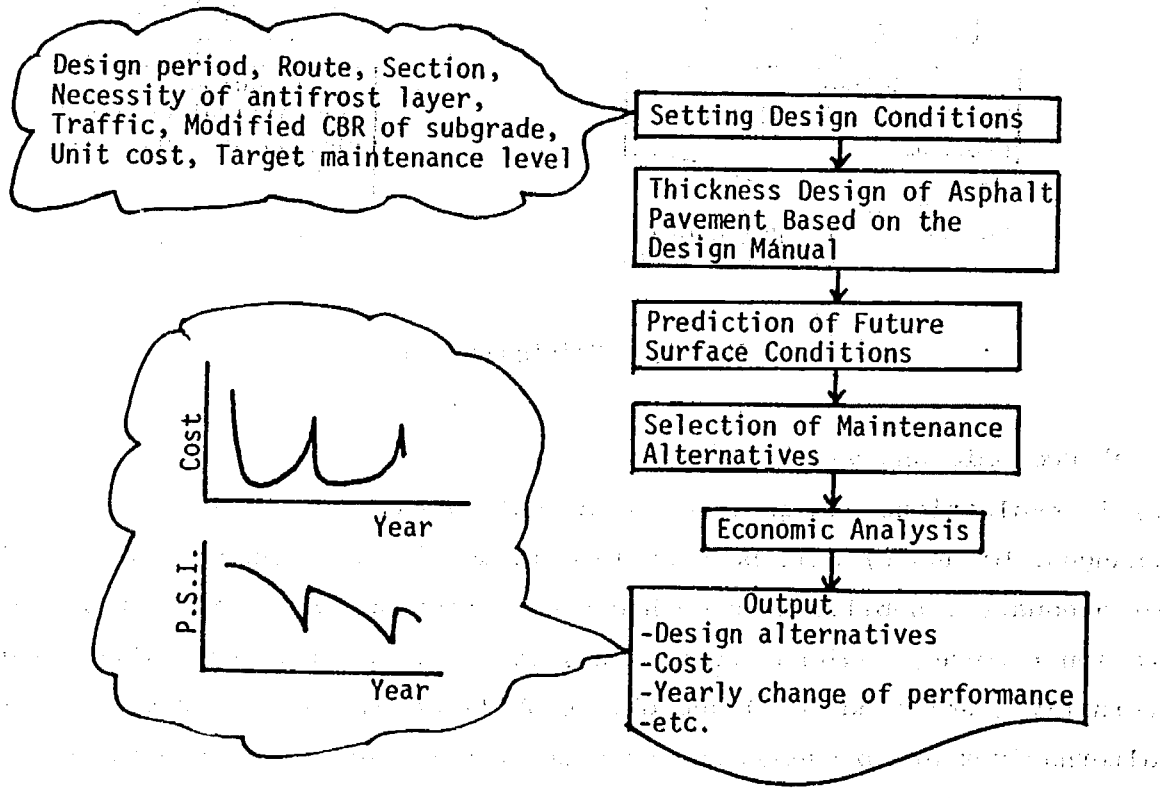


Fig.-14 Design Subsystem

4. PREDICTION OF RUT DEPTH PROGRESS AND REHABILITATION LENGTH

4-1 Prediction of Rut Depth Progress

Table-5 shows factors which influence the progress of rut depth. Prediction method of rut depth progress may become more accurate, if all the factors

shown in Table-5 are considered. However, there are many factors which are difficult to collect or estimate in order to input a prediction method . For example, it is clear that the equipped rate of tire chains or studded tires has a great influence on abrasion of pavement [2]. However, it is very difficult to collect or estimate the equipped rate data because the equipped rate varies extremely by place, time, traffic regulation condition, etc.. The same may be said about the pavement temperature. On the other hand, the traffic volume is related to the progress of rut depth [2] and it can be estimated correspondingly to a change of conditions which surround the expressways. So we tried to predict the progress of rut depth by large-sized vehicle traffic volume as a temporary measure. Large-sized vehicle is defined as the car which total weight is 8 tons or more or maximum carrying capacity is 5 tons or more, and the bus of which passenger capacity is 30 or more.

Table-5 Factors Influencing Rut Depth Progress

(1) Traffic Conditions	Traffic volume, Wheel load, Travelling speed, Equipped ratio of studded tires or tire chains, Travelling position
(2) Material and Mixture Conditions	Hardness of aggregate, Maximum size of aggregate, Asphalt content, Hardness of asphalt, Amount of coarse aggregate, Degree of compaction
(3) Environmental condition	Pavement temperature, Dry or wet of surface
(4) The Others	Snow removal condition, etc.

Prediction methods of future rut depth are classified into two classes ; the micro-method and the macro-method. As a previous portion of the prediction methods, there is a traffic prediction method. The outline of them will be described below.

4-1-1 Prediction of Traffic Volume

Future traffic volume is predicted by adding increase of traffic volume to the actual traffic volume in basic year and it is shown by Eq.-3.

$$T = T_0 + \beta \times t \qquad (3)$$

where, T : large-sized vehicle traffic volume in the predicting year (vehicles / day - one direction)

T_0 : actual traffic volume in basic year (vehicles / day - one direction)

β : increase rate of traffic

t : period of prediction (years)

β is a linear regression coefficient between large-sized vehicle traffic volume and year in the past 3 - 5 years. It is based on the conception that traffic volume in the near future (approximate 5 years after) is on the extension line of actual traffic volumes. However, when this conception cannot be applied, for example, if the network of expressways will be formed, or if the expressway is not opened to traffic, we use the traffic volume obtained from the projected traffic volume table which is developed by Nihon Doro Kodan.

4-1-2 Prediction of Rut Depth

Rut depth is predicted by either of the following two methods according to the existence or the non-existence of measured values.

- Micro-method : method predicting the rut depth based on the measured values
- Macro-method : method predicting the progress of rut depths distribution which is regarded as a normal distribution

(1) Prediction of rut depth by the micro-method

Rut depth is predicted by Eq.-4.

$$RD = RD_0 + \lambda \times T \times t \qquad (4)$$

where, RD = predicted value of rut depth (mm)

RD_0 = measured value of rut depth (mm)

T = estimated large-sized vehicle traffic volume (vehicles / day-one direction)

t = period of prediction (days)

λ = coefficient of rut depth progress classified by lane and structure type

$\lambda = \alpha \times \gamma \times \kappa$

α : average coefficient of rut depth progress on the first lane in the earth work section

γ : lane coefficient κ : structure type coefficient

α is calculated for every direction and section between two successive interchanges by Eq.-5. In this calculation, the rut depth data on the first lane in the earth work section, where rut depth was measured twice or more and rehabilitation was not conducted between these measurements, are used.

$$\alpha = [\ \Sigma\ (\ RD_2 - RD_1\ )\ /\Delta T\ ]\ /\ N \qquad (5)$$

where, RD_2 = rut depth in a certain year (mm)

RD_1 = rut depth in the year before a certain year (mm)

ΔT = passages of large-sized vehicles in one direction during the rut depth increase from RD_1 to RD_2 [10,000 passages]

N = number of measured value of rut depth

γ and κ are calculated by Eq.-6 and Eq.-7.

$$\gamma = \alpha_n\ /\alpha \qquad (6) \qquad \kappa = \alpha_s\ /\ \alpha \qquad (7)$$

where, α_n = α in the earth work section on some lane (second lane, passing lane or climbing lane)

α_s = α on the first lane in some section (bridge and viaduct or tunnel)

The above-mentioned are in the case that the coefficient can be calculated from the measured values of rut depth. On the other hand, in the case of no measured data, coefficients α, γ and κ are set as Table-6 in consideration of regional conditions, traffic conditions, etc..

Table-6 Coefficients of Rut Depth Progress and Correction Coefficients

Item	Mark	Classification	Coefficient	Remarks
Coefficient of rut depth progress on the first lane in the earth work section	α	Doo Expwy ~ Tomei Expwy	0.50 ~ 0.01	
Lane coefficient	γ	Second lane	1.20	
		Passing lane	0.55	
		Climbing lane	1.30	Tomei & Meishin Expwy
		Climbing lane	3.00	The others
Structure type coefficient	κ	Bridge & viaduct	0.80	Tomei & Meishin Expwy
			1.05	The others
		Tunnel	2.70	Region suffing abrasion
			0.70	The others

(2) Prediction of rut depth by the macro-method

Fig.-15 shows a relation between the average rut depth and its standard deviation in a section between two successive interchanges where rehabilitation is hardly conducted. As in Fig.-15, a correlation between the average rut depth and standard deviation is recognized. Namely, the larger the average rut depth becomes, the larger the standard deviation becomes.

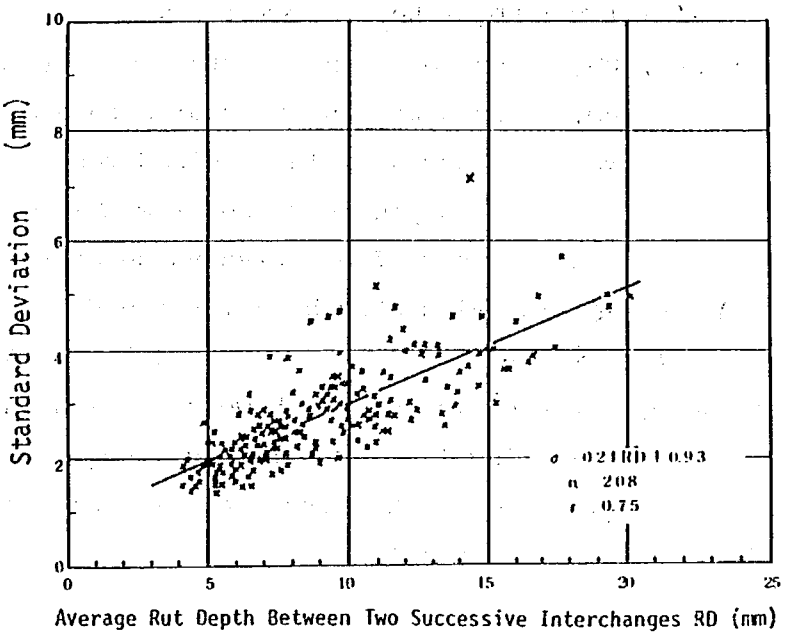


Fig.-15 Average Rut Depth and Standard Deviation

Fig.-16 shows the typical distibution of rut depths between two successive interchanges where rehabilitation is hardly conducted and it is possible to consider that rut depth between two successive interchanges distribute normally.

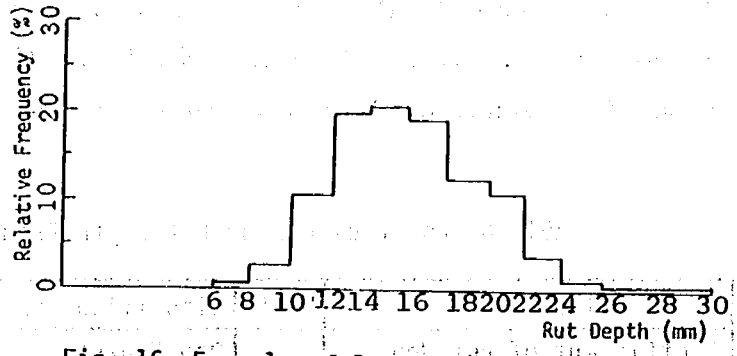


Fig.-16 Example of Rut Depths Distribution

Taking Fig.-15 and Fig.-16 into consideration with the fact that rut depth progresses with increase of traffic, it is considered that the distribution of rut depths will change with the increase of large-sized vehicles as shown in Fig.-17.

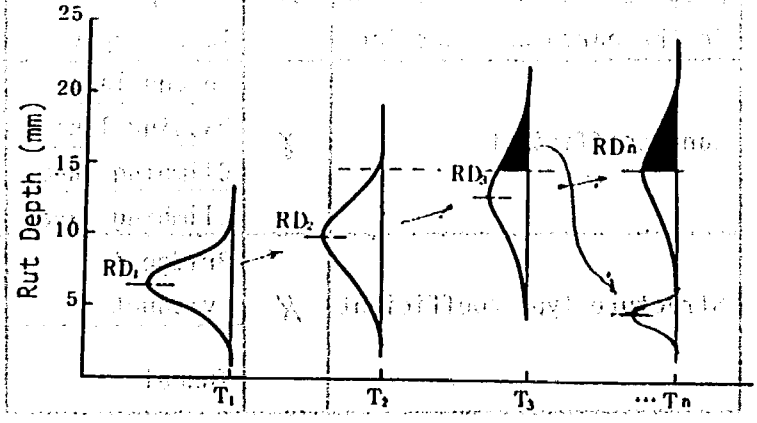


Fig.-17 Progress of Rut Depths Distribution

The macro-method is a method predicting a change of rut depths distribution with the progress of average rut depth by the above-mentioned relation. Procedure of prediction is as follows.

a) Setting initial average rut depth

Since the rut depth is defined as a vertical distance from the highest point to the lowest point within a lane in Nihon Doro Kodan as shown in Fig.-5 in 2-3-1 (1) and some transverse unevenness always occurs on the surface just after paving in the construction stage or the rehabilitation stage, its unevenness is measured as the initial rut depth (RD_0). Fig.-18 and Fig.-19 show examples of distribution of the initial rut depth on the surface just after paving in the construction stage and the rehabilitation stage [3]. The initial rut depth is determined as follows from these data.

In the construction stage RD_0 = 4 mm

In the rehabilitation stage (milling and overlay) RD_0 = 4 mm

In the rehabilitation stage (overlay) RD_0 = 6 mm

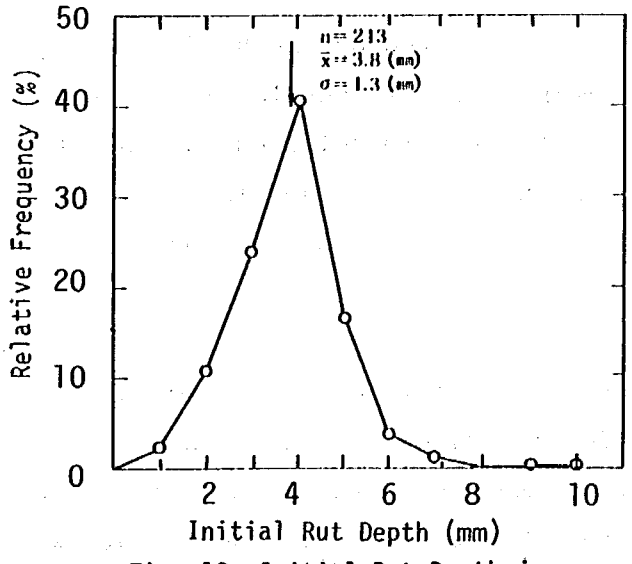


Fig.-18 Initial Rut Depth in the Construction Stage

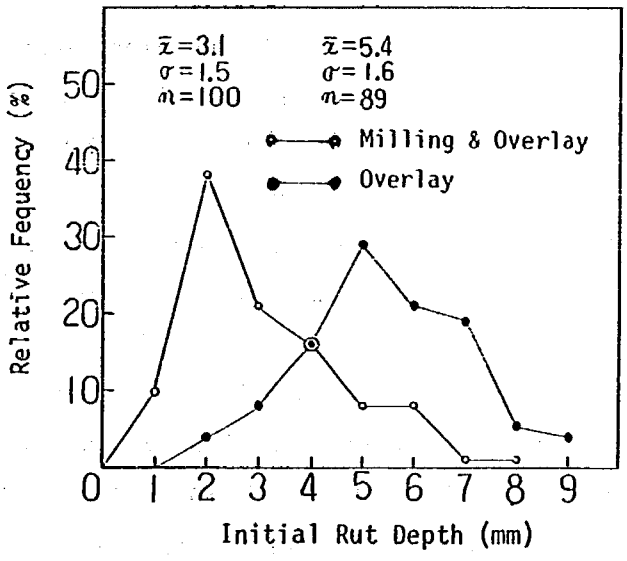


Fig.-19 Initial Rut Depth in the Rehabilitation Stage

b) Prediction of the average rut depth

The average rut depth ($\overline{RD}$) between two successive interchanges is predicted by Eq.-8.

$$\overline{RD} = \overline{RD_0} + \lambda \times T \times t \qquad (8)$$

where, $\overline{RD}$ = predicted average rut depth between two successive interchanges

$\overline{RD_0}$ = initial average rut depth between two successive interchanges

T, t, λ, α, γ, K ---- refer to Eq.-4

c) Calculation of standard deviation of rut depth

Standard deviation (σ) is calculated by Eq.-9 (see Fig.-15).

$$\sigma = 0.21\ \overline{RD} + 0.93 \quad (9)$$

d) Calculation of rehabilitation length

Lane length (L_R) whose rut depths are more than given judgement value (R) is calculated by Eq.-10.

$$L_R = L \times \frac{1}{\sqrt{2\pi}} e^{-\frac{X^2}{2}} \times (0.3194\ Y - 0.3566\ Y^2 + 1.7815\ Y^3 - 1.8213\ Y^4 + 1.3303\ Y^5) \quad (10)$$

where, $X = (R - \overline{RD}) / \sigma$, $Y = 1 / (0.23165\ X + 1)$

L = lane length between given two successive interchanges

4-2 Selection of Rehabilitation Sections

The selection of rehabilitation sections is conducted by the hands at present. The procedure is as follows : Some maintenance target level is set and sections whose rut depths exceed the given target are selected. At this time, if there are not much rut depth data which exceed the target in a section, the section is not selected. Lately, we have computerized the above-mentioned method and the method is described below.

After rut depth is predicted every 100 m by the micro-method, we order a computer to select sections which satisfy the following condition : " Lane length with a rut depth which is of X mm or more occupying Y m at least in a section of Z m long. " For example, for the condition : "Lane length with a rut depth which is of 19 mm or more occupying 300 m at least in a section of 500 m long ", the sections shown in Fig.-20 are selected. In this method, a unit section is 100 m long (there is one datum in 100 m in principle) and until a section which satisfies the given condition is found, the scale of Z m is being shifted 100 m at a time. Hereupon, values of X, Y and Z can

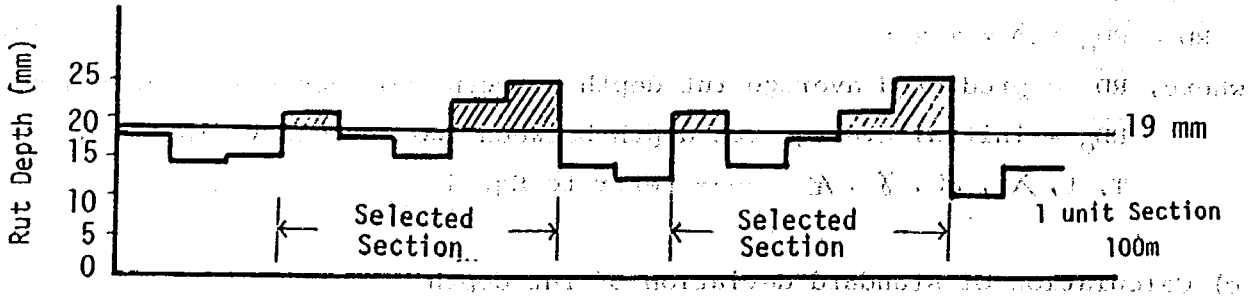


Fig.-20 Example of Selection of Rehabilitation Section

be chosen randomly. In the actual implementation, Z should be decided in considering a feasible day's execution length.

4-3 Prediction for Future Needs of Rehabilitation

Example of prediction for future needs of rehabilitation by the maintenance subsystem on rutting will be described below.

4-3-1 Example on Project Level

(1) Prediction conditions

The prediction conditions are as follows.

a) Objective section : All traffic lanes of the Tomei Expressway (Tokyo - Mikkabi)

b) Period of prediction : F.Y. 1983 - F.Y. 1987 (five years)

c) Basic data : F.Y. 1982

d) Conditions for rehabilitation : The selected sections where lane lengths with a rut depth which is of X mm or more occupying Y m at least in a section of Z m long. X, Y and Z are as follows.

Alternative	X (mm)	Y (m)	Z (m)
1	non-rehabilitation		
2	15	300	500
3	17	300	500
4	19	300	500

e) Initial rut depth after rehabilitation is 4 mm.

(2) Result of prediction

Fig.-21 and Fig.-22 show the typical example of predicted results of the rut depths distribution. Fig.-23 shows the yearly change of the average rut depth and the length of rehabilitation in the past, and the predicted result for the future. In Fig.-23, the average rut depth is the value after rehabilitation. From Fig.-23, the followings become clear : i) If the rehabilitation is not implemented, the average rut depth will increase annually at the rate of 1.3 mm. ii) If we intend to keep the present condition, alternative -2 should be selected.

Though it is predicted that the future rehabilitation length will become

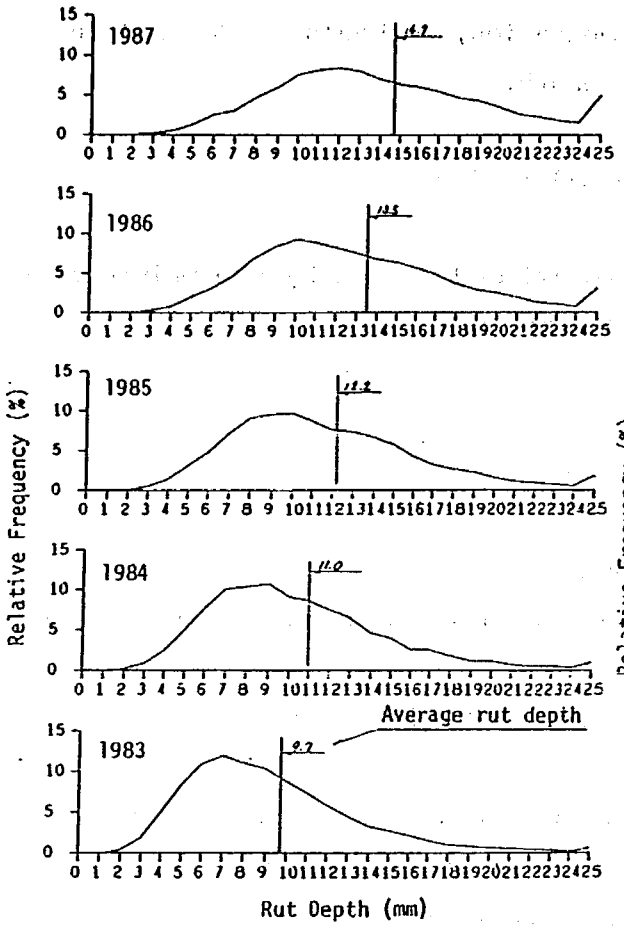


Fig.-21 Predicted Result of Alternative - 1 (non-rehabilitation)

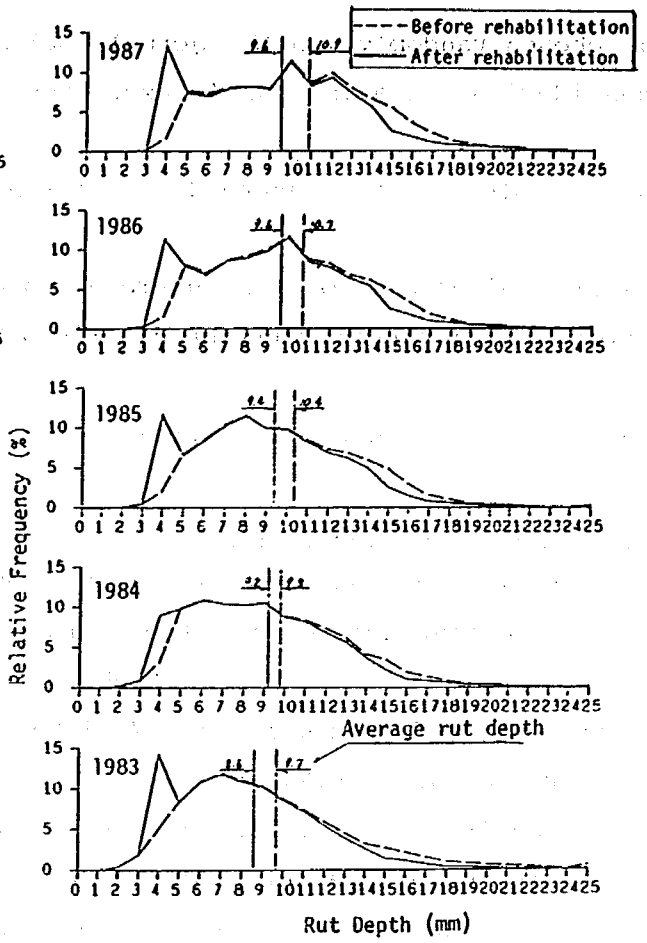


Fig.-22 Predicted Result of Alternative - 2 (X=15mm, Y=300m, Z=500m)

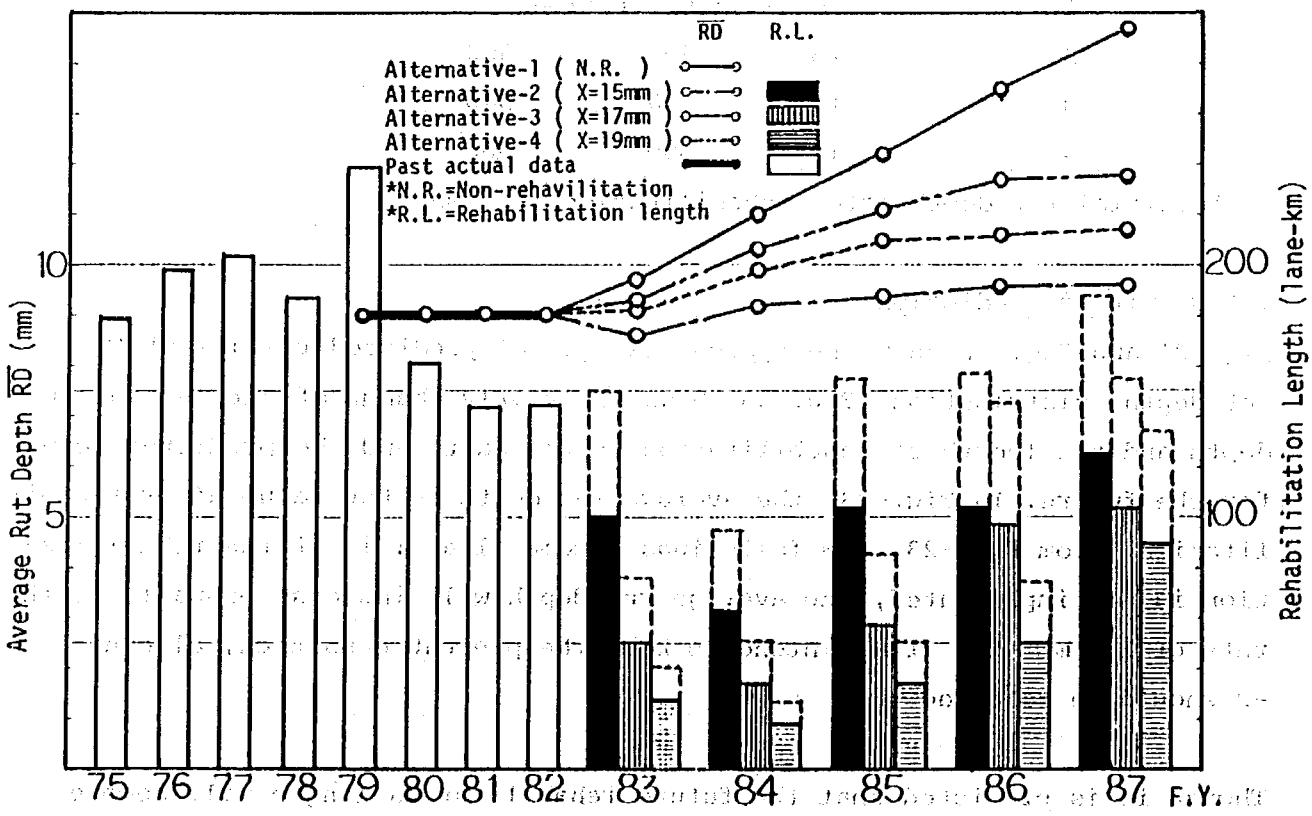


Fig.-23 Example of Predicted Result on Project Level

shorter, it is due to select the rehabilitation sections from a rutting point of view. In the case of the objective section of Tomei Expressway, the rate of lane length rehabilitated for rutting to the rehabilitation length was approximately two thirds and for cracking and the other was approximately one third. Since lane lengths which should be rehabilitated for cracking and the other are not included in the predicted result, future needs of rehabilitation may become the length shown by broken line in Fig.-23.

4-3-2 Example on Network Level

Fig.-24 shows the predicted result of the future needs of pavement rehabilitation on expressways in Japan. If there are measured rut depth data, future rut depths are predicted by the micro-method based on the data in F.Y. 1980. The other is predicted by the macro-method. From the result, the predicted rehabilitation length in 1987 will be 3.5 times longer than that in 1982. It is considered that the increase of rehabilitation length may be due to a rapid increase of old expressways and short-lived expressways by abrasion in the cold areas.

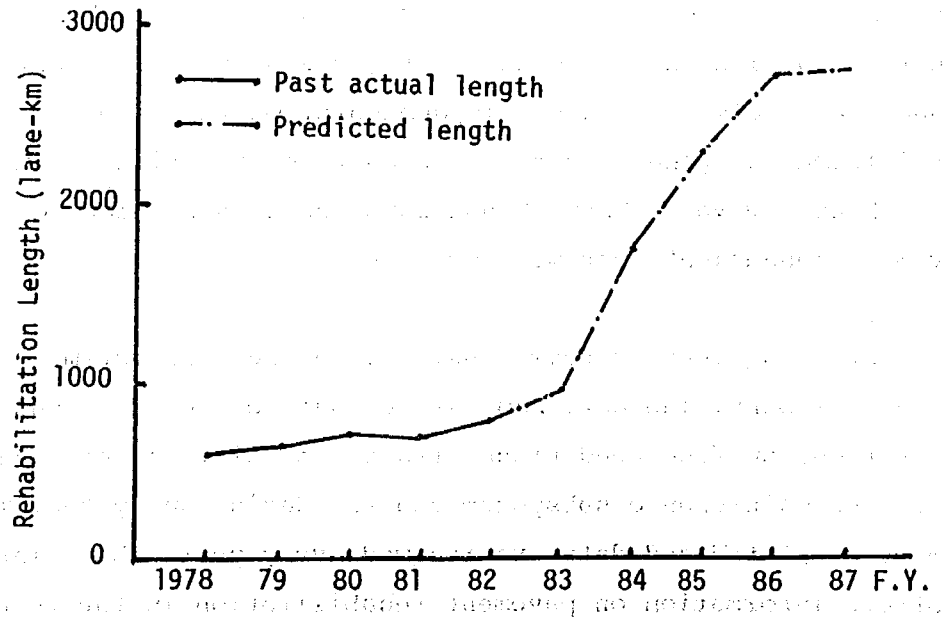


Fig.-24 Example of Predicted Result on Network Level

5. SUMMARY

So far, the outline of the pavement management systems in Nihon Doro Kodan

has been described and the summary is as follows.

(1) Pavement technology is divided into two groups in Nihon Doro Kodan : the construction stage and the operation stage. Technical standards on investigation, design, execution, quality control and testing in these stages are systematized in the various manuals.

(2) Evaluation of pavement conditions is the most fundamental activity for the reasonable and systematic maintenance of pavement. For evaluation of localized cracking, faulting, pot holes and longitudinal roughness, patrolling or visual inspection is the practical method used in Nihon Doro Kodan. On the other hand, for evaluation of rutting, cracking and skid resistance, special vehicles with high travelling speed aiming at quantitative and automatic measurement are used. In order to judge whether maintenance or rehabilitation is necessary, the target maintenance levels are set for respective surface conditions. If different types and degree of distresses exist on road surface, a synthetic evaluation is made by summing up each grade corresponding to the pavement value.

(3) Since pavement maintenance cost is increasing rapidly with deterioration and increase of length in operation of expressways hereafter, more efficient and reasonable maintenance is demanded. On the other hand, amount of pavement information generated from the past construction and maintenance activities is becoming vast. Then, Nihon Doro Kodan is developing the computerized pavement management systems since 1980.

(4) The pavement management systems consist of design subsystem, maintenance subsystem and data bank. The data bank stores data on design, construction and rehabilitation, surface conditions, traffic conditions, etc. and offers these data to the maintenance subsystem and the design subsystem. The data bank is almost completed and data are renewed every year. The maintenance subsystem offers information on pavement rehabilitation in the future in order to invest rehabilitation budget efficiently and to make effective and reasonable rehabilitation programs. Analysis programs of surface conditions and rehabilitation, prediction methods of future rut depth and selection system of rehabilitation sections for rutting have been developed. The design subsystem offers information on thickness design of asphalt pavement

in construction stage. The thickness design program of asphalt pavement based on the Design Manual of Nihon Doro Kodan has been developed.

(5) A prediction method of rut depth progress by large-sized vehicle traffic volume has been developed. Prediction methods are divided into the micro-method and the macro-method. The micro-method is a method predicting the rut depth based on the measured values. On the other hand, the macro-method is a method predicting the progress of rut depths distribution which is regarded as a normal distribution. Future needs of pavement rehabilitation for rutting on project level or network level can be predicted by these methods.

As mentioned above, a part of the pavement management systems are already operative. Though there are some problems such as establishment of implementation system, we intend to expand and improve the pavement management systems while introducing the systems to the actual daily activities.

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Pavement Maintenance Management, Materials and Procedures for Urban Road Systems

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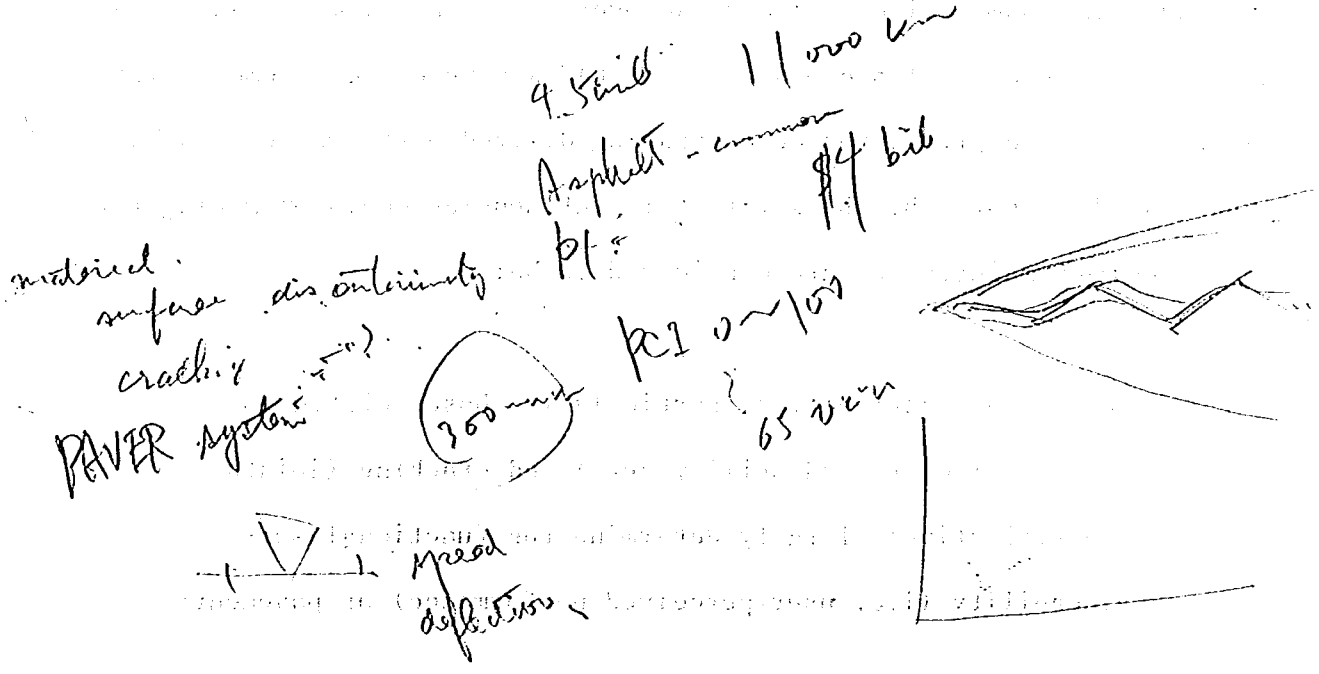
ABSTRACT

John J. Emery

This contribution integrates consulting experience with urban road systems maintenance problems in Southern Ontario in terms of pavement maintenance management (APWA PAVER-based approach), current materials and procedures that are giving desired performance (improved drainage, modification to utility cut restorations, systematic crack sealing, etc.), new approaches with promise (geotextiles based on fiberglass to reduce reflection cracking, high stability steel slag asphaltic concrete to reduce milling requirements, etc.) and new testing methods (falling weight deflectometer, for instance). The importance of minimizing the funding impact of competing demands for government revenues ("infrastructure crisis" of United States that is not as acute in Canada presently) on existing urban road systems, i.e. economics, is emphasized throughout. For this reason, a systematic approach to determining current pavement conditions on a network and project level, and determining cost effective maintenance and rehabilitation strategies, are given in detail. From the available systems, PAVER has been selected based on its APWA backing and applicability to all classes of urban pavements (flexible, rigid and composite) and associated features, recognizing that the key is adoption of a systematic approach and several are available to be used in conjunction with local experience and input. General findings with implementation in a large urban area are given with extension to required procedures and materials, with desired short and long-term performance, to carry out the preferred maintenance and/or rehabilitation strategies identified. Areas of particular concern have been drainage improvements, crack sealing materials and procedures including inspection, and the impact of utility cuts.



PAVEMENT MAINTENANCE MANAGEMENT, MATERIALS AND PROCEDURES FOR URBAN ROAD SYSTEMS

by

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Toronto, Ontario, Canada

"Horrendous costs will face taxpayers for road and bridge repairs and replacement if the severe reduction in spending of the past decade is allowed to continue." TRIP Canada, 1983 RTAC Conference [1]

While increased expenditures will do much to restore and expand the urban pavement infrastructure, a systematic approach to optimizing the use of maintenance funding will also have a significant impact, particularly given the competing demands for government revenues. From applied experience for urban road systems in Southern Ontario, with its wide range of pavement types and environmental conditions, an overall urban pavement management approach has been developed based on the PAVER pavement management system and materials and procedures demonstrating desired performance. As background to this approach, the key factors influencing decision making for urban pavement maintenance are considered to be:

- surface discontinuities, particularly those related to services (covers and utility cuts) and cracking (joints and reflective), largely determine the functional serviceability (i.e. user-perceived performance) of pavements

- water action is the prime cause of structural integrity loss (i.e. load carrying performance), indicating the im-

portance of minimizing water ingress through surface discontinuities and providing effective drainage systems for pavement structures

- with the still escalating, energy-related price of many pavement maintenance materials (asphalt cement price increase of about twenty percent this paving season for instance), they must be used cost-effectively and/or high performance alternatives developed

- systematic and timely maintenance-restoration-rehabilitation-resurfacing can protect the urban pavement investment (about $50.00/m²) and minimize the on-going high cost of reconstruction (resurfacing at about $5.00/m² on a 15 to 20 year cycle rather than reconstruction at about $40.00/m² for flexible pavements, for instance)

- pavement management systems in the urban context must be able to incorporate surface discontinuities (type, severity and density) and handle a full range of pavement types (flexible, rigid and composite)

- systematic pavement management will have a catch-up period if there has been under-funding of maintenance and/or emphasis on capital projects.

As a first step in establishing overall urban pavement maintenance management, a systematic approach to determining current pavement conditions on

a network and project level, and developing cost-effective maintenance and rehabilitation strategies, is required. From the available pavement management systems, PAVER has been adopted given its American Public Works Association (APWA) support and applicability to all types of urban pavements and associated features. However, it is recognized that the key factor is the adoption of a systematic approach and several are available to be used in conjunction with local experience and input.

APWA PAVER PAVEMENT MANAGEMENT SYSTEM

APWA PAVER is a pavement management system developed some ten years ago by the U.S. Army Corps of Engineers Construction Engineering Research Laboratory (CERL), and recently adopted and supported through the APWA for city and region use on a low-cost, time-sharing basis [2,3]. Currently, over 30 agencies in Canada and the United States are using PAVER, including Niagara Region, Hamilton-Wentworth Region, Hamilton, Winnipeg and Calgary. The system consists of a computerized data base and a number of programs that store, retrieve and manipulate the data as well as perform analyses required for network, project and project management-level decisions. The data base includes distress, deflection, ride quality and skid resistance data for each section of pavement as well as data on secondary structures, drainage, traffic, pavement structure, maintenance history, cost records, maintenance policy, etc. New types of data and streets can be added on an incremental basis. Distress information collected by trained inspectors on a sampling basis (Pavement Condition Index) forms the basis for ranking, budgeting and forecasting. An organization's data base is maintained in-house with the analyses completed interactively with the data base manager (Boeing Computer Services time-sharing).

Pavement Condition Index (PCI)

The key component of the PAVER system is the pavement condition survey and rating procedures which are used to derive the Pavement Condition Index (PCI). Data obtained from these surveys form the basis for determining pavement conditions, maintenance and rehabilitation needs, priorities and budget forecasts. For purposes of definition, an urban pavement network is subdivided as follows:

- o branch - any identifiable part of the network which is a single entity and has a distinct function, normally an individual street (arterial, collector or local)

- o section (feature) - a division of a branch (street) which has common characteristics such as pavement type, surface width, structure, construction and maintenance history, drainage, traffic and general condition

- o sample unit - an inspection sub-division of a section. Guidelines for dividing sections into sample units (smallest unit of network), and the percent sample units per section to be inspected for a statistically valid section PCI determination, are given in PAVER manuals [2]. For asphaltic concrete (AC), composite (APC) and surface treated (ST) pavements, a sample

unit is an area of approximately 250 m², for instance.

A number of factors influence a pavement's functional and structural performance and condition such as traffic, structural capacity, structural integrity, roughness, skid resistance, rate of deterioration and maintenance requirements. In order to accurately represent the pavement's overall condition, any indexing system must consider all of these factors. Direct measurement of all of these condition "indicators" requires extensive equipment and personnel capabilities. However, indirect measurement based on observable distress as adopted with PAVER is very effective in estimating the ramifications of the factors influencing pavement performance as shown in Tables 1 and 2 for flexible and rigid pavements in terms of traffic and environment/materials.

TABLE 1 CATEGORIES OF FLEXIBLE PAVEMENT DISTRESS [2]

Observable Distress Type	Primarily Traffic Loading Related	Primarily Environment/ Materials Related
Alligator or Fatigue Cracking	X	
Bleeding		X
Block Cracking		X
Corrugation		X
Depression		X
Joint Reflection Cracking from PCC		X
Lane/Shoulder Dropoff or Heave		X
Lane/Shoulder Separation		X
Longitudinal and Transverse Cracking		X
Patch Deterioration	X	
Polished Aggregate	X	
Potholes	X	
Pumping and Water Expulsion	X (M,H)	X (L)
Ravelling and Weathering		X
Rutting	X	
Slippage Cracking		X
Swelling		X

TABLE 2 CATEGORIES OF RIGID PAVEMENT DISTRESS [2]
(JOINTED PC CONCRETE)

Observable Distress Type	Primarily Traffic Loading Related	Primarily Environment/ Materials Related
Aggregate Durability Cracking/Distress		X
Blow Up		X
Corner Break	X	
Depression		X
Faulting of Transverse Joints and Cracks	X	
Joint Load Transfer System Deterioration	X	X
Joint Seal Damage at Transverse Joints		X
Lane/Shoulder Dropoff or Heave		X
Lane/Shoulder Joint Separation		X
Longitudinal Cracking		X
Longitudinal Joint Faulting	X	X
Patch Deterioration	X (M,H)	X (L)
Patch Adjacent Slab Deterioration	X	X
Popouts		X
Pumping and Water Expulsion	X (M,H)	X (L)
Scaling, Map Cracking and Crazing		X
Spalling (Transverse and Longitudinal Joints)	X (M,H)	X (L,M.H)
Spalling (Corner)		X
Swell		X
Transverse and Diagonal Cracking	X (M,H,L)	X (L)

L - Low Severity M - Medium Severity H - High Severity

The degree of pavement deterioration for each observable distress type is a function of specific distress type, level of severity (low, medium and high in terms of crack width, rut depth, etc.) and density (percentage of pavement area affected). These are combined through deduct values in terms of severity and density in the PCI approach to give a composite numerical rating (scale from 0 (failed) to 100 (excellent))for the pavement sample unit, that agrees closely with the collective judgement of experienced pavement

engineers. From field information on observable distress types, collected in accordance with detailed inspection manuals on standard forms, the computerized PAVER system determines the PCI automatically for each sample unit (sampling rate of approximately 15 percent) and section of the network as shown schematically in Figure 1. The other primary information for implementation of PAVER on a network level is the last surface/overlay construction date.

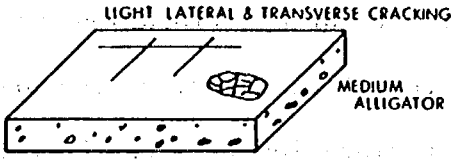
In addition to the primary data in terms of PCI and construction date for implementation of PAVER, supplemental information is incorporated such as: traffic; pavement structure and history; roughness in terms of Ride Comfort Index; skid resistance problems; Road Needs Study data which includes factors such as geometry and capacity; and general serviceability information such as zoning (residential, commercial, central business district, industrial), curb and gutter/ditching condition, bus route, railway crossings, manhole/catchbasin conditions and height, utility cut restorations, previous maintenance (crack sealing, square patching, slurry sealing, etc.) and probable maintenance requirements. Deflection testing (falling weight deflectometer) is generally reserved for project level work identified from network level implementation and updates.

General PAVER Capabilities

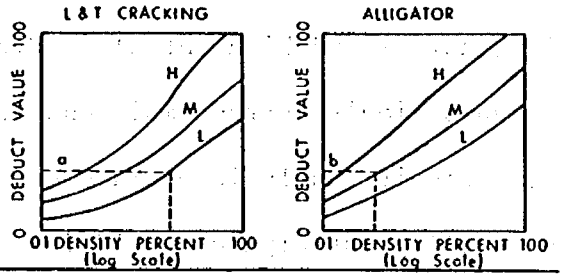
The general PAVER data analysis and management capabilities include: interactive data storage and retrieval (System 2000 data base manager); network definition and inventory reports; PCI reports; section or project prioritization based on pavement type, pavement class, traffic, PCI range or a combination of factors, to meet the organization's policy; inspection

STEP 1. DIVIDE PAVEMENT FEATURE INTO SAMPLE UNITS

STEP 2. INSPECT SAMPLE UNITS; DETERMINE DISTRESS TYPES AND SEVERITY LEVELS AND MEASURE DENSITY

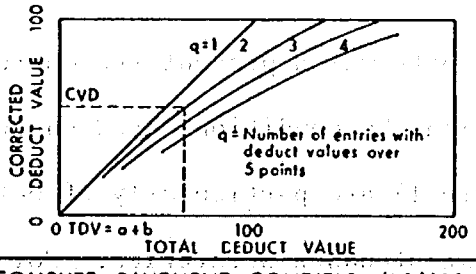


STEP 3. DETERMINE DEDUCT VALUES



STEP 4. COMPUTE TOTAL DEDUCT VALUE (TDV) a+b

STEP 5. ADJUST TOTAL DEDUCT VALUE



STEP 6. COMPUTE PAVEMENT CONDITION (PCI) 100-CDV FOR EACH SAMPLE UNIT INSPECTED

STEP 7. COMPUTE PCI OF ENTIRE FEATURE (AVERAGE PCI's OF SAMPLE UNITS)

STEP 8. DETERMINE PAVEMENT CONDITION RATING

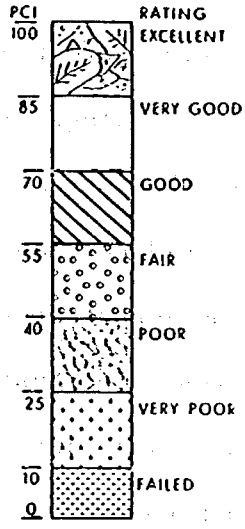


FIGURE 1 PROCEDURES FOR DETERMINING PCI OF A SAMPLE UNIT AND SECTION (FEATURE)

scheduling to efficiently maintain current condition data (criteria of minimum PCI reached and rate of deterioration); determination of maintenance and repair needs (M and R); performance of economic analyses; and budget planning. Predictions during implementation of PAVER are based on linear extrapolations as only the surface/overlay date and inspection date are known. As such, the actual rate of deterioration may vary and subsequent inspections are required to refine deterioration predictions, and hence budget planning. This deterioration rate can be compared with typical overall urban pavement experience, such as rate of deterioration curves for rigid pavements (low, normal and high rates) in Figure 2, to determine the impact of the organization's current maintenance materials and procedures. For instance, a significantly lower rate of deterioration (PCI decrease with time) than normally anticipated would indicate effective standard maintenance has been applied for the network.

Pavement condition frequency reports which present the current and predicted distribution of the sections by condition based on PCI rating (failed through excellent, Figure 1) are particularly effective in demonstrating the impact of various maintenance strategies. For instance, Figure 3 shows the 1984 condition and predicted 1989 condition, based on current rate of deterioration (i.e. no changes to maintenance policy), for an urban pavement network. Such information assists the pavement engineer in developing maintenance strategies and obtaining resources to meet performance objectives through interaction with engineering and funding committees (typically elected officials).

The determination of M and R needs for each pavement section is based on a first-level decision based on PCI value, types of distress and deteriora-

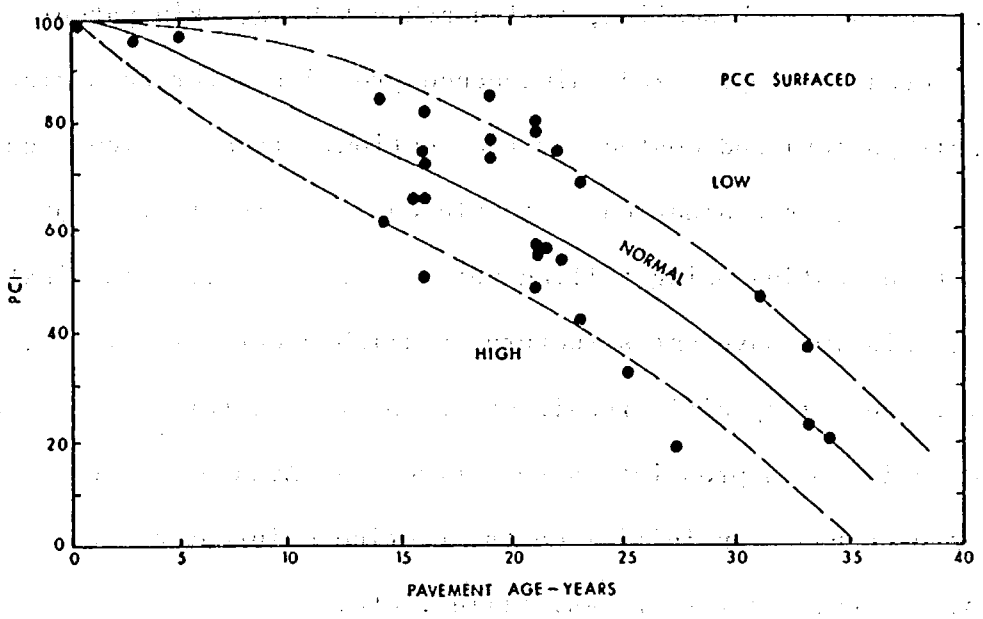


FIGURE 2 RATE OF DETERIORATION OF RIGID PAVEMENT (PCI DECREASE WITH TIME SINCE CONSTRUCTION)

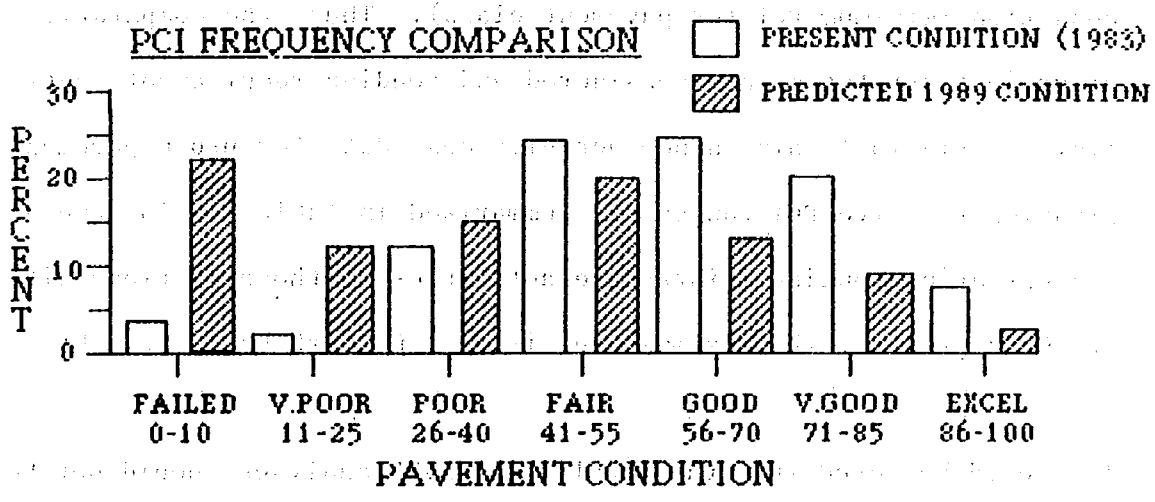


FIGURE 3 TYPICAL CONDITION FREQUENCY REPORT FOR URBAN PAVEMENT NETWORK

tion rate (ranking and sorting). If a pavement section does not require detailed analysis (project level treatment) at present, routine maintenance practices such as crack sealing, full-depth patching, skin patching, etc. can be selected from optimized maintenance procedures for each distress type (repair policy) and costed for the section. If a section requires further analysis, an evaluation is completed in terms of structural capacity (deflection testing using falling weight deflectometer, for instance), roughness, existing pavement structure (available data or coring), drainage, etc. to select specific repair or reconstruct alternatives. To help in the selection of appropriate alternatives, a PAVER economic analysis program is used based on initial costs, routine maintenance costs, major maintenance costs, interest rates, timing, etc.

Finally, a budget planning report provides an estimate of the funding required over a given period to achieve a desired level of pavement network condition (minimum PCI for pavement class). Thus, the comparative costs of various strategies are considered and funding requirements anticipated. Typical principal maintenance activity cost data for urban pavement budget planning for five PCI ranges are summarized in Table 3. In general, by concentrating routine maintenance activities on those sections with a PCI of 60 or greater, the maximum benefit on a network basis is achieved.

It should be noted that the results of PAVER analyses should not be used as a rigid basis for the selection of the "best" M and R alternative, but rather as an aid to the pavement engineer in making an optimum selection, as other factors such as local experience and funding constraints may be involved.

TABLE 3 TYPICAL COST DATA FOR VARIOUS PCI RANGES

PCI Range	Principal Maintenance Activity	Approximate Cost/m² Flexible	Composite
0-20	reconstruct, including curb and gutter	43.00	50.00
20-40	repair and 75 mm overlay, including 40 mm of surface milling	11.70	14.30
40-60	maintenance and 40 mm overlay	5.40	5.40
60-80	routine maintenance	0.40	0.90
80-100	crack sealing	0.10	0.40

EVALUATION OF PAVEMENT STRUCTURAL CAPACITY

For project level work, it is important to evaluate the structural capacity of the pavement to ensure satisfactory performance of any maintenance-restoration-rehabilitation-resurfacing selected. The impulsive non-destructive, deflection testing device that has been adopted for evaluating load-carrying capacity is the falling weight deflectometer (FWD) shown schematically in Figure 4 (Dynatest 8000 FWD), noting that other methods are available. The FWD trailer unit is capable of exerting a load impulse similar in magnitude and duration to moving truck wheel loads. The unit can produce loads from 6600 N to 106,000 N with a duration of approximately 27 μsec. Impulsive load is applied to the loading plate by dropping a weight package on a dampening system with the resulting pavement deflection measured by seven seismic deflection transducers spaced at predetermined intervals from the loading plate. Signals from the load cell in the loading plate and deflection transducers are fed into the system processor which selects the peak values and transfers this information to a Hewlett Packard 85 computer in the tow van. This HP-85 monitors and controls the complete operations of the FWD and the data can be stored on magnetic tape for later processing. A normal sequence is approximately two minutes long.

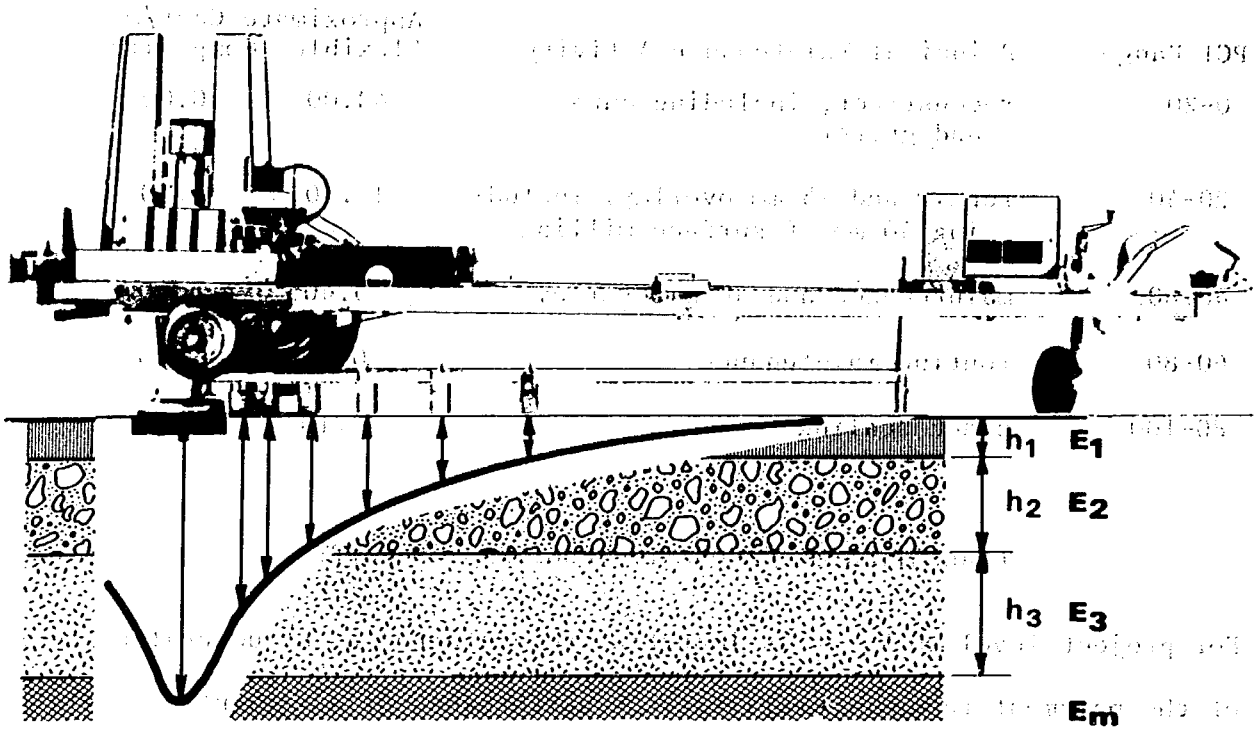


FIGURE 4 DYNATEST 8000 FALLING WEIGHT DEFLECTOMETER USED BY TROW LTD

The FWD data is used routinely for the following: determining uniformity of the pavement section; determining required overlay thickness to carry future design loadings; locating deteriorated concrete; locating voids under slabs and determining their extent; determining load transfer across joints and identifying joints where remedial action is necessary; estimating future life of a pavement in terms of load applications where existing distress is considered; and supplementing PCI information when determining M and R alternatives. In addition, the FWD data can be used along with computerized mechanistic models and analyses to determine factors such as: modulus of subgrade reaction; moduli of pavement layers; critical deflections; critical edge and corner stresses; percent life consumed; load carrying capacity; and subgrade strains.

MATERIALS AND PROCEDURES

Free Draining Subbase

As surface discontinuities (services and cracking) largely determine the functional serviceability of urban pavements, materials and procedures to reduce their impact have received considerable attention. However, it is also recognized that pavements require economical, effective and long lasting drainage systems to maintain load-carrying capacity. In addition to the wide-range of new drainage products (geotextiles for filters, plastic perforated pipe, etc.), attention has been paid to developing a free-draining modified open graded subbase. As the provision of drainage in the pavement structure is critical, and dense graded granular base/subbase is not free draining, it is necessary to incorporate an open graded subbase or some other drainage system in satisfactory pavement designs. Unfortunately, most open graded granular materials are very low in stability and this has resulted in both construction and infiltration problems unless geotextile filters are also incorporated. Recently, Penn DOT has developed and proven a new specification for open graded subbase that provides a sensible compromise between drainability and stability with construction equipment. This gradation, with slight modification, is given in Table 4.

TABLE 4 OPEN GRADED SUBBASE

Sieve Size	Percent Passing	
	Penn DOT OGS	Modified OGS
50 mm	100	100
19 mm	52-100	52-100
9.5 mm	36-65	30-65
4.75 mm	8-40	8-40
1.18 mm	0-12	0-12
600 µm	0-8	0-8
75 µm	0-5	0-5

The modified OGS is required to have 100 percent crushed particles, be well graded and to have a coefficient of uniformity ≥ 4. For fine grained subgrades, a dense graded subbase layer must be placed first to prevent fines intrusion and the modified OGS of 100 to 150 mm thickness is generally placed immediately below the base. This OGS layer is effective in both draining the pavement structure and acting as a moisture cutoff for frost action.

Utility Cuts

The general impact of utility cut restorations on urban pavement systems has recently been determined for Metropolitan Toronto (Metro) during a study involving: problems and potential for improvement; impact of functional serviceability impairment; overall cost benefits through specification changes; and implementation. The overall significance of the subject to Metro and other municipalities can be seen when randomized inspections show a mean utility cut density (cut patch area to pavement section surface area) of 3.5 percent, with a standard deviation of 6.7 percent for Metro. The general problems associated with utility cut restorations - backfill settlement, pavement damage/deterioration, joint/crack opening/deterioration, protracted maintenance, etc. - are relevant to flexible, rigid and composite pavements. For composite pavements, all of the utility cut problems affecting AC and PCC pavements show up, particularly where load transfer decreases and/or there is rocking/cracking of slabs due to voids.

To quantify their current impact, 43 randomly selected, recent utility cut restorations in Metro were inspected for condition, with deflections measured using the falling weight deflectometer to determine the presence

of discontinuities (deflections, uniformity across cuts and load transfer across cuts) and voids. Most restorations had discontinuities, but few voids were detected. Only 49 percent of the utility cut restorations had a rating of 9 or greater out of 12, considered the minimum level to not decrease the functional serviceability of the pavement section. When the qualitative rating was adjusted by removing surface condition, 65 percent of the restorations had ratings of less than 7 out of 9, considered to be the minimum level to not decrease the functional serviceability. Clearly the utility portion of restorations is "building in" problems with a significant maintenance cost increase. It was recommended that unshrinkable fill be adopted in the utility cut restoration specification, as indicated in Table 5, to resolve current problems with backfill quality and compaction procedures. Implementation is in progress.

TABLE 5 GUIDE SPECIFICATION FOR UTILITY CUT RESTORATIONS

To ensure utility cuts do not impair condition/use of roadways:

- cut edge of excavation in a straight line
- keep within specified limits of utility cut
- avoid undermining of adjacent roadway
  - bracing, shoring and/or sheeting as necessary
  - excavation support system removed as backfilling proceeds
- remove unsuitable soil from bottom of utility cut
- support/restore all services encountered
- remove excavated soil from site
- keep utility cut free of water
- bedding to utility/agency specification
- backfill using unshrinkable fill

- to within 50 mm of pavement surface
- protect until set (24 hours)
- place 50 mm HL-3 hot mix

o desired features of unshrinkable fill (or equivalent material)
- reasonable cost
- desired strength development
- readily excavated following restoration
- widely available
- standard material without need for inspection
- demonstrate use

o unshrinkable fill specification
- 25 kg/m³ cement
- concrete aggregates
- 0.07 MPa (10 psi) within 24 hours
- not to exceed 0.4 MPa (60 psi) at 28 days
- AEA to reduce segregation

o it will be necessary to cut back any undermined areas and remove loose material during backfilling

o granular backfill can be used with an exemption
- suitable soil conditions, sufficient space, inspection
- thin lifts, 95% modified (100% standard) compaction

Fiberglass Reinforcement

The use of geotextiles to reduce reflection cracking of overlays is growing, and the development of fiberglass reinforcement for this purpose is particularly promising. Bay Mills Limited of Ontario has recently developed a fiberglass reinforcing mesh for use in strengthening paved surfaces. Termed Glass Road-Grid (patent pending), this product consists of a semi-rigid, resin impregnated, continuous filament, fiberglass reinforcement which can be incorporated into paved roadways to provide increased strength and resistance to cracking. This reinforcement is easily rolled out on a pavement surface which has been sprayed with tack coat, and is subsequently overlaid with asphaltic concrete. The large grid openings permit the asphaltic concrete overlay to encapsulate each strand of mesh, permitting intimate contact and bonding with the underlying layer. The product has a

high modulus and high strength to cost ratio, its coefficient of expansion approximates that of road construction materials and it resists corrosion by materials used in road construction or found in the road environment, such as deicing salt.

Trial sections of fiberglass-reinforced pavements were constructed in 1983 in Kitchener, Burlington, Hamilton-Wentworth and Metro.

An on-going study of these sections has confirmed significantly increased resistance to reflection cracking compared to adjacent identically constructed but unreinforced sections. Quantitative assessment of structural improvements attributable to the Glass Road-Grid will be performed in 1984 using the falling weight deflectometer.

OTHER DEVELOPMENTS

A number of other materials and procedures for urban pavement maintenance are being used, or assessed for future use, such as: high stability steel slag asphaltic concrete overlays to reduce milling requirements (reduce thickness to about 75 percent of conventional asphaltic concrete); systematic crack sealing with detailed materials, placement and inspection specifications to ensure a six to eight year performance (considered the most cost-effective maintenance procedure for pavements with PCI greater than about 60); heater/scarifying of asphaltic concrete surfacings to remove existing crack patterns and wheel ruts, including plant for incorporating a thin overlay; patch master for repairing small areas of asphaltic concrete and particularly utility cut problems; etc. With the support of the various organizations involved, it is anticipated that details on these other

materials and procedures will be developed for assessment and general use by urban pavement engineers. To make progress with the pavement infrastructure "crisis" it is critical that existing and innovative pavement management, materials and procedures technology be applied.

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Evaluation of the Effect of Winter on the Degradation of Roads and Streets

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ABSTRACT

In regions where pavement riding quality may be affected by winter, it is necessary to integrate in a rational pavement maintenance-management system a sub-system to minimize the effect of winter on the roughness of pavements. This article presents such a sub-system. The sub-system identifies winter-affected pavements by measuring changes in roughness and degradation, classifying them according to severity standards and establishing priorities according to road conditions and according to the effect of winter on roads.

The development of such a system requires the identification of factors influencing the measurement of roughness, the evaluation of the effect of low temperature on Mays ride meters and the determination of the normal overall variation between test runs in rural and urban pavements.

The sub-system is currently used for pavement maintenance management in Quebec (PEP II) and also for a study of causes of degradation.

Experience shows that the sub-system is efficient in detecting road sections that need to be protected against premature deterioration during winter. In-depth analysis of data generated from the sub-system show that the degradation in the roughness during the winter is influenced by such factors as frost heaving at culverts, heaving and settlement at cracks, and also the amount of cracking in a pavement.

EVALUATION OF THE EFFECT OF WINTER ON THE DEGRADATION OF ROADS AND STREETS

by

J. Hode Keyser
Ecole Polytechnique, University of Montreal and Construction Control and Research Center

1. Introduction

In northern and sub-artic countries, no one ignores the disastrous effect that winter exercises on road networks. That, at least, is the reason often given by road engineers to explain the poor performance of many pavements; the effect of winter on pavement performance can generally be measured either by the change in pavement roughness caused by heaving or settlement, or by the increase in surface defects during successive years.

Up to now, little attention has been given to examining the effect of winter on pavement maintenance, even though enormous progress has been realized in the field of pavement maintenance-management; a near total absence of articles on the subject [2] clearly indicates the need to develop pavement management sub-systems to minimize the effect of winter. The development of such systems is important, chiefly because it will: (a) allow design engineers to verify existing design methods, and (b) enable maintenance engineers to make a rational choice of preventive maintenance measures which could possibly alleviate the cause of deterioration, or at least slow down the rate of deterioration.

This article describes the management sub-system developed by the Construction Control and Research Center and which is currently used by Quebec Provincial Highways Authorities (Office des Autoroutes du Québec and Société de Développement de la Baie James) and also by many Quebec Municipalities.

2. Concept of the sub-system

The purpose of the sub-system is to detect, through pavement roughness measurements and condition survey, the degree of deterioration in the riding quality and the degradation of pavement during the winter and to use the results to establish maintenance needs and priorities.

The general framework of the sub-system is illustrated in Fig. 2 and consists of the following steps:

(a) Measure riding quality during the frost-free season; preferably late in the fall but before freezing;

(b) Measure riding quality during the following winter; preferably when the effect of frost on the riding quality of the pavement is most severe;
(c) Correct the results of winter riding-quality measurements to account for the testing temperature, if at the time of the test it was below 10°C;
(d) Calculate the difference between riding quality measured during the frost-free season and the riding quality measured during the winter and corrected for temperature;
(e) Establish standards on the severity of degradation, based on the significance of differences in pavement roughness, taking into account the overall normal variation of test. Classify pavement sections according to these standards;
(f) Classify pavement sections according to the riding quality in the winter;
(g) Evaluate the pavement condition during winter;
(h) Determine the appropriate maintenance needs and priorities;
(i) Integrate the results from the sub-system with other elements of the pavement maintenance management system to establish a maintenance program.

To assure the reliability of the sub-system, it has been necessary to develop:

- Precision criteria and calibration methods for the test vehicle.
- Correction factors for the test vehicle, to account for low testing temperatures.
- Information on factors influencing test results and the overall standard deviation of the tests which includes the run-to-run variation.

3. Testing equipment

A second generation Mays Ride Meter was selected to measure the riding quality of pavement, principally because of local experience gained with the equipment [3] from the evaluation of many thousands of kilometers of urban and rural roads in Quebec. Alternate roughness measurement equipment exists and can be used, especially when more detailed or precise information is desired.

4. Factors influencing results or riding quality

Factors that influence the results and the relative importance of riding-quality measurements have been identified in a literature review [1] and are summarized in Table 1, along with their possible effect on the test

results and the measures that can be taken to control them. Of these factors, only the effect of temperature and the effect of variations within wheel path from run to run are unknown, all other factors can easily be controlled.

Table 1

Factors influencing riding quality measurements using ride meters

Factors	Possible influences on results	Required control
Automobile		
Vehicle	Amplitude and the frequency of vertical oscillations depend on the type and state of the vehicle, in particular the suspension	Use the same vehicle Periodically calibrate the vehicle Make sure that the suspension is in order
Tires	Low pressure in tires underestimates the results	Balance tires Inflate tires to constant pressure
Weight of automobile	Car weight accentuates the deformations	Use same operator Ensure that the quality of gas is approximately the same
Operation		
Speed	Increase in speed increases the amplitude of measured deformation	Operate vehicle at specified speed
Acceleration and braking	Accelerating and braking indicate faults that in reality do not exist on the pavement	Avoid these operations if possible, if not make sure that they are smooth and regular
Wheel path	Changing conditions across the wheel path gives way to more or less important differences in the results	Ensure that vehicle rides as much as possible in the center of the wheel-path
Exterior		
Temperature	Temperature modifies the oscillation of the car and the rigidity of the suspension system	Operate road meter when the temperature is above -10°C or correct the results for temperature
Wind speed	Wind speed affects results when it is greater than 24 km/h	Do not measure riding quality under rain, snow or windy conditions
Condition of surface	Snow or ice or mud on the pavement can significantly affect the results	Do not measure riding quality when there is snow or ice

4.1 Influence of test temperature

Low winter temperatures influence the suspension systems of test vehicles and thus the result of riding-quality measurement. For this reason,

Hughes and Patrick [5] recommend the use of a road meter only when the ambient temperature is above -10°C, while Chong and Phang [6] recommend -12°C as a benchmark.

The effect of winter on pavement riding-quality is determined by measuring the Mays ride value during the winter or early spring, when the temperature is often below -10°C; it was therefore necessary to establish, for each test car, a correlation between the ambient test temperatures and the riding quality, in order to correct for temperatures between 15°C and -25°C [7]. Several miles of an elevated structure were chosen for the test (the Metropolitain Boulevard in Montreal) in order to eliminate any influence that frost could have on the riding quality of the pavement. Five test sections were selected to represent different riding qualities. Testing speeds were maintained at 60 km/h as much as possible and three repeated tests were done for each testing temperature in order to increase precision. As illustrated in Fig. 3, a good correlation was found between the temperature and the value of the Mays riding quality for the five test sections. The coefficient of correlation varied between a low of 0.73 and a high of 0.90, and the slope varied between 1.6 and 2.4. The following equation was therefore established to correct the Mays value measured at any temperature between -10 and -25°C to a standard temperature of 10°C.

$$\text{Mays (10°C)} = \text{Measured Mays} + 2\,\Delta T \text{ (in cm/km)}$$

(where ΔT is the difference between the ambient temperature during the test (<10°C) and the standardized temperature of 10°C).

For example, if the riding quality measured at -25°C is 285 cm/km, the corrected riding quality will be:

$$\text{Mays (10°C)} = 285 + 2\,(10 + 25) = 355 \text{ cm/km}$$

4.2 Influence of lateral displacement across the wheel track

As indicated earlier, the variation of results in riding quality is mainly caused by the different wheel paths followed within the wheel tracks from one test to another, while the temperature and other variables are controlled.

Information on the effects of wheel paths has been obtained by repeated tests in the summer for both rural and urban roads.

The James Bay penetration road [3] was used to evaluate the total normal variation due to the variation in wheelpaths and the variation of other

controlled variables. The road was divided into 476 one-kilometer sections and the test was repeated twice during the same summer. Statistical analysis of the differences obtained is summarized in Table 2.

Table 2

Difference in the Mays Riding Index (MRI) of two repeated tests

Number of sections	Mean of differences	Standard deviation of differences	Standard error of differences
476	1.64	22.2	1.02

A Student paired t-test was applied to test the null hypothesis of difference between the two sets of observations. The calculated t-value is

$$t = \frac{\bar{D}}{s/\sqrt{n}} = \frac{1.64}{1.02} = 1.61$$

where $\bar{D}$ is the mean of differences and s is the sample standard deviation of the differences, and n is the number of sections.

The probability of observing a t-value that is greater than or equal to 1.61 is 0.1094 for a two-trial test.

Therefore we can safely say, at the 5% level of significance, that: There is no significant difference between the two sets of measures i.e., the observed differences can be almost explained by sampling variation alone.

Furthermore, the histogram in Fig. 4 illustrates the variation of the differences between the two series of Mays measurement. It shows that the distribution conforms to the shape of a normal distribution curve.

Based on these analysis and observations, it is assumed that the overall standard variation Mays for highways expressed in cm/km is equal to 22 cm/km. This value is used in the sub-system to assess if the difference between Mays measured during winter and the one measured in summer of fall can be attributed to the effect of winter.

It is easy to conceive that the overall standard deviation of differences in Mays values obtained for highways is not applicable to city streets where wheelpaths are generally not well defined and where results are influenced by varying average speeds (30-50 km/h). Similar measurements were thus made on 3 groups of streets, the results of which are summarized in Table 3.

Table 3

Difference in MRI determined in summer

Group of streets	Number of sections	Mean of differences	Standard deviation of differences	Student "t"
1	58	21	108	1.5
2	4	16	137	0.7
3	17	8	121	0.3
Total	119	15	122	1.55

Examination of the results indicate that the difference in the mean can not be attributed to an assignable cause and that the overall standard deviation of the differences in Mays for city streets is in the order 122 cm/km.
A Fisher F test was conducted to compare the standard deviation of city streets and rural roads:

$F = 122/22.2 = 5.5$

This value is highly significant (less than 10^{-7}) since the degree of freedom of the F distribution is respectively 118 and 475 for the numerator and the denominator. Therefore the standard deviation of Mays is very much different for city streets and rural roads.

5. Evaluation of the effect of winter

Once the effect of temperature is controlled and the overall standard deviation known, it is possible to describe decision rules for the influence of the effect of winter on the riding quality of pavements.

Table 4

Standardized scale for maintenance sub-system

Influence of the winter on the riding quality	Difference in Mays in cm/km (winter value-summer value)	
	For highways	For city streets
Nil	80 and less	240 and less
Significant	80-160	240 - 480
Very significant	>160	>480

The scale in Table 4 was defined and standardized for use in the maintenance sub-system.

6. Examples of application of the sub-system

An application of the sub-system to establish maintenance priorities is given as an example hereafter. It deals with the requirement and priority of maintenance for 110 km of pavement of the Montreal-Sherbrooke autoroute. The winter and summer riding qualities of the pavement are illustrated on the histogram in Figs 5a and 5b. Each number on the histogram represents a kilometer of road (km 10 to km 121). Examination of the figures indicates that the riding quality of the pavement of many sections changes drastically from summer to winter. For example, the section between km 112 and 113 (numbered 112), which has an "average" riding quality during the summer, degrades to a "bad" riding quality in the winter.

Figure 6 shows the extent of the effect of winter from kilometer to kilometer. the histogram ranks the 111 sections according to their severity. Cumulative frequency curves such as the one shown in Fig. 7 are a practical way of illustrating the effect of winter on the whole highway. It clearly shows, for example, how the 111th kilometer of pavement (which has only 5%) of the total length classified as "bad" riding quality during summer) increases the proportion of "bad" to nearly 50% in the winter.

An example of a computer print-out of the list of maintenance priorities for pavement sections with "bad" riding quality during the winter is shown in Figs 8a and 8b. Figure 8a lists, according to the degree of severity, sections of pavement that are very sensitive to the effect of winter; consequently they have been given maintenance priority "one". Figure 8b shows pavement sections which are moderately sensitive to the effect of winter; they have been given maintenance priority "two". Figures 9a and 9b show pavements that have "intermediate" riding quality during the winter; the ones which are sensitive to the effect of winter have been given priority "three", whereas those which are slightly sensitive to the effect of winter have been given priority "four".

Note that within any of the priority groups 1, 2, 3 and 4, it is again possible for maintenance purposes to select pavement sections according to the magnitude of the effect of winter; each section of road having been numbered and indicated on the histogram.

Two more examples are given here to show how the information obtained through the sub-system can be used to generate useful information on causes

of degradation during the winter.

The first example deals with the performance of the James Bay access road. Bumps at cracks and at culverts were recorded during the measurement of riding quality during the winter. Figure 10a shows the correlation obtained between the number of bumps at cracks per kilometer and the corresponding average Mays riding index, whereas Fig. 10b shows the correlation between the number of bumps and the average Mays riding index. It is apparent that both bumps at crack and at culverts have an influence on the riding quality of the road with the effect of bumps at the culverts being more pronounced. A similar example for 61 street blocks is illustrated in Fig. 11. It shows that there is a relationship between the riding quality of city streets and the total length of cracks expressed as a percentage of the length of the street.

The second example deals with 304 blocks of City streets. The purpose here was to determine if the condition of the pavement had an influence on the effect of winter. The streets were thus divided in two categories: those with a riding comfort index above 2.25 (i.e. 54 blocks), and those with riding comfort index below 2.25 (i.e. the remaining 250 blocks). The cumulative frequency curves for the two categories of road are illustrated in Fig. 12. It is interesting to note that streets with lower riding quality were more severely affected by winter. This is not surprising, since cracks and thus frost heaves are generally, but not necessarily, more frequent on pavements with poor riding quality.

7. Evaluation of pavement degradation

A rational method has been developed to evaluate the degradation of pavements during winter. It consists of measuring globaly the pavement conditions and, in detail, the extent and severity of the following defects:

- Transversal cracks with deformation and/or degradation.
- Pot holes.
- Deformation of the pavement surface.
- Peeling to asphalt surfacing.

The severity of the defects is expressed in terms of the maintenance requirements:

- Level 1 means that the defect is in the process of being developped.
- Level 2 means that the defect is fully developped.
- Level 3 means that maintenance or repair is necessary.

The degree of the degradation is expressed by two parameters:

- The deformation index, which is the total number of deformed cracks and deteriorated cracks multiplied by 100 and divided by the length of the pavement;
- The degradation index, which is the product of the number of holes times the degree of degradation of holes multiplied by 100 and divided by the length of the pavement.

Pavement with a high deformation or degradation index need special attention.

8. Conclusions

1. In northern countries, heaving and settlement of pavements built on frost-susceptible materials degrade riding quality during the winter.
2. To alleviate deterioration of pavements during winter, it is necessary to integrate, in a rational pavement management system, a sub-system to minimize the effect of winter on the roughness of pavement.
3. When the effect of winter on the pavement is severe, a simple crack may significantly deteriorate from one winter to another.
4. A sub-system has been developed to assess the effect of winter and to establish maintenance needs and priorities.
5. It is possible to evaluate pavement roughness during winter with a road meter. However, it is necessary to correct the results in order to account for the effects of temperature.
6. The overall standard deviation for the Mays Ride Meter has been established. It takes into account the normal variation of all the controlled variables and the run-to-run variation due to the change of wheel path.
7. The overall standard deviation of a Mays Ride Meter test for rural roads differs considerably from the one for city streets:

σ_{road} = 14 in/mile, 22 cm/km

σ_{street} = 75 in/mile, 120 cm/km

8. The sub-system to minimize the effect of winter is currently used in Quebec for maintenance management and for the assessement of causes of degradation.
9. Standards have been developped to rank pavements according to their roughness and their susceptibility to degradation during the winter.
10. Experience shows that only a limited, but significant, portion of pavements from a road or street network are subject to the effect of winter.

11. Several studies have indicated that degradation of roughness during the winter may be attributed to different causes: the main cause for the 620 km James Bay penetration roads (built in 1976) is deformation at cracks and culverts, whereas for city pavement of varying ages it seems to be the condition of existing pavements.

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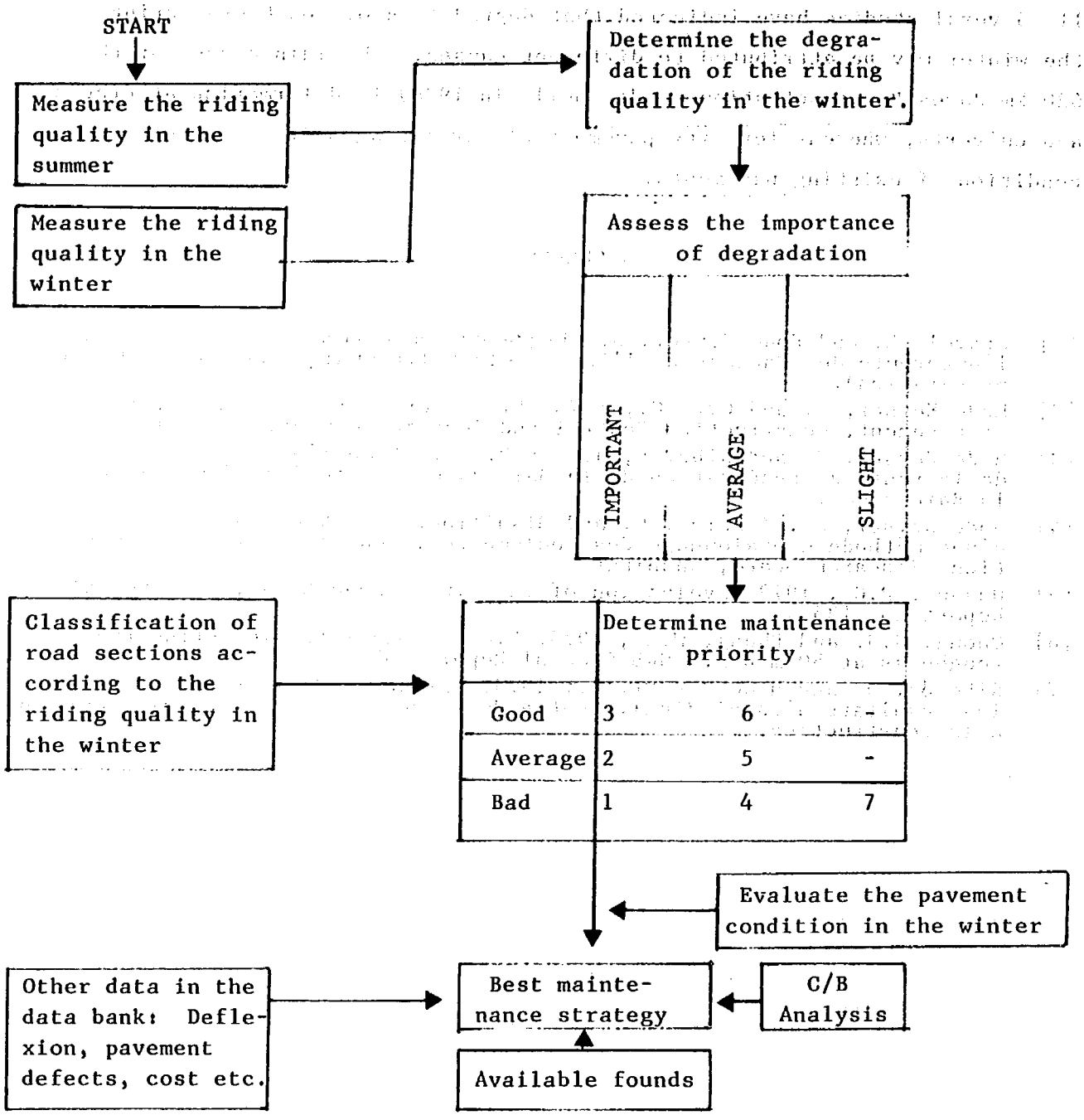


Fig. 2. Concept of the sub-system to minimize the effect of winter.

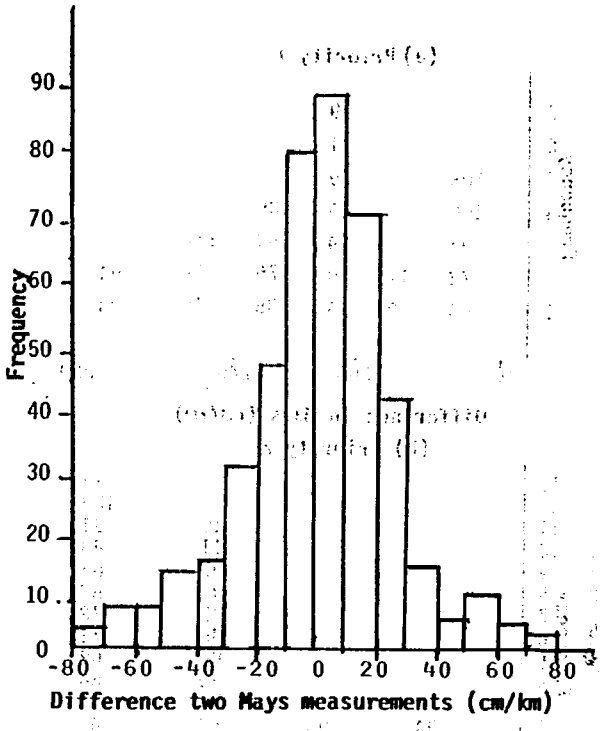
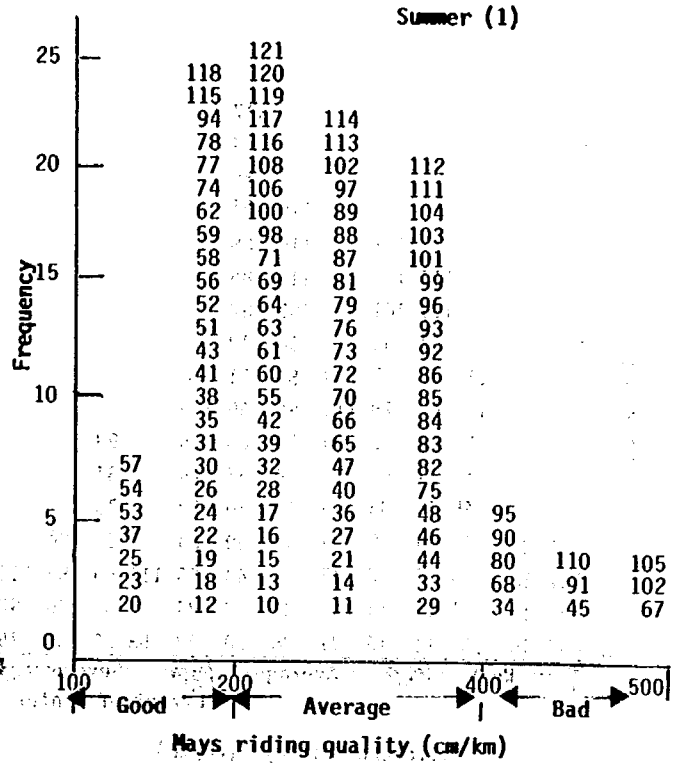
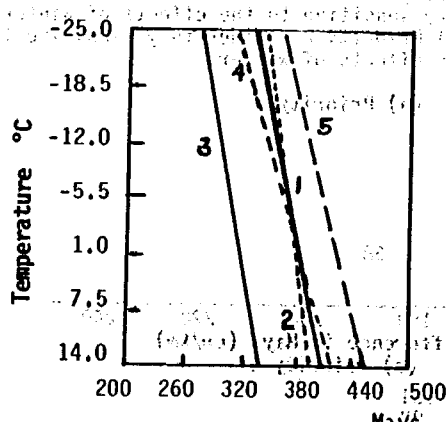


Fig. 4. Difference between two series of riding quality measurements of a rural road.





SECTION	SLOPE	COEFFICIENT OF CORRELATION
1	1.8	0.76
2	1.6	0.73
3	1.8	0.87
4	2.4	0.90
5	2.2	0.79
AVERAGE	2.0	

Fig. 3. Influence of temperature on the Measured Riding Index.

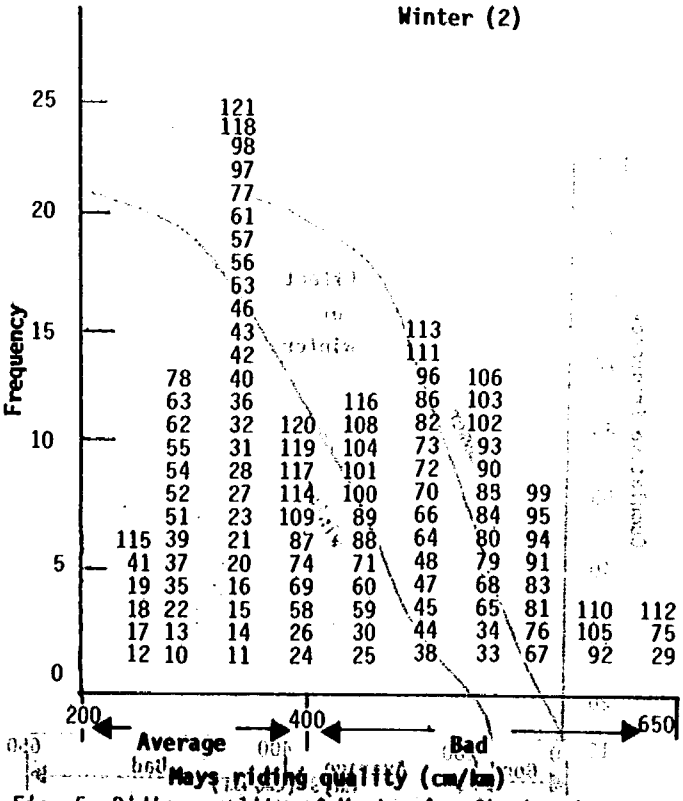


Fig. 5. Riding quality of Montreal - Sherbrooke autoroute; (1) Summer (2) Winter.

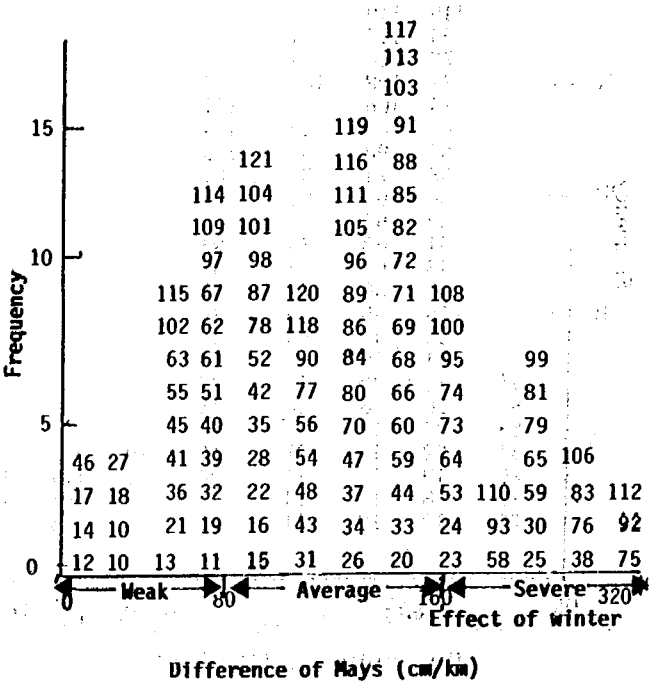


Fig. 6. Deterioration of riding quality during the winter.

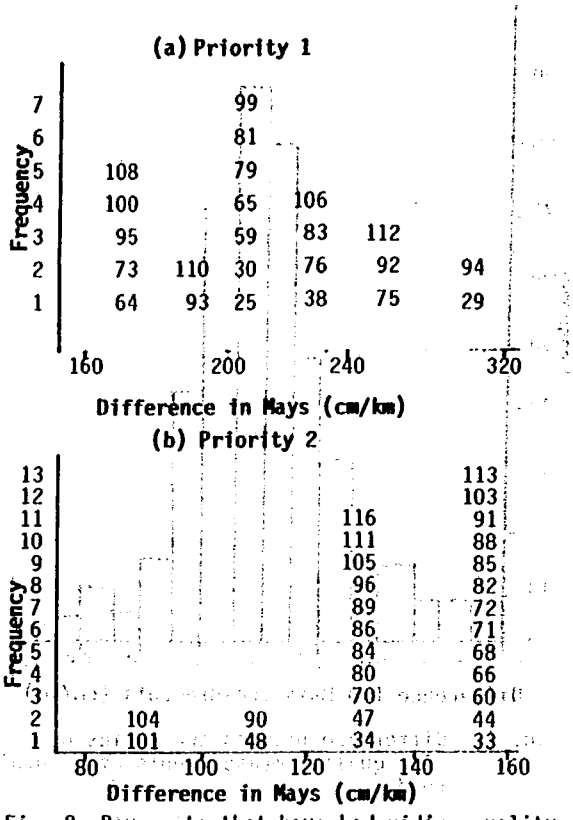


Fig. 8. Pavements that have bad riding quality during the winter; (a) Priority 1 - very sensitive to the effects of winter, (b) Priority 2 - moderately sensitive to the effects of winter.

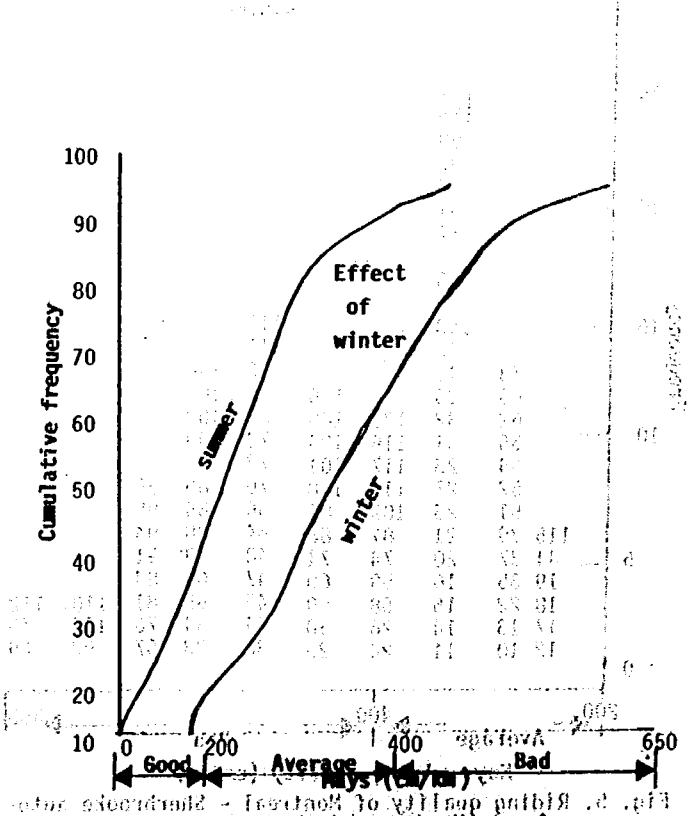


Fig. 7. Effects of winter on the Montreal - Sherbrooke autoroute

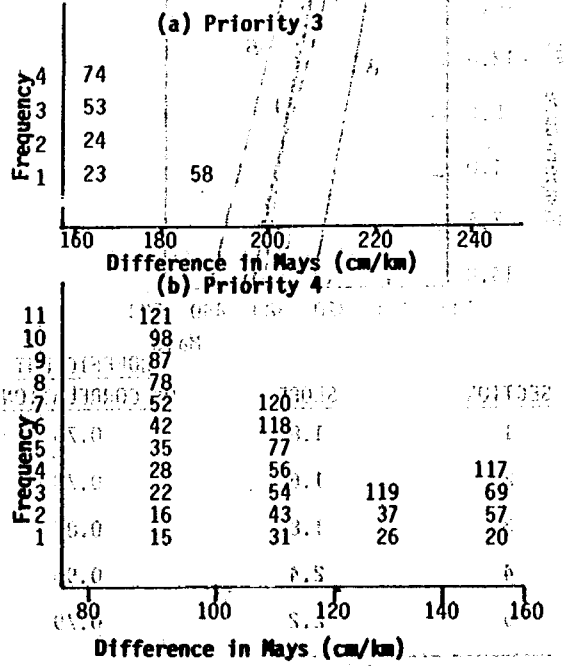
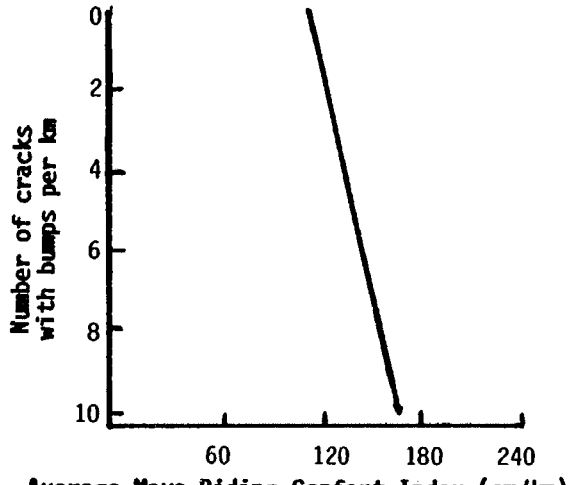


Fig. 9. Pavements of intermediate riding quality during the winter and which need protection against water infiltration into the cracks; (a) Priority 3 - sensitive to the effects of winter, (b) Priority 4 - slightly sensitive to the effects of winter.



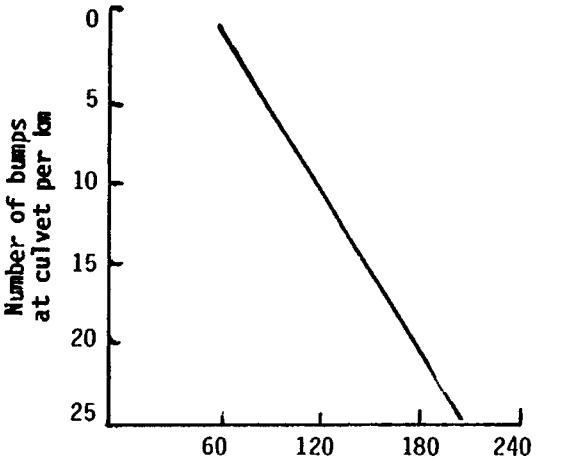


Fig. 10. Influence of bumps at crack (a) and at culvert (b) on the riding quality.

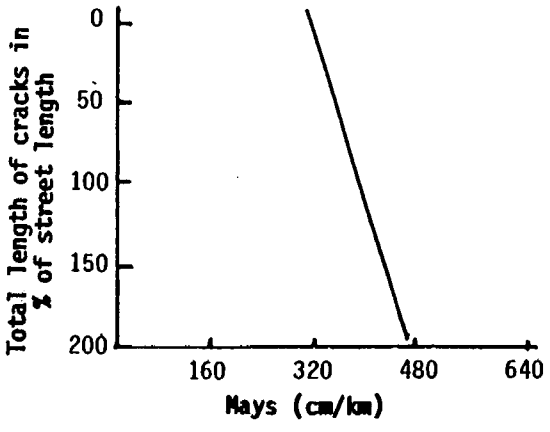


Fig. 11. Relation between riding quality and number of cracks in 61 streets.

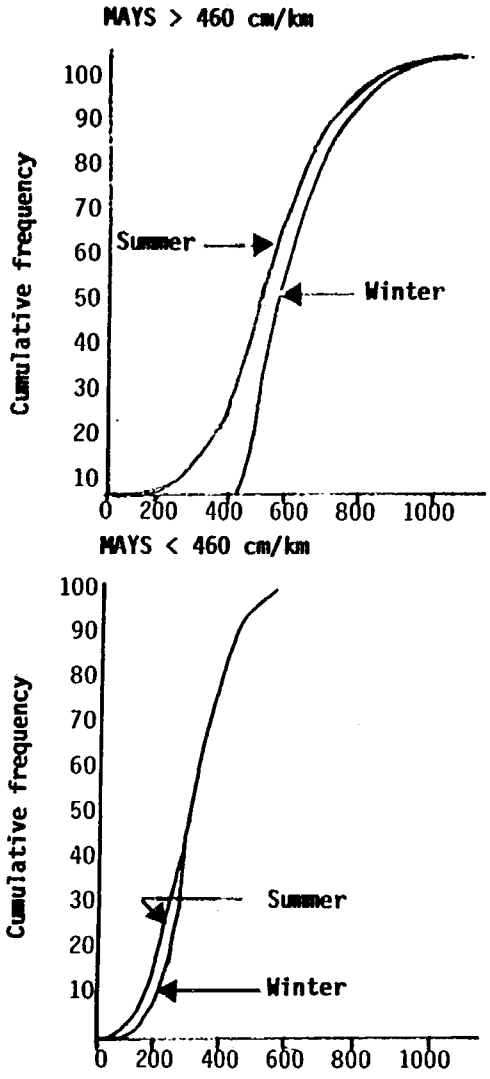


Fig. 12. Influence of pavement conditions on the effect of winter (for 304 blocs of pavement).

Thermal Fracture of Bituminous Mixtures

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ABSTRACT

This paper describes the test results on thermal stress and fracture of bituminous paving mixtures investigated from the following experiments;

a. Effects of air voids, composition of mixtures and type of mixtures.
b. Change of temperature induced stress at constant low temperature for a long duration.
c. Temperature induced stress under the various modes of temperature change.
d. Behavior of mixture under the repeated cooling in various temperature levels and amplitudes.

The conclusions based on the findings of this study are stated below.

a. Lower temperature induced stress was ontained in the coarser mixture, and higher temperature induced stress was obtained in the higher density mixtures.
b. When temperature of specimen is maintained at constant temperature, the decrease of stress was shown. And in some cases, the fracture was observed during the suspending temperature at close to the Fracture Point. This can be represented from both growth of micro cracking and stress relaxation in the specimen.
c. Under the repeated cooling, stress is gradually decreased, and fracture was shown in some cases. The increase of repetitions is liable to lead the mixture to the fatigue type failure.
d. It seems that the Transition Point will serve as an index in the mixture behavior under various modes of temperature change.

THERMAL FRACTURE OF BITUMINOUS MIXTURES

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1/ INTRODUCTION

In the previous paper(1) which was presented at the 1982 PICA Workshop, fracture strength, permmissible strain of asphalt concrete observed by the conventional mechanical tests at low temperatures and temperature induced stress and the fracture properties of asphalt concrete obtained from the direct thermal fracture tests were discussed. And also the nomogram for the prediction of thermal fracture temperature was presented.
A present paper describes the test results on thermal stress and the fracture investigated from the following experiments;

a. Effects of air voids, composition of mixtures and type of mixtures on the thermal properties of mixtures
b. Changes of temperature induced stress at constant low temperature for a long duration
c. Temperature induced stresses under the various modes of temperature change.
d. Behavior of mixture under the repeated cooling in various temperature levels and amplitudes.

2/ TEST METHOD AND COMPOSITION OF MIXTURES

2.1/ Test Method

A specimen measuring 2.5 cm by 2.5 cm by 26 cm was virtually constrained to maintain its initial length throughout a range of temperature. Then the specimen was subjected to cooling at a programmed cooling rate. The loads associated with the temperature lowering were measured by a load cell. A specimen coated with thin emulsified rubber to avoid water penetration was cemented with epoxy resin to steel end pieces which were connected with a rigid steel frame through the extension rod made of invar steel. A water

bath filled with methanol was placed in such a way that the invar extension rods passed through both end walls of the water bath. The load cell was placed at the end of extension rod at the outside of the water bath. In order to expect higher accuracy, this equipment including the steel frame was placed in a constant temperature box.

Fig. 1 shows the schematic diagram of this equipment which was consists of temperature programming unit, cooling unit, measuring and recording unit, steel frame, load cell and water bath. The measurable lowest temperature in this equipment is set at -40°C at a cooling rate of 30°C/hr.

2.2/ Composition of Mixtures

The types of mixes are as followings:

Type-A	:	Dense graded asphalt concrete
Type-B	:	Fine graded asphalt concrete
Type-C	:	Coarse graded asphalt concrete
Type-D	:	Stabilized asphalt base material

Pen. 86, and Pen. 60 asphalts were used in this study, Table-1 shows the property of asphalts and Fig. 2 shows the grading curves of each type of mixes and Table-2 shows the mix compositions. The Type-A mix which was composed of 5.8 % asphalt was adopted as a control mixture in this study, the Type-A mix includes a variation of asphalt content and air void. The air void was controlled from a number of roller passage, while an effect of the asphalt contents was obtained from the controlled air void specimen.

2.3/ Reproducibility of Test Results

To obtain a reproducibility of the test result, temperatures and stresses at both Transition Point and Fracture Point were measured on 10 specimens. Table-3 and Fig. 3 present the test results. There were scattering on temperature and stress at the Fracture Point in some extent, but there were very small scatterings at the Transition Point. From Fig. 3 this scattering at the Fracture Point is attributes by fluctuation depending on the fracture mechanism itself at low temperature, because the fracture points lie in a narrow band of stress-temperature curve. Widths of this

band were 3.3 kg/cm² in stress and 1.6°C in temperature at the Transition Point, and also 7.4 kg/cm² in stress and 2.9°C in temperature at the Fracture Point. Then it was concluded that this type of the thermal fracture test can be used with a reasonable level of accuracy.
In this study, 30°C/hr of a cooling rate was adopted, lower fracture temperatures and higher fracture stresses (strength) were shown compared with the case of 12°C/hr, these differences might be caused from a difference of rate of deformation.

2.4/ Analysis of Test Results

Fig. 4 is a schematic representation of the temperature induced stress versus temperature relationship obtained from the direct thermal fracture tests.
There is no shrinkage in the restrained specimen, and the lowering temperature causes an elongation in the specimen. This elongation can be calculated from the temperature difference and the thermal contraction coefficient of the specimen. A value of 2×10^{-5} /°C for the coefficient was chosen based on the thermal contraction test in this study. The slope of this illustrated stress-strain curve gradually increases as the temperature goes down to certain temperature; beyond this critical temperature, the slope of this curve reaches a constant value and a straight line is seen. It is certain that the temperature induced stress is relaxed down to this temperature, while there is no stress relaxation in a straight line zone. In this study, this particular temperature was defined as a Transition Point. The presence of this particular point shows that the stress-strain curve consists of two zones of relaxation and non-relaxation where the strain is independent of time; thus the Transition Point is critical.
In a present study on a repeated cooling test of bituminous mixture under the various modes of temperature changes, this Transition Point was used as an index of selection of temperature levels to be tested.

2.5/ Nomogram for Prediction of Probable Fracture Temperature

Fig. 5 which was presented at the 1982 PICA workshop is the nomogram for the prediction of thermal fracture temperature of asphalt concrete

constituted from various types of asphalts. This approach is basically a variation of the limiting stiffness approach based on the direct thermal fracture test and the van der Poel stiffness nomograph(2). This is valid for the conventional asphalt concrete for pavement having 0.84 to 0.86 for the volume concentration of aggregate.

3./ EFFECT OF AIR VOID, COMPOSITION AND TYPE OF MIXTURES

3.1/ Effect of Air Void

Thermal stresses and fracture temperatures of the Type-A mix having 4 levels of the air void of 2.3-2.4, 2.8-3.4, 4.8-5.1 and 6.3-6.7 % were measured. The air void was controlled by changing a number of roller passage. Test results were shown in Fig. 6. Relatively, a higher density specimen showed lower fracture temperature, while the lower fracture strength is seen on a low density mixture with high air void content. Although there are no remarkable difference on temperature and stress at Transition Point, there are the considerable difference on the slope of stress-temperature curve at low temperatures. This drop in the fracture strength is not necessarily the only cause of the tensile strength of the material, but the slope of the stress-temperature line at below the Transition Point is corresponded to the mixture stiffness obtained from conventional mechanical tests at low temperatures.
A range of straight line zone (elastic zone) was shorten in the higher void mixture, this might be expressed from a view point of the growth of a micro cracking in the specimen.
As a result, it was concluded that the effect of the void contents of the mixture is considerable on temperature induced stress.

3.2/ Effect of Asphalt Content on Thermal Stress in Constant Air Void Mixtures

Three levels of asphalt contents, 5.3, 5.8 and 6.8 % were chosen to study a contribution of the asphalt content, the void content was kept at 3 % in this study. Fig. 7 shows the test results. There were no differences in the fracture temperatures, while a few difference on stress was found. This reason was analyzed as a result of the influence of stress relaxation

of high asphalt content mixture at above the Transition Point. It was concluded that the asphalt content was not critical factor regarding temperature induced stress in the conventional paving mixtures.

3.3/ Effect of Mix Type

Four types of mixture described above were used to study the effects of the mixture type on the thermal property. Test results were shown in Fig. 8. There are significant differences in the fracture stress. Finer the aggregate grading, higher fracture stress was obtained, but no remarkable difference in the fracture temperature itself. These results were similar to those of the effects of air void contents which were shown in Fig. 6. The Transition Point is not seen in the coarser mixture, and also the elastic zone was narrowed. This result show lower temperature induced stress was given in the coarser mixtures, it is suspected that the coarser mix keeps higher resistance to a generation of the thermal cracking, a further discussion is needed from a permmissible strain of mixtures. This phenomena should be also discussed from a view point of a thermal fatigue and a stress relaxation of the mixture. The tensile strength obtained from an indirect tension test, is measured as 60-70 kg/cm^2 in Type-A and Type-B mixes. This means the strength shown by thermal fracture test is only one half of that of the fracture strength which was obtained from a mechanical test, this drop in strength was suspected of effects of both growth of micro cracking and loading time in specimen during the test.

3.4/ Findings

From three types of test, the following conclusions were reached.

a. It is considered that the effects of the void content appeared sensitively in the thermal stress.
b. Higher density mixture shows lower thermal stress.
c. The effect of the asphalt content is not critical in the conventional dense graded asphalt concrete type mixture.
d. It is difficult to control the generation of thermal stress in the conventional dense surface mixture by the selection of mix composition.

e. The properties of asphalt itself might be a most influential factor on thermal fractures as described in the previous paper.

4/ TEMPERATURE INDUCED STRESS UNDER VARIOUS MODES OF TEMPERATURE CHANGE

4.1/ Modes of Temperature Change

Following three modes which were shown in Fig. 9 were adopted in this investigation.

a. Case-1 Cooling down to the appointed levels of temperature (-14.5, -20.5, -22.0, -26.0 and 27.5°C) and then maintain this temperature for 20 hours.

b. Case-2 Cooling down to appointed levels of temperatures (-10, -15, -18, -20 and -25°C) and maintain this temperature for 2 hours, and again cooling was given down to the Fracture Point.

c. Case-3 Cooling down to appointed temperatures (-15, -20 and -25°C) and raise temperature up to initial temperature (10°C) and again cooling was given down to the Fracture Temperature.

Appointed temperatures were chosen at three levels of the Transition Point, above the Transition Point and below the Transition Point.
In this study, a temperature programming equipment was used.

4.2/ Case-1

To study the changes of temperature induced stress at constant low temperature, a specimen was cooled down to appointed temperature, -14.5, -20.5, -22, -26 and -27.5°C, and then temperature suspended at these levels and the changing of stresses was measured. A cooling rate of 30°C/hr was adopted in the Case-1 study. Fig. 10 shows the test results, mean fracture temperature was 30.4°C and mean fracture stress was 33.8 kg/cm² in the Type-A mixture in the single cooling test. Two appointed temperatures of -27.5 and -26.0°C are very close to the Fracture Temperature. The fractures were observed in these two cases within 3 and 4 hours after the

cooling is suspended. Although temperature does not reaches the Fracture Temperature, the specimen is broken, this was likely due to the growth of the micro cracking in the specimen. In other cases, the fractures were not seen, but there were decreases in stress. A marked decrease of stress was shown in a initial stage at all appointed temperatures, and stress gradually decreases in the following time duration. These decreases were considerable at higher temperatures. Decreases of stress were 34, 50, and 64 %, at -22, -20.5, and -14.5°C respectively after 20 hours. These decreases might be caused from both stress relaxation and growth of micro cracking in the specimen. This represents that the stress relaxation property is distinguished in the relaxation zone above the Transition Point, and also both relaxation and micro cracking are generated in the non-relaxation zone below the Transition Point.
It was concluded that although there is no change in temperature, stress decreases gradually with the passage of time and this stress would lasts for a long period of time.

4.3/ Case-2

The specimen was cooled down to appointed temperatures, -10, -15, -18, -20 and -25°C, at the cooling rate of 12°C/hr and then maintained these temperatures for two hours and then again was cooled down to the Fracture Point. Fig. 11 shows the test results. During the suspending temperature for 2 hours, stress was decreased, this decrease is essentially the same as to those of the Case-1, the stress-time lines during the second cooling were quite similar to those in the first cooling, and there were no significant differences in the Fracture Temperature. This result indicates the temperature induced stress is generated regularly depending on the temperature change.

4.4/ Case-3

After the cooling down to appointed temperatures, -15, -20 and -25°C, temperature was raised back to 10°C, and the second cooling was given. Fig. 12 shows the test results, to those obtained the stress-temperature lines in the second cooling showed quite similar in the first cooling. It is very interesting result that compressive stress is shown at near the

maximum temperature (10°C). These were a common through the three tests. This means that the stress relaxation property is generated during the cooling, this also means an elongation of specimen, and no stress condition appears before the strain reaches zero during the raising temperature. This suggests that the complex stress condition exists during cooling and raising temperature, there are the complicated effects of stress relaxation as function of time and temperature.

4.5/ Findings

The major findings of this investigation can be summerized as followings:

a. Temperature induced stress is generated regularly depending on the temperature change.

b. When temperature of specimen is maintained at constant, the decrease of stress with the passage of time was observed at all temperatures. It was suspected that the effect of the progress of damage is distinguished at below the Transition Point, while the stress relaxation is distinguished at above the Transition Point.

c. There are a generation of compressive stress in the specimen during the raising temperature.

d. In some cases, the fracture was observed during the suspending of temperature at low temperature, such as -26, -27.5°C, this might be caused from the growth of micro cracking under higher tensile stress.

e. In general, bituminous mixes represent the complex behavior under the temperature change. It is quite difficult to obtain a conclusive result.

5/ BEHAVIOR OF BITUMINOUS MIXTURES UNDER REPEATED COOLING

5.1/ Repetition of Cooling

A behavior of bituminous mixtures under the repeated cooling was investigated. Levels of temperature to be tested and the temperature amplitudes are widely varied. Table-4 shows the temperature ranges, amplitudes and representative test results. Maximum temperature (higher

limit during the test) and minimum temperature (lower limit during the test) were chosen at following three levels ;

a. Between the Transition Point and below the Transition Point
b. Between the Transition Point and above the Transition Point
c. Between below and above the Transition Point

A high viscosity asphalt having Pen. 60 was also used to discuss an effect of binder consistency. Fig. 13 shows a schematic representation of the test result. In this case, a word of repetition means a couple of a maximum stress caused from lower side temperature and a minimum stress caused from higher side temperature. In general, maximum and minimum stresses decrease steeply in first three to five repetitions, and then decrease gradually, and finally decrease rapidly at near the Fracture Points. A deviation in resulting maximum and minimum stress is nearly constant until the fracture. When temperature was set at higher temperature, compressive stress is observed, this is quite similar to those of Fig. 12.

5.2/ Fracture After Few Repetitions of Cooling (Between -9.5 and 29°C, -23 and -27°C)

In specimens No.4 and No.1, the fractures were seen after the very few repetitions. Fig. 14.1 shows the fracture occured at the second cooling. In this case, minimum and maximum temperatures were set at -29 and -9.5°C, the amplitude was 19.5°C and the deviation in stress was 33.7 kg/cm². This fracture affected by temperature itself which was set close to the Fracture Temperature rather than an effect of repetition. In a case of Fig. 14.2 minimum and maximum temperature were set at -23°C and -27°C, both temperatures are located in non-relaxation zone, the stress deviation showed 8.3 kg/cm². The maximum and minimum stresses are rather steeply decreased in a first stage, and the fracture occured at the fourth cooling. The decreases of the stresses are similar to those of the conventional fatigue test performed at a high level of the stress amplitude. From this test, it was suspected that the mixtures would fail by fatigue at low temperatures. It should be noted that there is an effect of the time duration in this type of test. If the temperature amplitude is higher, the

test is lasts for long period, an analysis of the test results is very difficult to discuss the effects of the repeated cooling itself.

5.3/ Changes of Thermal Stresses Under Repeated Cooling Between the Transition Point and Below Transition Point (Between -20.5 and 25.5°C, 20 and 25°C)

The changes of the stresses induced with temperature changes between the Transition Point and below the Transition Point were investigated. Fig. 15.1 and Fig. 15.2 show the representative test results. Stresses and stress deviations were plotted on the vertical axis and the number of repetition was plotted on the horizontal axis. A stiffness of mixture can be calculated from this stress deviation and thermal coefficient.
Fig. 15.1 shows the fracture of the specimen after the 120 repetitions of cooling between -20.5 and -25.5°C. Both maximum and minimum stresses were rapidly decreased in a first three to five repetitions, and then gradually decreased, after 100 repetitions, stresses were rapidly decreased again. The stress deviation was kept at 9 kg/cm² up to 50 repetitions, while the stress deviation calculated from Fig. 3 was 8.8 kg/cm². This represents an elastic behavior of specimen during the repetitions at low temperature. After 50 repetition, stress deviation was gradually decreased and rapidly dropped at 100 repetitions. It was suspected that these decreases in stress deviations and the fluctuation of stresses show the initiation of fatigue type damage. Fig. 15.2 also shows the same result. In this case temperature was set between -20 and -25°C, the temperature amplitude was 10°C and the resulting stress deviation was 10.2 kg/cm².
It was concluded that the results obtained from the repeated cooling test represent the same tendency with the conventional fatigue test, but the number of repetitions to failure was very small in the repeated cooling. This can be explained from level of applied stress and state of stress.

5.4/ Changes of Thermal Stresses Under Repeated Cooling Between Below and Above the Transition Point (Between -15 and -25°C)

Fig. 16.1 and Fig. 16.2 show the test results on the effects of the repetition of cooling between -15 and -25°C, below and above the Transition Point, on Pen. 86 and Pen. 60 asphalts. Higher temperature induced stress

was exhibited in the Pen. 60 asphalt. The resulting stress deviation was 16-18 kg/cm² in the Pen. 86 asphalt and there is no marked change during 50 repetitions, while the stress deviation was 16-17kg/cm² in the Pen. 60 asphalt, this deviation rapidly decreased after 40 repetitions, this was likely due to decrease of the mixture stiffness, i.e., the generation of fatigue type failure. This result represent the higher resistance of the Pen. 86 asphalt to the repetitions of cooling compared with the Pen. 60 asphalt. The deviation in maximum and minimum stresses calculated from Fig. 3 was 15.3 kg/cm² between -15 and -25°C, this figure presents the reasonable correspondence to 16-18 kg/cm² in this test. Minimum stress is approaching to zero stress line with increasing the repetitions in both Pen. 86 and Pen. 60 asphalts, this is also the same as to those of Fig. 12.

5.5/ Changes of Thermal Stresses Under Repeated Cooling Between the Transition Point and Above the Transition Point (Between -10 and -20°C)

Fig. 17.1 and Fig. 17.2 present the test results on the effects of the repetition of cooling between -10 and -20°C, between the Transition Point and above the Transition Point, for Pen. 86 and Pen. 60 asphalts. The resulting maximum and minimum stresses were decreased at the first three to five repetitions. Stress in the Pen. 60 asphalt was also higher than that of the Pen. 86 asphalt. After 20 repetitions in the case of Pen. 86, and 12 repetitions in the case of Pen. 60, compressive stresses are seen in the minimum stresses. After these repetitions, alternative tensile and compressive stresses are generated in the specimen. The stress deviation between maximum and minimum are 16.5 kg/cm² in the Pen. 86 specimen and 14.5 kg/cm² in Pen. 60 specimen, while 11.7 kg/cm² in the stress deviation was obtained from Fig. 3. There are significant differences between stress deviation calculated from result of single cooling and resulting stress deviation which was shown in the repeating test. A reason of this difference in deviation can be explained from the decrease in minimum stress (at higher temperature) caused from the stress relaxation. In a same reason, the deviation in the Pen. 86 was larger than that of the Pen. 60. It was concluded that the stress relaxation is distinguished in decrease of stress in this temperature range.

5.6/ Effects of Repeated Cooling in Small Temperature Amplitude. (Between -16.5 and -19.5°C)

The effect of the repeated cooling under the small temperature amplitude, 3°C, between -16.5°C and -19.5°C, at above the Transition Point was investigated. The stress deviation was 5.2 kg/cm², while the stress deviation obtained from Fig. 3 was 4 kg/cm². Although the temperature range and resulting stress deviation are relatively small, the degree of decreasing in stress was the same as to those of other cases. It was also suspected that the stress relaxation is distinguished, but there is an effect of the level of minimum stress. In this case, the level of minimum stress is larger than that of -10 - -20°C case, then the larger relaxation, higher decreasing in stresses appeared.

5.7/ Findings

As the results of the study on the changes of thermal stress under the cyclic temperature changes, following conclusions were reached.

a. The mixture shows a very complicated behavior under the cyclic temperature change.

b. In this study, the temperature range are placed in following three levels.

   1) Below the Transition Point

   2) Above the Transition Point

   3) Between below and above the Transition Point.

   In these three cases, somewhat different results were obtained respectively. There are the fatigue type fractures when minimum temperature was set at close to the Fracture Point. When temperatures were set between below and above the Transition Point, both fatigue type and relaxation type changes of resulting stress were obtained. At the higher temperature above the Transition Point, the relaxation type change was distinguished. These behavior caused from the effects of combined relaxation and progress of damage in the specimen.

c. In the harder asphalt, the fatigue type failure was observed earlier compared with the softer asphalt.

d. Stiffness modulus is one of the critical factors for the cyclic temperature change.

e. There were marked fluctuations in temperature induced stress during the repetitions. This might be caused from a minute movement of aggregate during the cooling. Then this fluctuation should not be neglected to discuss the effects of temperature change, a further research is needed on the change of material property during the repeated cooling test.

6/ CONCLUSIONS

This report summerizes the findings of a study on the effects of the mixture composition and the mechanical behavior of mixture under the various modes of the temperature change including the cyclic temperature change. The conclusions based on the findings of this study are stated below.

(1) Lower temperature induced stress was obtained in the coarser mixture, and the higher temperature induced stress was observed in the high density mixture. The asphalt content is not the critical factor.

(2) The generation of temperature induced stress is quite reasonable depending on the temperature change.

(3) When temperature of specimen is maintained at constant temperature, the decrease of stress was obtained. In some cases, the fracture was observed during the suspending of temperature at close to the Fracture Temperature. These can be represented from both growth of micro cracking and stress relaxation.

(4) Under the repeated cooling, stress is gradually decreased with the increase of the number of repetition. The increase of repetition is liable to lead the mixture to the fatigue failure.

(5) It seems that the Transition Point will serve as an index in the mixture behavior under various types of temperature change.

(6) A mechanism of change of the mixture behavior under the cyclic temperature change has yet to be sufficiently clarified, a further research is needed.

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(2) van der Poel, C., "A General System Describing the Visco-Elastic Properties of Bitumens and Its Relation to Routine Test Data," Journal of Applied Chemistry, Vol.4, 1954.

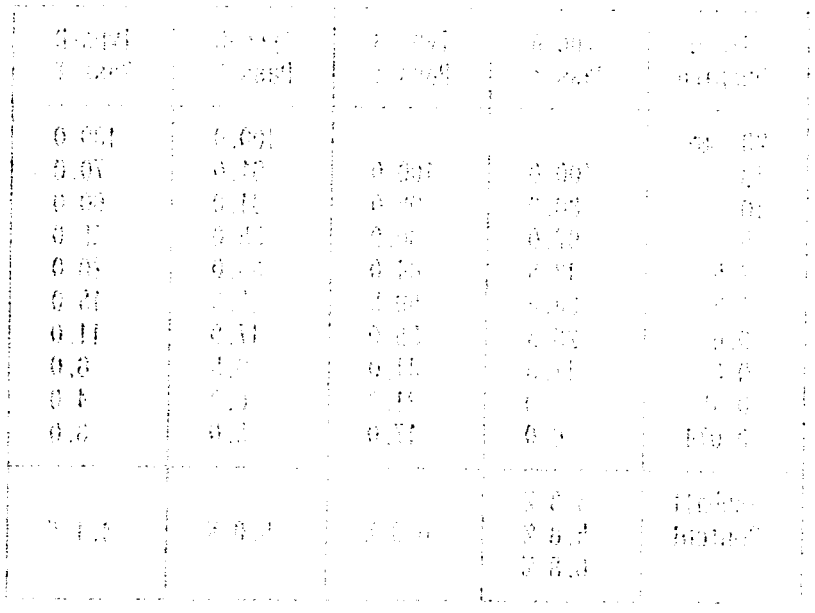

Table-1 Property of Asphalt

Pen. Grade	80/100	60/80
Pen.	86	60
$T_{R.B.}$	47.8℃	50.6℃
Mix. Temp.	149.5℃	156.0℃
Comp. Temp.	104.0℃	107.5℃

Mix. Temp. : 2 poise temp.

Comp. Temp. : 30 poise temp.

Table-2 Mixture Composition

Sieve Opening	Type-A Pass %	Type-B Pass %	Type-C Pass %	Type-D Pass %
20 mm			100.0	100.0
13	100.0	100.0	91.0	70.0
10	88.0	95.0	81.0	60.0
5	67.0	80.0	55.0	35.0
2.5	48.5	64.0	35.0	20.0
1.2	34.4	59.5	25.5	15.0
0.6	28.5	55.0	17.0	11.0
0.3	17.5	41.0	9.5	6.0
0.15	9.0	21.0	6.0	4.0
0.074	6.0	17.0	5.0	3.0
Asphalt Content	5.3 % 5.8 % 6.8 %	6.3 %	5.0 %	4.1 %

Table-3 Thermal Fracture Tests on 10 Specimens

Rate of Cooling : 30℃/hr
Starting Temp. : 10℃

No. of Specimen	Transition Point		Fracture Point		Slope
	Stress kg/cm^2	Temp. ℃	Stress kg/cm^2	Temp. ℃	$x10^4$ kg/cm^2
1	16.2	-19.9	32.8	-29.2	9.0
2	16.1	-20.2	35.4	-31.9	8.3
3	17.5	-20.9	31.3	-29.2	8.8
4	18.3	-21.2	33.2	-29.3	9.5
5	15.9	-19.6	36.7	-31.4	9.0
6	15.6	-20.7	33.7	-31.8	8.6
7	17.6	-20.5	33.4	-29.5	9.2
8	15.6	-20.4	29.3	-29.0	8.8
9	15.4	-20.0	35.1	-31.0	9.3
10	18.7	-21.0	36.7	-31.9	9.1
Aver.	16.7	-20.4	33.8	-30.4	9.0

Slope : Slope of Elastic zone at Lower than Transition Point

Table-4 Test Condition of Repeated Cooling

Rate of Cooling : 30℃/hr
Starting Temp. : 10℃

No.	Pen	Group	Temp. Range ℃			Stress, kg/cm^2			No. of Repeat.	Fracture
			Max.	Min.	ΔT	Max.	Min.	Δσ		
1	86	Below	-23.0	-27.0	4	33.1	24.8	8.3	3	
2	86	T. P.	-20.5	-25.5	5	26.3	17.1	9.2	120	
3	86		-20	-25	5	28.6	18.4	10.2	50	No
4	86	Above and	-9.5	-29.0	19.5	35.6	1.9	33.7	1	
5	86	below T.P.	-15	-25	10	26.0	10.1	15.9	50	No
6	86	Above	-10	-20	10	19.4	5.2	14.2	50	No
7	86	T.P.	-16.5	-19.5	3	17.4	11.8	5.6	50	No
8	60		-15	-25	10	29.6	13.5	16.1	50	No
9	60		-10	-20	10	20.9	7.0	13.9	50	No

T. P. : Transition Point

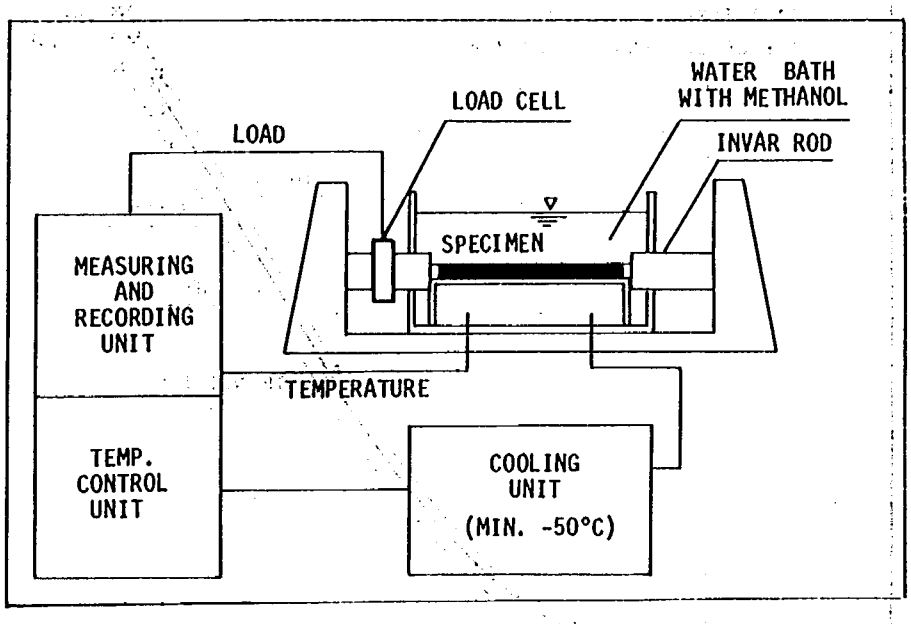

Fig 1 Schematic Diagram of Test Equipment

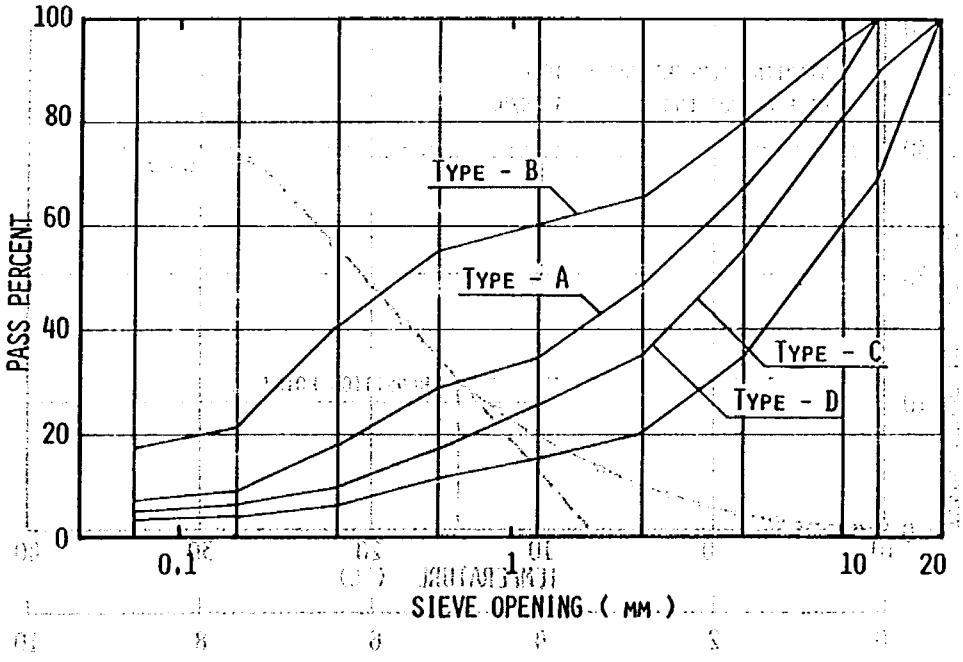

Fig. 2 Aggregate Grading

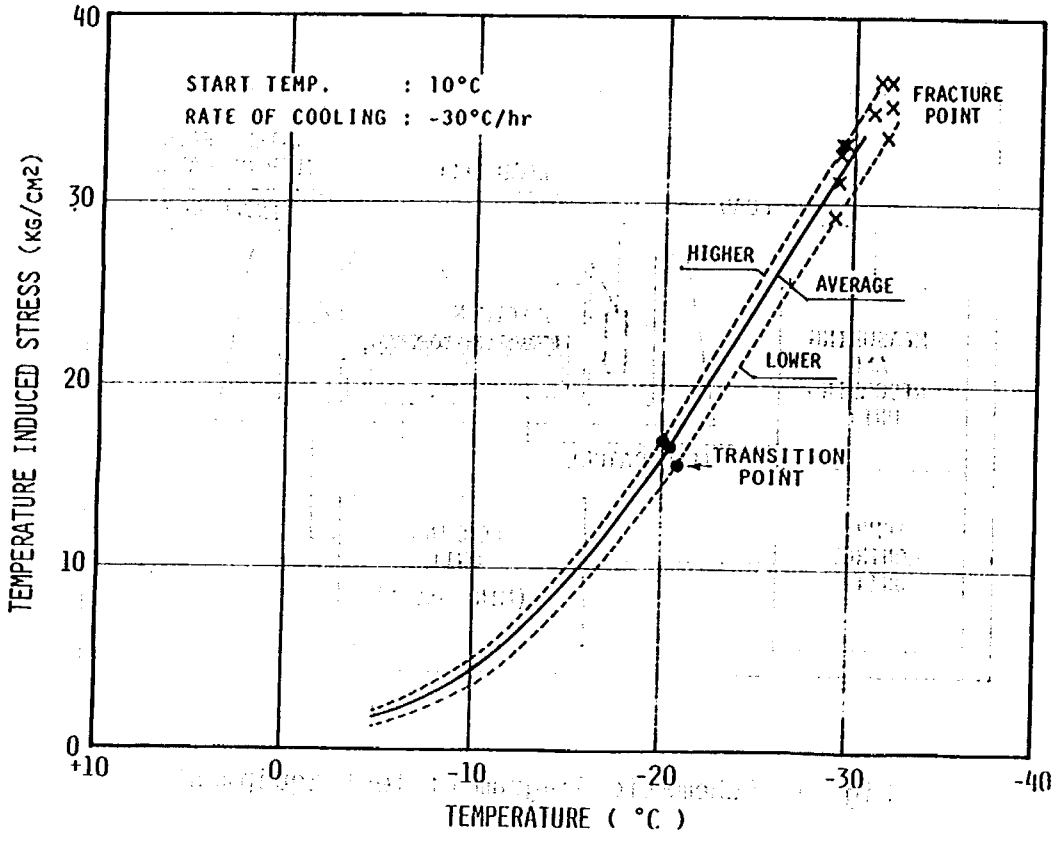

Fig. 3 Thermal Fracture Tests on 10 Specimens

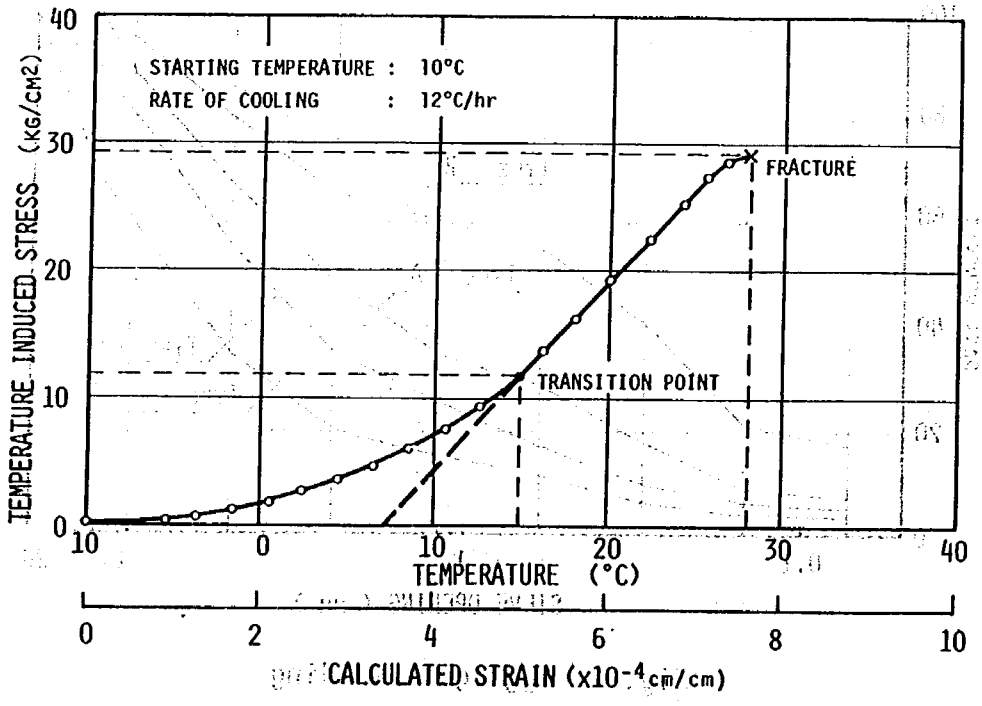

Fig. 4 General Pattern of Temperature versus Temperature Induced Stress

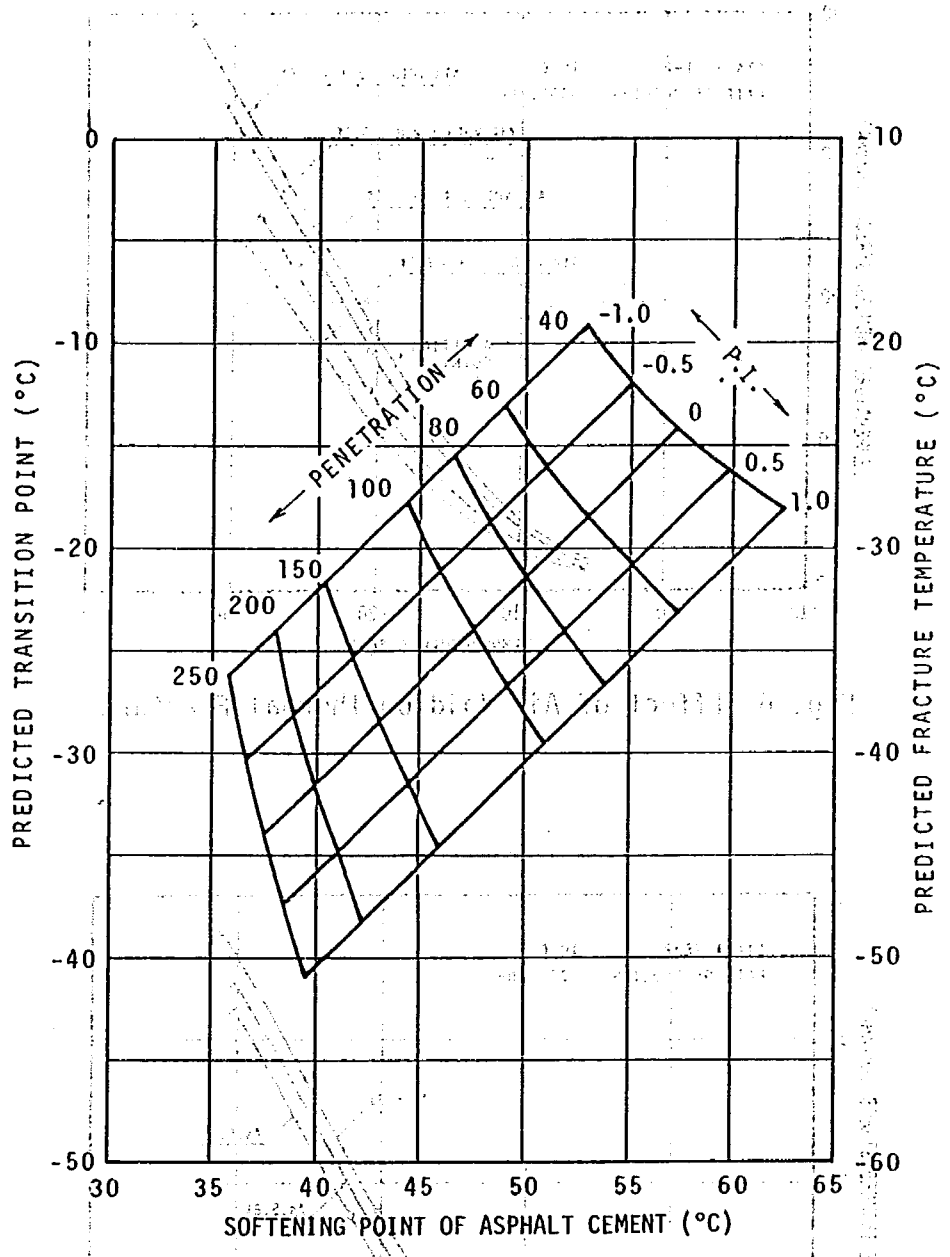

Fig. 5 Nomogram for Prediction of Thermal Fracture Temperature

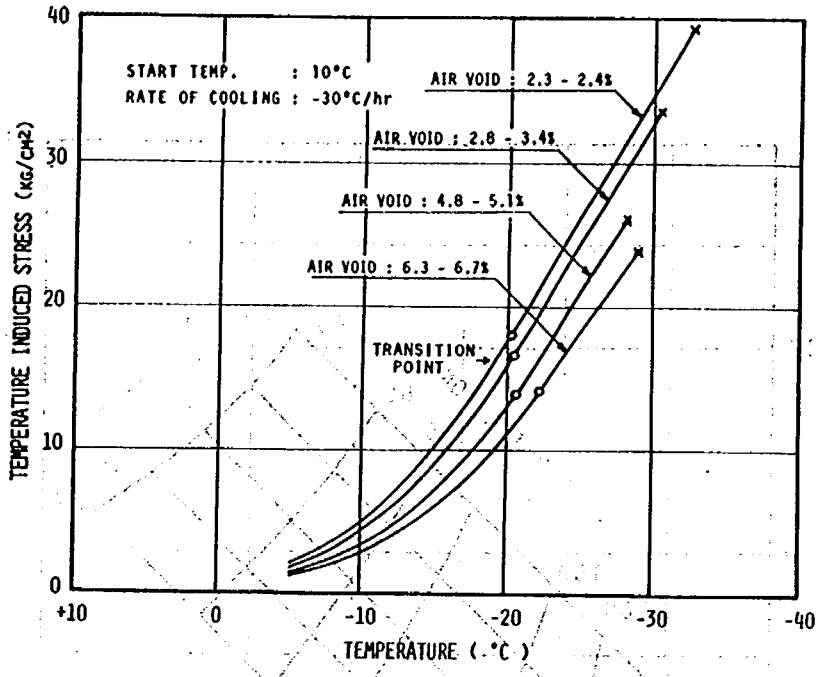

Fig. 6 Effect of Air Void on Thermal Fracture

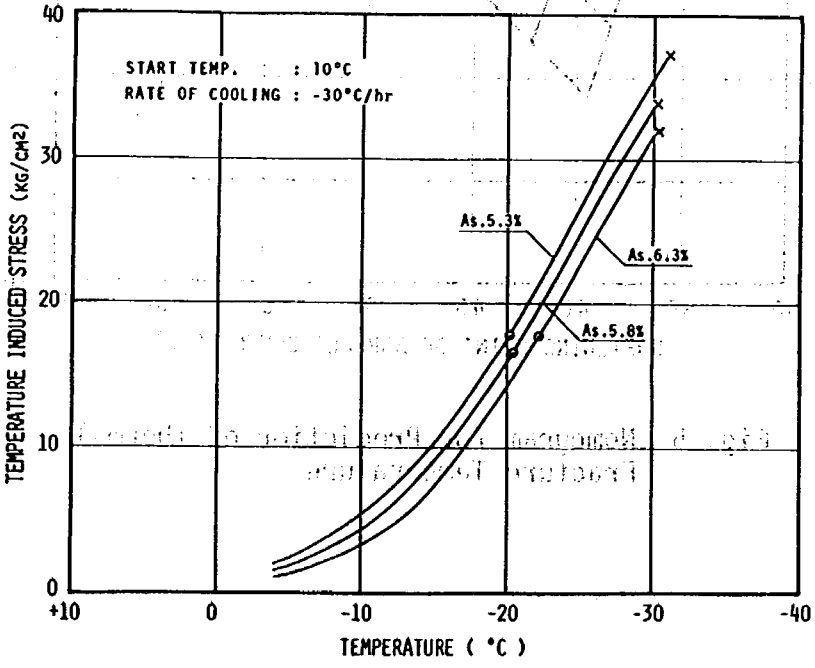

Fig. 7 Effect of Asphalt Content on Thermal Fracture

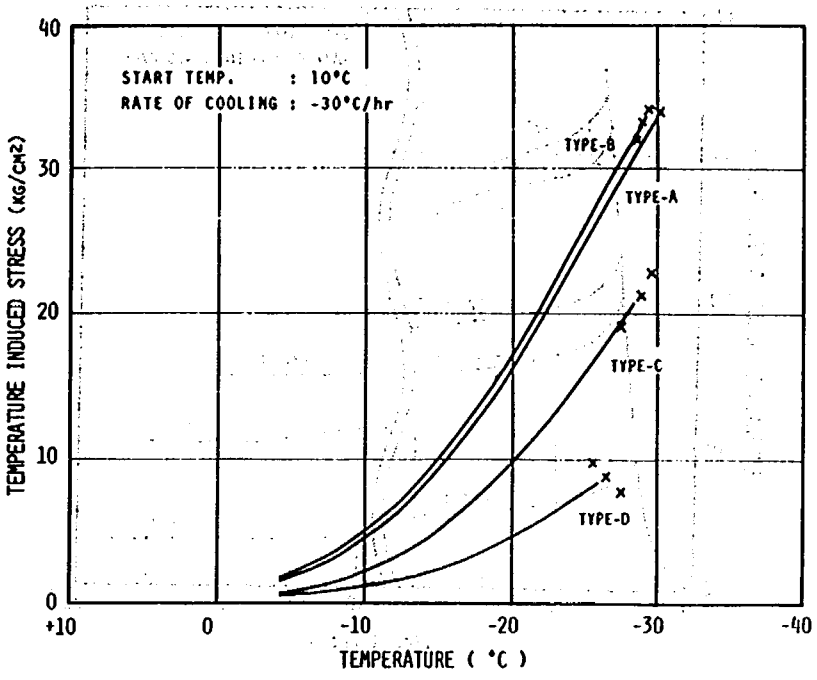

Fig. 8 Effect of Mixture Type on Thermal Fracture

	Case-1	Case-2	Case-3
Temp. Programming	10℃ → T_1 →	10℃ → T_1 →	10℃ → T_1 →
Starting Temp.	+10℃	+10℃	+10℃
Rate of Cooling	−30℃/hr	−12℃/hr	−12℃/hr
No. of Specimen	T_1 (℃)	T_1 (℃)	T_1 (℃)
	1 -14.5	6 -10.0	11 -15.0
	2 -20.5	7 -15.0	12 -20.0
	3 -22.0	8 -18.0	13 -25.0
	4 -26.0	9 -20.0	
	5 -27.5	10 -25.0	

Fig. 9 Mode of Temperature Change

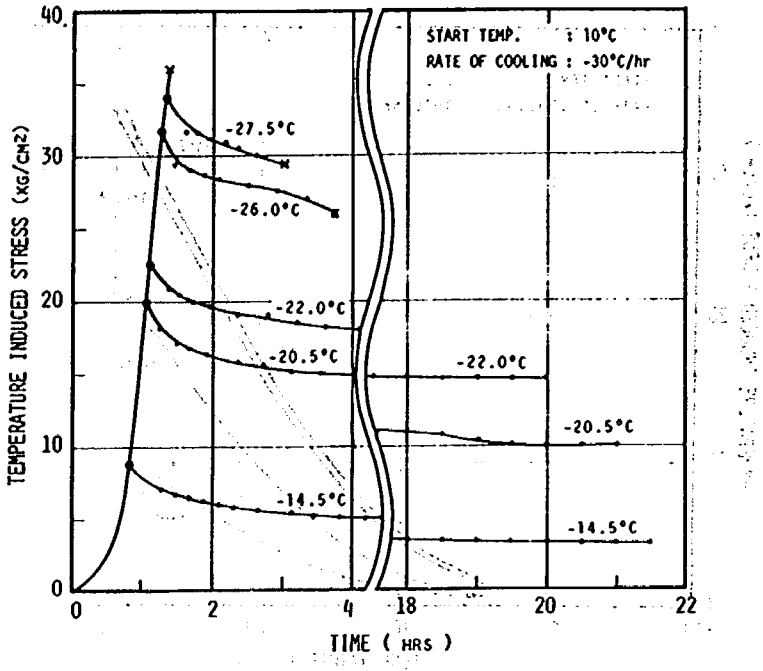

Fig.10 Change of Temperature Induced Stress at Constant Low Temperature

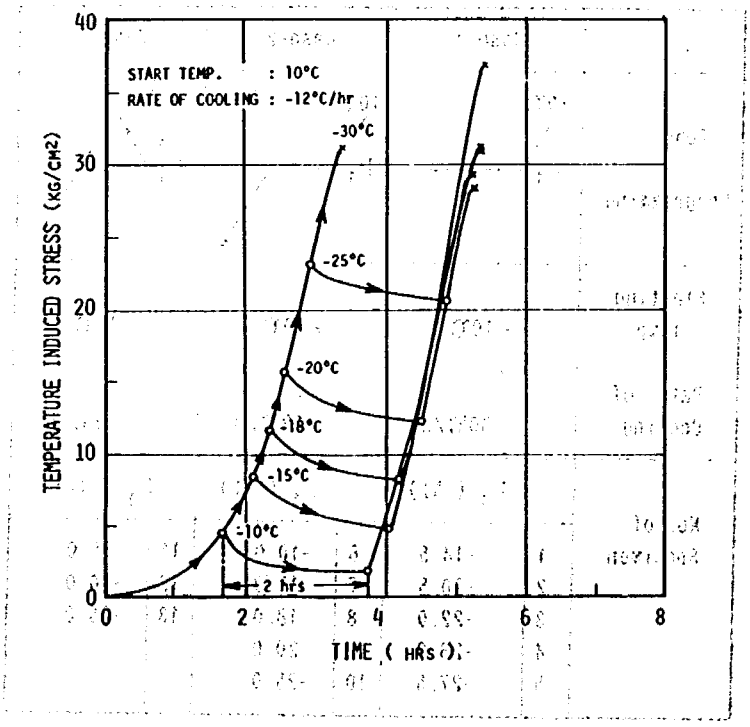

Fig.11 Change of Temperature Induced Stress at Second Cooling After 2 Hours Temperature Maintaining at Low Temperature

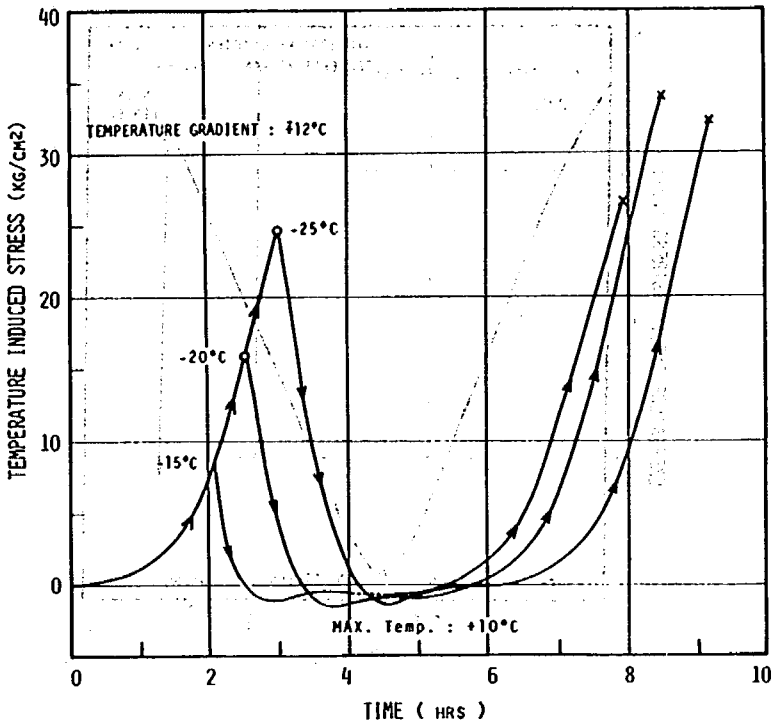

Fig.12 Change of Temperature Induced Stress Under Cyclic Temperature Change

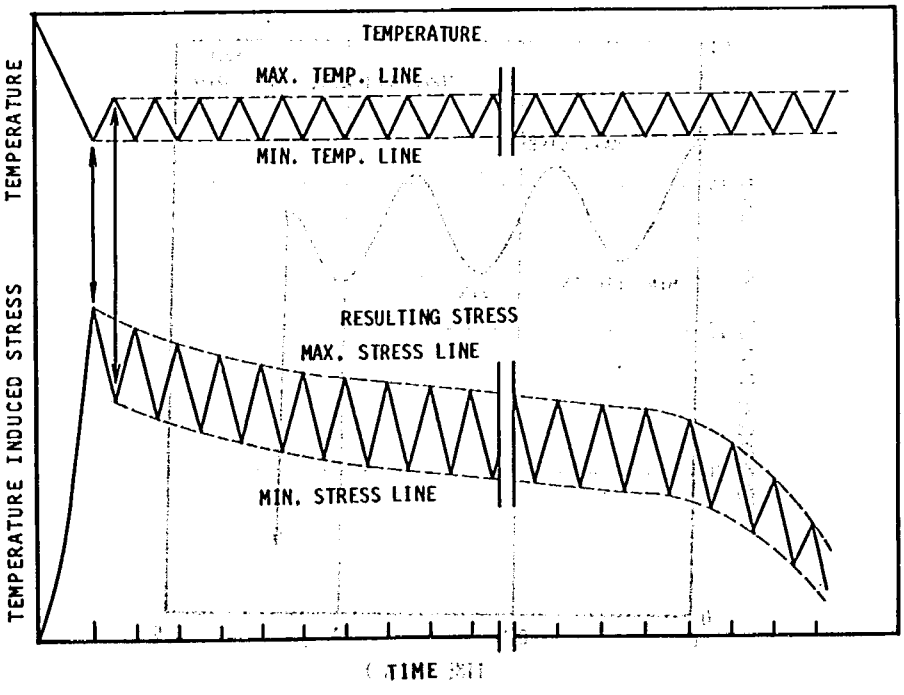

Fig.13 Schematic Diagram of Repeated Cooling and Resulting Temperature Induced Stress

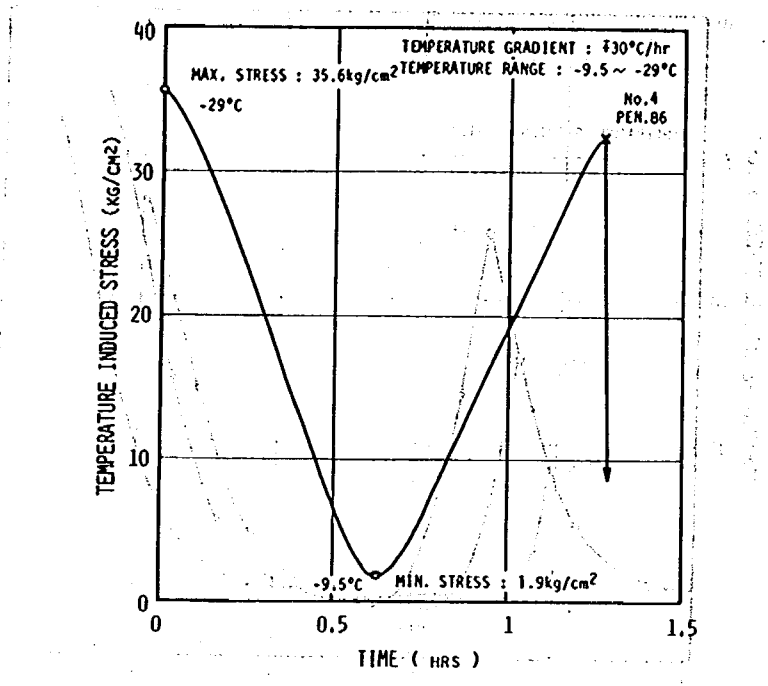

Fig.14.1 Thermal Fracture at Second Cooling (between -9.5 and -29°C)

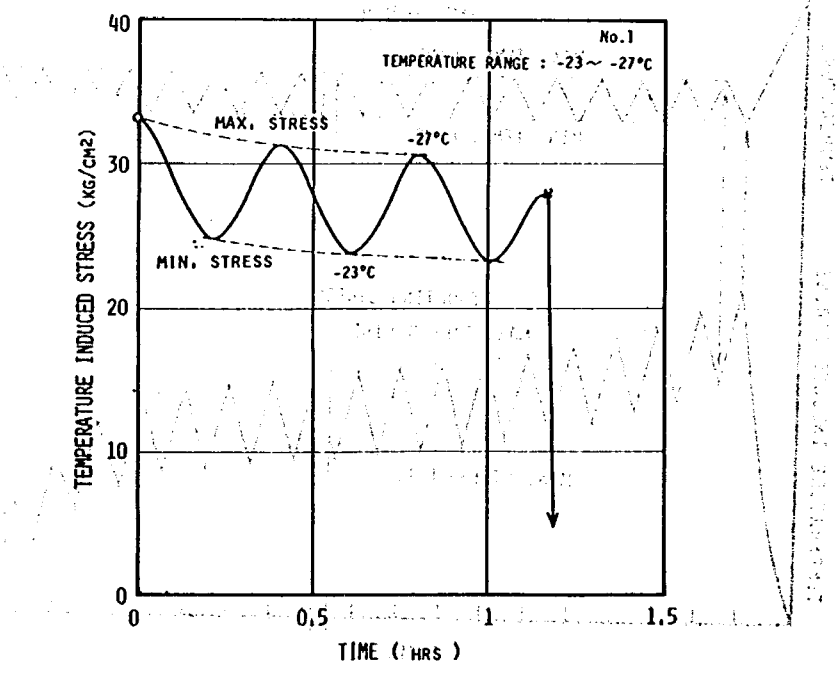

Fig.14.2 Thermal Fracture at Fourth Cooling (between -23 and -27°C)

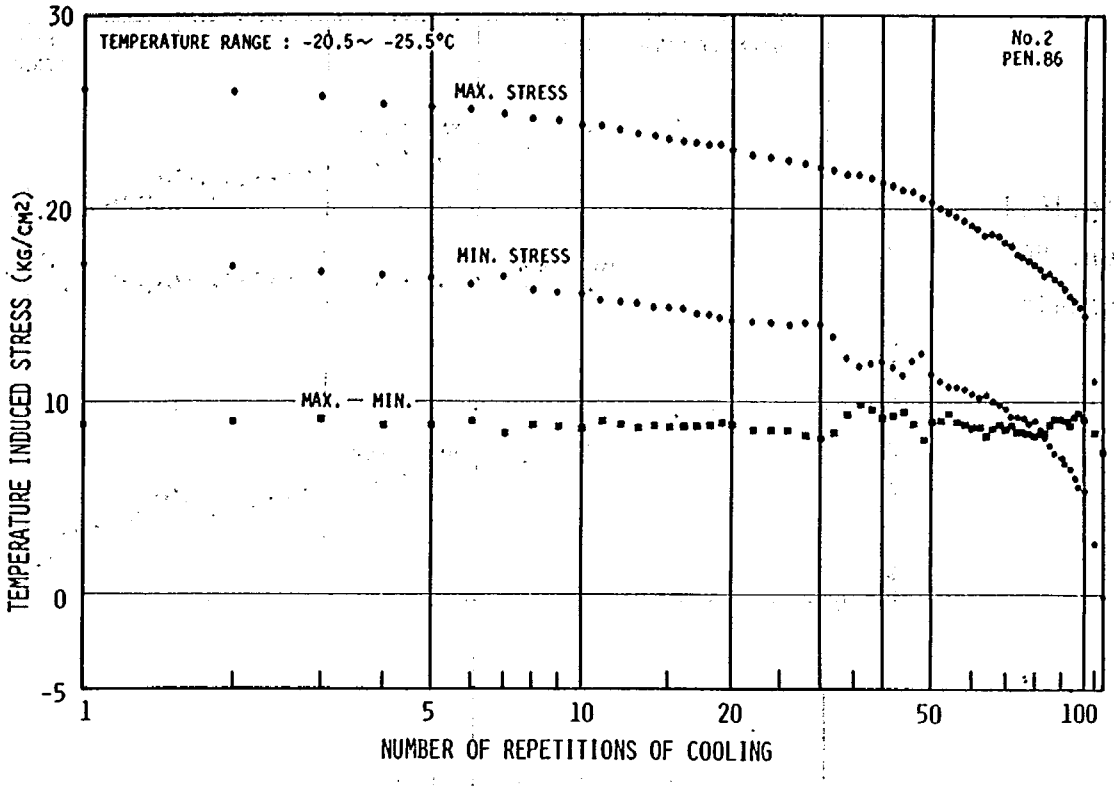

Fig.15.1 Change of Thermal Stress and Fracture under 120 Repetition of Cooling (between -20.5 and -25.5°C)

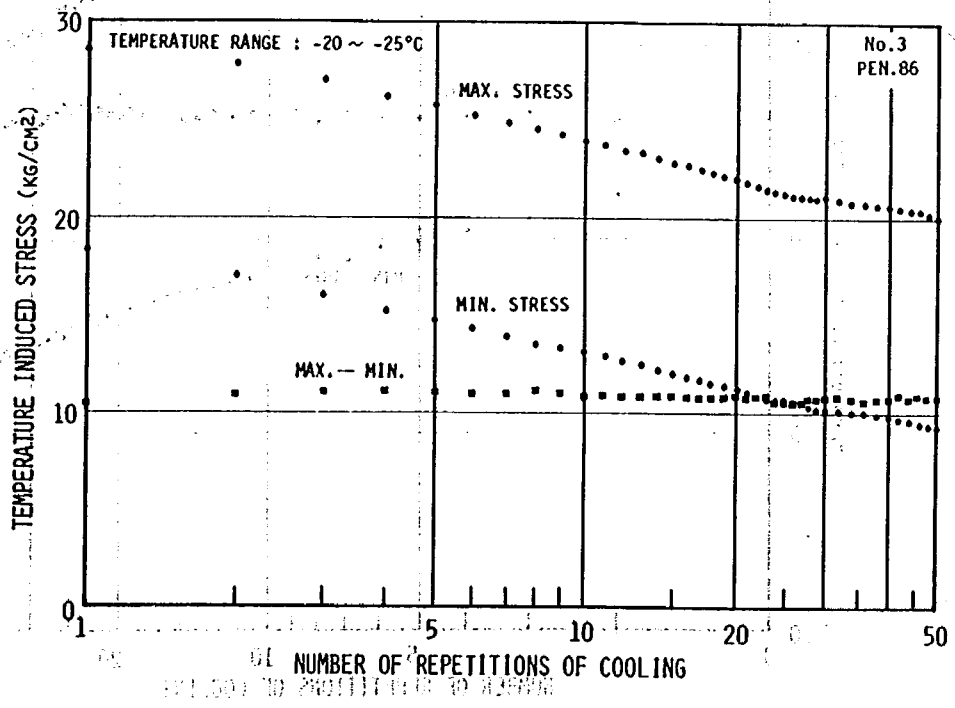

Fig.15.2 Change of Thermal Stress under Repeated Cooling (between -20 and -25°C)

Fig.16.1
Pen. 86
Asphalt

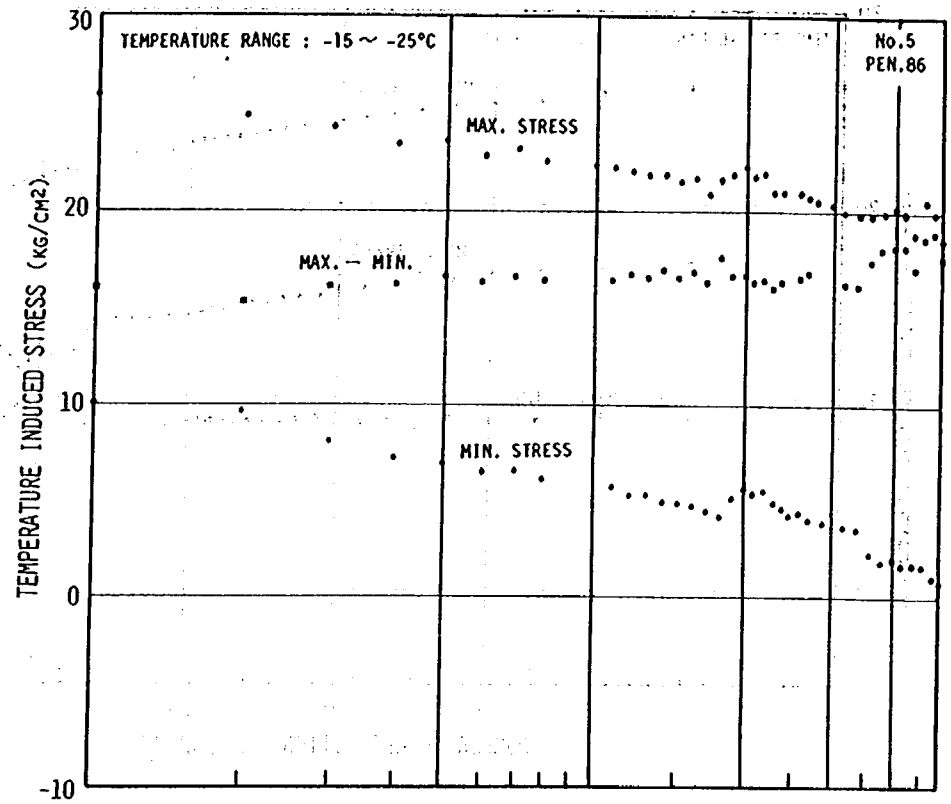

Fig.16.2
Pen. 60
Asphalt

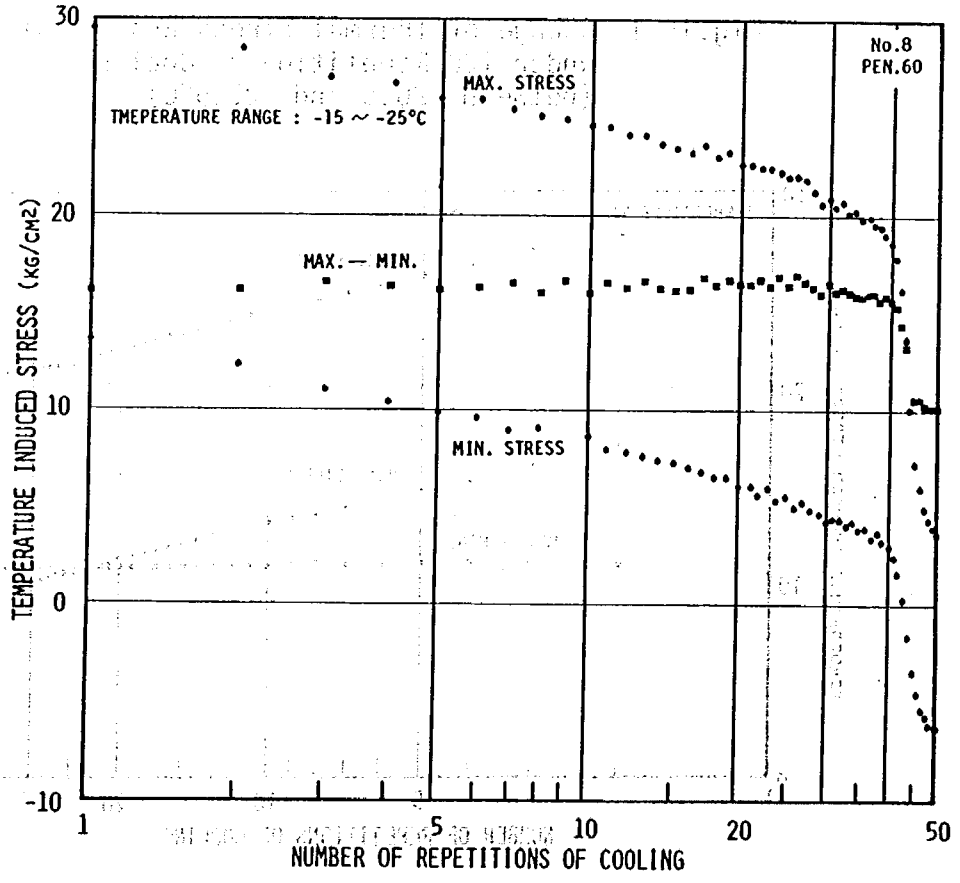

Fig.16 Change of Thermal Stress under Repeated Cooling for Pen. 86 and Pen. 60 Asphalt

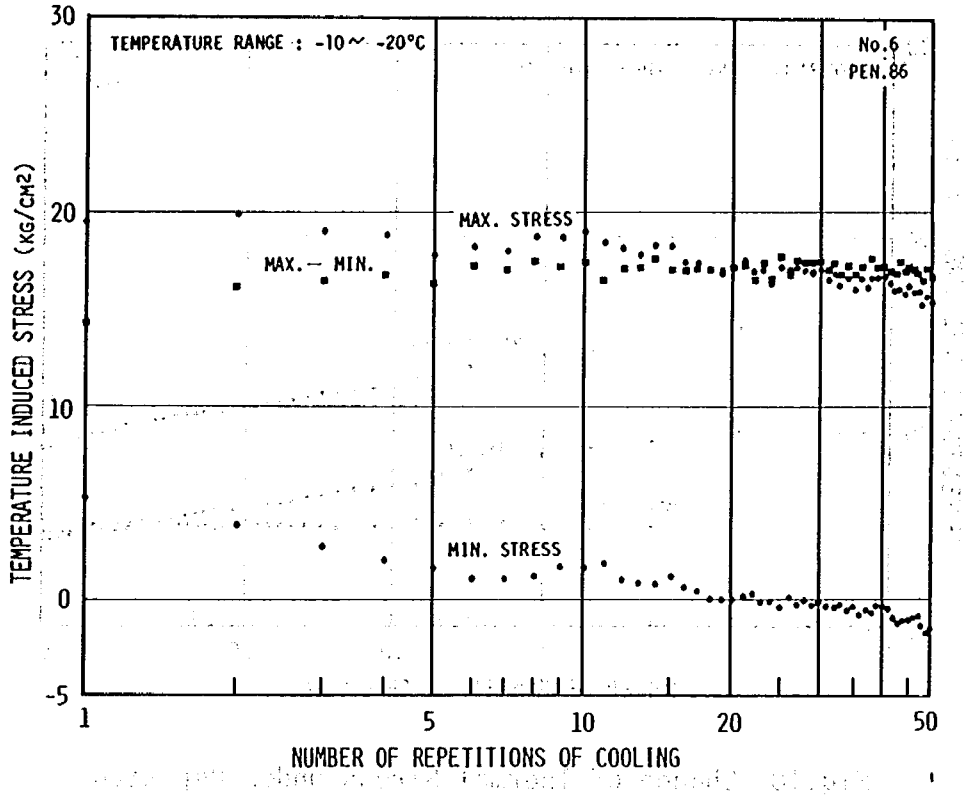

Fig.17.1 Change of Thermal Stress under Repeated Cooloing for Pen. 86 Asphalt (between -10 and -20°C)

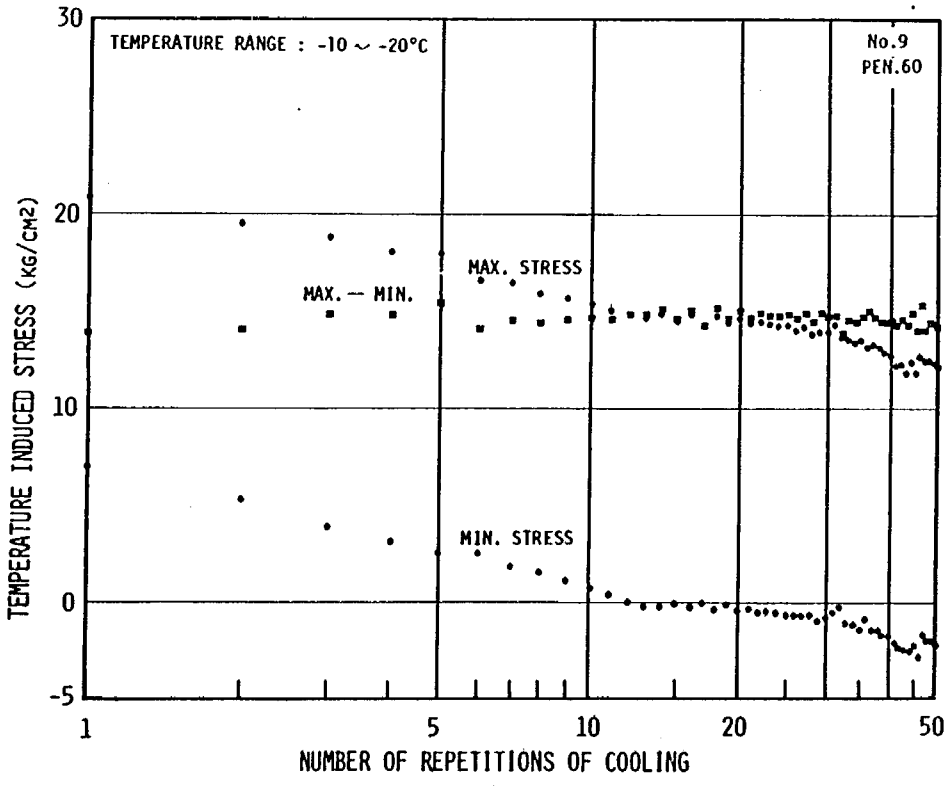

Fig.17.2 Change of Thermal Stress under Repeated Cooling for Pen. 60 Asphalt (between -10 and -20°C)

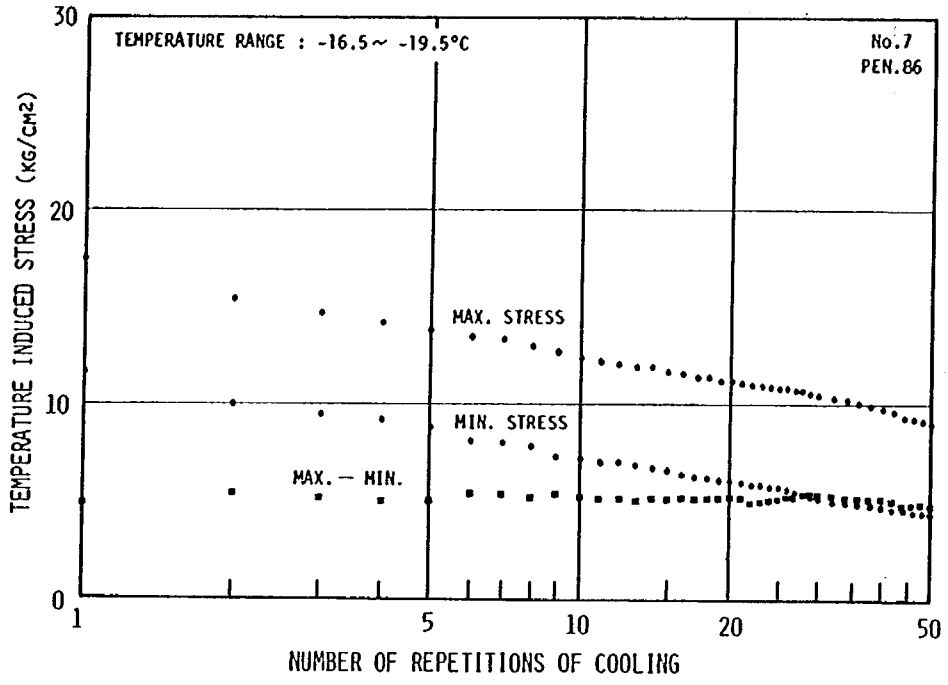

Fig.18 Change of Thermal Stress under Repeated Cooling (between -16.5 and -19.5°C)

Reflective Cracking in Bituminous Overlays in Quebec

Mr. R. Langlois
Quebec Ministry of Transportation

ABSTRACT

Many experiments were conducted on Quebec highways in order to find an economic solution to the problem of reflective cracking in bituminous overlays. This study gives a brief bibliography of the rupture mechanism, a summary of the experimental sections in Quebec, and presents the principal conclusions drawn from these experiments. A more detailed analysis of experiments on Route 161 is given as well as a general discussion comparing results of Quebec experiments with similar experimental sections in the United States and the province of Ontario.

From the results obtained to date, the following conclusions can be drawn:

An open graded mixture substantially reduces reflective cracking of the wearing course; a conventional repair mix overlay, 22,5 kg/m² is superior to heat scarification for reduction of reflective cracking; the addition of "Reclamite" to heat scarification improves this process with regard to reflective cracking in the first years but does not make it as good as a correction course; increasing the thickness of the open graded mix does not seem to appreciably improve behaviour of the wearing course to reflective cracking in the first years, but does appear more effective in the long term; a very good correlation exists between the rate of placing of an overlay and the number of reflected cracks per kilometer, regardless of the type of mix used.

REFLECTIVE CRACKING IN BITUMINOUS OVERLAYS IN QUEBEC

by

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Quebec Ministry of Transportation

1. Introduction

During the past several years, the Department of Transport of Quebec has renounced a budget mainly oriented toward highway construction in favour of a more substantial budget for highway maintenance. Problems of cracking of bituminous overlays require a significant share of this budget and of these, reflective cracks cause the most damage.

Reflective cracks are defined as fractures in an overlay which result from cracks or joints in the underlaying layer, and may be either environmental or traffic induced.

This report discusses some results of experimental sections made in Quebec to find an economic and effective method of preventing or reducing reflective cracking. To do so, a bibliography of the rupture mechanism is first discussed. Secondly, the first experimental sections which were conducted on Route 175 are reviewed and experimental sections on Route 161 are analyzed. Finally, a general discussion is presented which compares the Quebec results with similar experimental sections in United States and Ontario.

2. Rupture mechanism

Induced stresses in a material concentrate near a defect and constitute a source of crack propagation. The rupture mechanism recognizes that propagation of a crack may occur in three ways (see figure 1-A). Type 1 shows the opening of a crack by tension, in which the crack surfaces are displaced perpendicularly from one another. Mode II is a perpendicular shearing at the crack tip which causes its propagation and the crack surfaces are displaced in the same plane in a direction perpendicular to the crack tip. Mode III is a tear caused by shearing in a direction parallel to the crack tip and the crack surfaces are displaced in the same plane

in a direction parallel to the crack tip.

FIG. 1-A
Modes de propagation des fissures

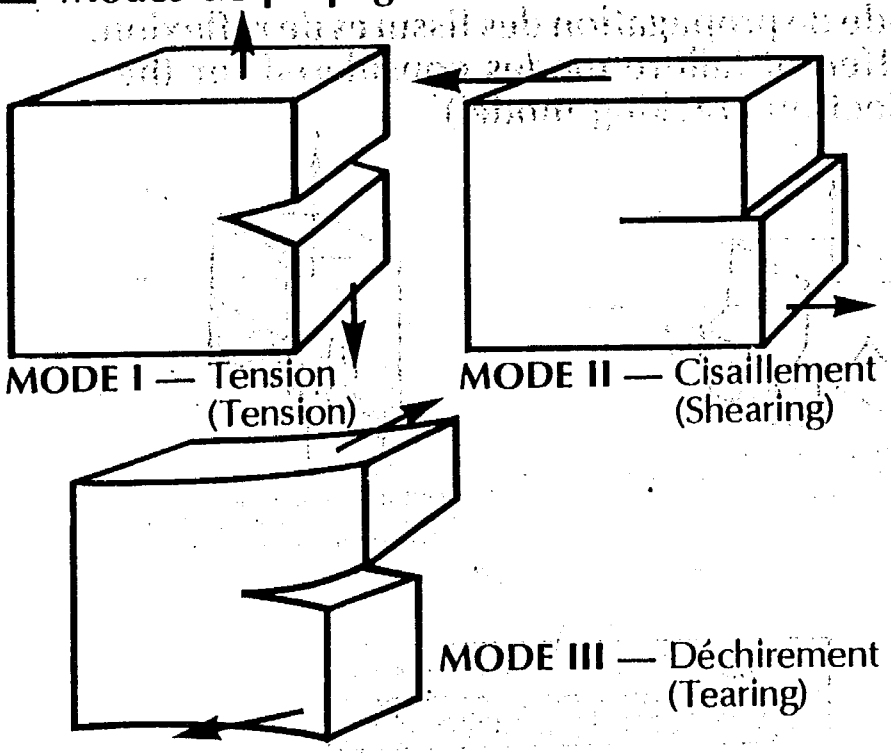


FIG. 1-A CRACK PROPAGATION MODE

In pavement maintenance, where too often a new bituminous overlay is placed over and old cracked overlay, the crack in the old overlay acts as the defect which concentrates stresses in the new surface course. Falling temperatures and heavy truck loading induce stresses and cause modes I and II propagation respectively (see figure 1-B). Mode III is practically non existent in bituminous overlays.

In hot climates, it is mostly vehicular loading which induces stresses and crack reflection in new course takes place according to mode II. However, in very cold climates such as Quebec winters, contraction of pavements resulting from falling temperatures induces significant stresses and cracks in old overlays reflect through new overlays, mostly according to mode I. This phenomenon is convincingly demonstrated in Quebec and

Canada where reflective cracking occurs almost exclusively in winter time.

FIGURE 1B
Mode de propagation des fissures de réflexion. (Different failure modes considered for the reflection cracking model).

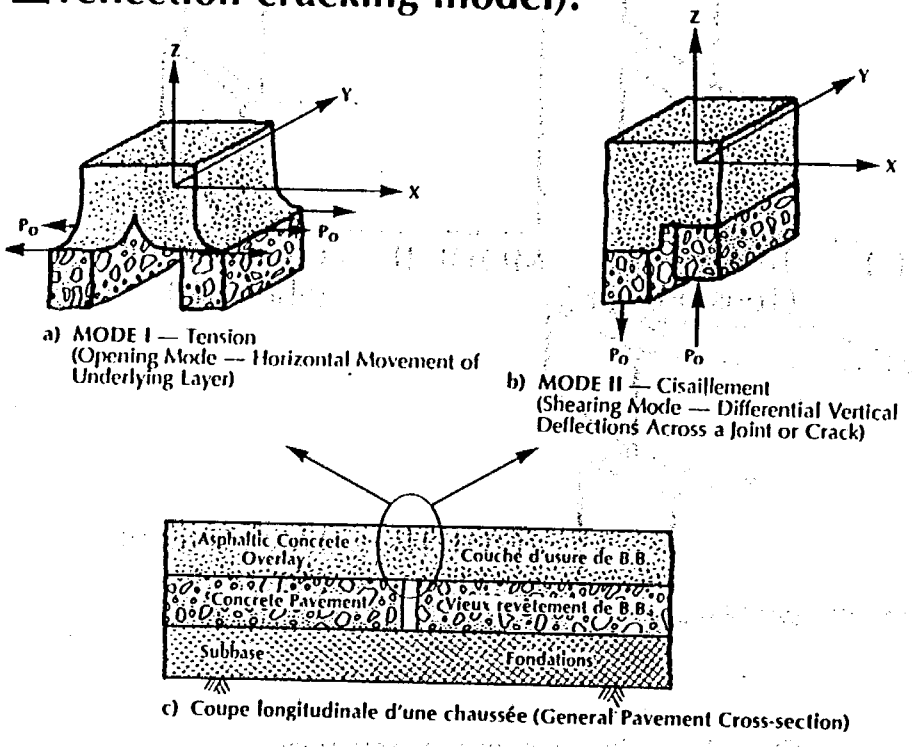


FIG. 1-B DIFFERENT FAILURE MODES CONSIDERED FOR THE REFLECTION CRACKING MODEL

Many studies (1,2,3,4,5,6) were conducted in the United States, but these mostly involved the effect of loading and mode II crack propagation. Another group of researchers directed their studies to the utilization of a stress release course, to minimize reflective cracking, with reinforcing cables (7,8), non embedded aggregates (9,10), rubberized membranes (11,12,13), geotextiles (14,15). Some research was also conducted on the modification of bituminous overlay mixes by the addition of reclaimed rubber (16) or fibre reinforcement (17). Meanwhile, the majority of studies were designed to prevent reflective cracking of joints in concrete through bituminous concrete overlays (18,19,20,21,22,23,24,25).

Furthermore, two studies embracing many experimental highway sections, involving 24 different procedures, were conducted in Arizona (26) and Colorado (27). Because of the different environmental conditions in Quebec, experimental highway sections were constructed to test the most promising procedures for the reduction of reflective cracking. These test sections were constructed on Routes 175 and 161.

3. Experimental sections of Route 175

This experiment was the object of a report by Mr. Pierre Demontigny (28). The article summarily describes the experimental conditions and the results obtained.

Route 175 crosses the Laurentide Park and links Quebec and Chicoutimi (see figure 2) since 1948. This route has an elevation of 750 metres approximately for the most part and is thus subjected to extremely severe climatic conditions due to very long winters and a frost which reaches a depth of 225 to 250 cm.

FIGURE 2
Localisation des sections expérimentales de la route 175

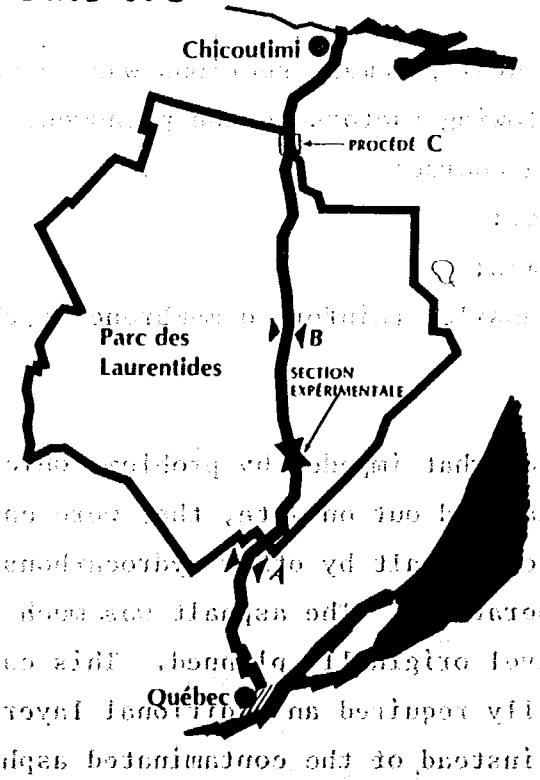


FIG. 2 LOCATION OF EXPERIMENTAL SECTIONS OF ROUTE 175

Traffic, a rigorous climate and the intensive use of de-icing salts caused cracking and spalling and the road had to finally be paved over in 1959 and 1964. A few years later, reflective cracks had already reappeared and new corrective methods had to be considered.

In 1973, a study that was carried out, based on a few visual observations and technical data from highway condition files, indicated that the load bearing characteristics were quite satisfactory but that there were numerous transversal cracks spaced about 2 to 3 metres apart which were frequently intersected by longitudinal cracks. Furthermore, at many locations, notably on slopes, abrupt surface protrusions near the crack edges accompanied by intermittent spalling, made driving quite uncomfortable at high speeds. Nevertheless, bumps of large amplitude were quite rare although the drainage was often inadequate which, notably at the shoulders adjacent to slopes, caused heaving that was more or less accentuated in winter with spalling of the overlays.

To test solutions for this severe problem, the area was chosen on a 10,8 km test section which crossed a hitherto horizontal area that was now a little wavy, which rested on a slightly variable sub-base. The bituminous overlay in this section was also severely fissured.

The purpose of the experimental pavement sections was to study the influence of each of the following factors on the phenomenon of crack reflection in the new wearing course:

- thickness of the layer;
- asbestos content in mix;
- presence of a high tensile, reinforced membrane at the base of the new overlay.

However, the program was somewhat impeded by problems which occured while the work was being carried out on site, that were caused by the unfortunate contamination of asphalt by other hydrocarbons at the refinery during the various operations. The asphalt was much softer than the 150-200 penetration level originally planned. This caused bleeding in many locations and finally required an additional layer, made this time with 150-200 asphalt instead of the contaminated asphalt previously

used. Table 1 describes each of the sections and their performance.

TABLE 1 - DESCRIPTION AND PERFORMANCE OF EXPERIMENTAL SECTIONS INSTALLED IN 1973

DISTANCE (KM) / SLAB NO.		0.00 — 1 — 1.33	2 — 2.46	3 — 3.49	4 — 4.55	5 — 5.78	6 — 6.91	7 — 7.88	8 — 10.81
TOTAL OVERLAY	SUPPLEMENTARY LAYER (67 KG/M²)								
	EXPERIMENTAL LAYER - KG/M² - ASBESTOS % - MEMBRANE	118 —	118 1	109 —	117 2	111 1	88 —	105 2	80 —
	REPAIR LAYER (KG/M²)	35	35	35	35	36	37	37	37
NO. OF CRACKS/(KM)	MARCH 1973	0.22 ← 301 → 1.82				5.02 ← 382 → 6.62 6.78		← 358 → 8.38	10.51 313
	MARCH 1974	59	81	98	48	35	86	50	80
	FEBRUARY 1976	107 | 60	121	142	54	57	114 | 183	101	172
	SEPTEMBER 1980	172 | 86	186	168	55	78	134 | 228		
MAYS SMOOTHNESS (CM/KM)	8 MARCH 1974	33	38	35	28	32	41	27	44
	16 OCTOBRE 1974	17	25	17	16	14	17	13	17
	31 JANUARY 1975	35	46	32	25	35	73	43	70
	12 FEBRUARY 1976	44 | 32	55	39	35	46	95 | 103	60	85
	25 MARCH 1980	45	52	55	40	52	— —	← PAVED IN 1978 →	

(ACCORDING TO REFERENCE 28)

The use of soft asphalt, unless it causes excessive instability due to ruts or ridges (which was not the case) can only be beneficial, especially for resistance to cracking. Nevertheless, all experimental sections cracked during the first winter and the number of cracks per kilometer varied from 35 (in the case of section No. 5) to 98 (in section No. 3).

These results prevented the use of either process on the most damages sections of the road. Moreover, in view of difficulties encountered during the installation of the overlay and the costs incurred, either by the addition of asbestos or use of the membrane, it appeared more advantageous to use conventional bituminous concrete.

In light of these facts, three other procedures (see figure 3) were evaluated: the first, procedure A, consisted of scarification and removal of existing overlay, followed by scarification of the granular layer to

a minimum depth of 15 cm to make it homogeneous and remove some of the stones which it might have contained before being recompacted, then adding to it, 15 cm of crushed gravel and 10 cm of bituminous concrete; the second, procedure B, consisted of covering the existing pavement with 20 cm of crushed gravel and 10 cm of bituminous concrete; the third, procedure C, was installed in a 3,7 km long section extremely cracked but without marked differential heaving in winter, and consisted of a 7,5 cm thick overlay of open graded mix, followed with a conventional bituminous concrete overlay.

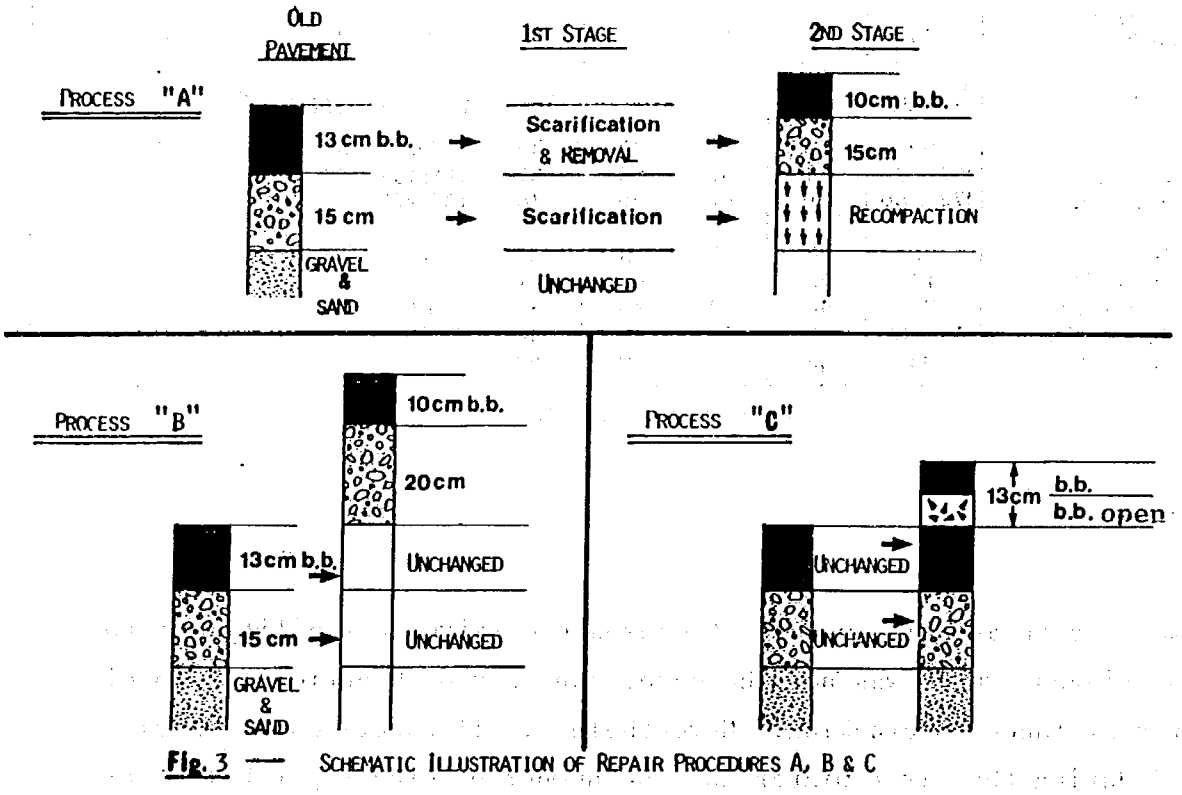


Fig. 3 — SCHEMATIC ILLUSTRATION OF REPAIR PROCEDURES A, B & C

(ACCORDING TO REFERENCE 28)

The open graded mix had to meet the following requirements: 100% on 50 mm sieve, 75-90% on 37 mm, 50-70% on 19 mm, 8-20% on 4,75 cm and 0-5% on 150 mm.

The use of such a mix constituted a first in Quebec and caused concern in the field, particularly on the part of the contractor who tended to use a gradation just bordering on fine limits. This mix was perhaps denser a bit more than was desirable to maximize prevention of crack

reflection.

Performance results for these three procedures are shown on Table 2 where the roughness and the cracking are compared with results of Section 4, the one with a membrane and additional overlay.

Table 2 - Comparison of several test sections repaired according to Procedures A, B or C and compared to Slab No. 4

	Date of Take-off	Results obtained according to procedure used			
		A (1974)	B (1974)	C (1974)	Slab 4 (1973)
No. of cracks/km	2 years old	33	14	———	49
	4 years old	38	16	24	54
	6 years old	77	27	36	55
Mays smoothness (cm/km)	Summer 1975	18	19	———	18
	1st winter	22	29	———	28
	1 year old (summer)	17	19	16	———
	3rd winter	35	34	27	35
	6th winter	46	38	33	40
Test section examined		5	5	1	1
Overall length (km)		11	19,8	3,75	1,0

(According to Reference 28)

The performance analysis for all experimental sections on Route 175 leads to the following conclusions:

1) The use of asbestos has not appreciably affected either the number of reflective surface cracks or the quality of pavement during subsequent winters.

2) The soft asphalt reduced reflective cracking by forming a buffer between the old cracked pavement and the new overlay.

3) A 20 cm thick layer of gravel over the old pavement (procedure B)

has for all intents and purposes prevented reflective cracking and has produced an overlay with the same performance as a new pavement. However, transversal cracks are generally of the multiple type, i.e. a cracked area 12 to 18 cm wide with two or three, more or less, parallel cracks.

4) Procedure A, used almost entirely on steep sloping sections and generally on variable soils including a major portion of rock cuts, seems generally inferior to procedure B and does not offer the same guarantee of success as the latter.

5) The thick bituminous concrete (procedure C) was very good. It even resisted cracking as well as procedure B. However, in winter, it gives rise to some differential in crack levels, even though these had been sealed about three years after the overlay was placed.

6) The influence of the reinforcing membrane used on sections 3 and 4 was very marginal with regard to degradation of surface smoothness and reflective cracking in time.

7) The very good performance recorded on sections 4 and 5 after 7 years - the actual number of cracks being seven times less than before the road was repaired, seems to indicate that a rich soft asphalt layer in between the old pavement and the new overlay would be one of the most viable solutions to the problem of reflective cracking.

4. Experimental sections of route 161

This project, which covers a distance of over 17,6 km on route 161 and links route 226 to Highway 20, was first divided into seven sections. Location diagram (see figure 4) shows the characteristics of the specific procedure used on each section. Visual inspection of each section, including establishment of levels and the counting of cracks at regular intervals during the experiment, has served to evaluate the performance and effectiveness of procedures used. Furthermore, rut depth measurements, the Mays Test and Benkelman Beam Test completed this evaluation.

It is easy to imagine the great variety of factors which can affect the performance and durability of the road. Bad performance results from weakness of the pavement, foundations or the infrastructure. On the

other hand, certain natural or indirect causes such as frost and climate, because of sudden variations of conditions from one season to the next or from one year to another, then too, air and sun, which contribute to oxydation and polymerization of asphalt thus making bituminous overlays less flexible, lead to road deterioration.

This study concentrates mostly on the effect of the old underlying pavement on the new wearing course and on the role of the intermediate procedure in the reduction of reflective cracking. The infrastructure, foundations and climatic conditions are considered identical for the purpose of this analysis.

FIGURE 4
Route 161 — Réhabilitation de la chaussée
Schéma de localisation

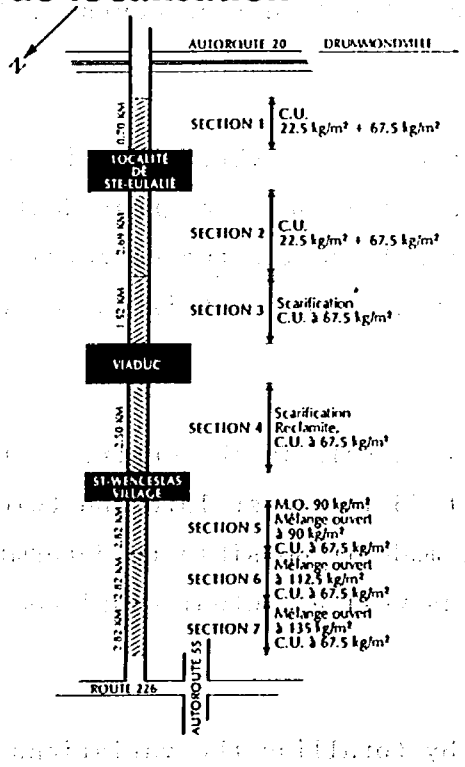


FIG. 4 - ROUTE 161 - PAVEMENT REHABILITATION
DIAGRAM OF AREA

This experimental project started in 1976 and was carried out to evaluate various procedures which would produce the most acceptable, long

lasting wearing surface at the lowest possible cost, while preventing or reducing reflective cracking.

An amount of 280 000,00$ was allocated for the actuel paving course on this project with an additional budget of approximately 100 000,00$ reserved for the experimental part. Table 3 shows the cost of each section per kilometer.

Table 3 - Cost of Work

Section No	1 and 2	3	4	5	6	7
* Cost/km	11,438	16,120	17,062	19,751	21,875	24,000

* The cost of bituminous binder is also included.

Table 4 shows characteristics of the wearing course and the open graded mix.

Route 161 is exposed to particularly severe climatic conditions due to long, cold winters. Frost usually causes a great deal of damage, and it is therefore necessary to closely follow the performance of each section in order to determine if heaving from frost is one of the main causes of failure.

Elevation readings at 60 m intervals in the centre of the road and on both sides, made by District 33 in August 1976 and twice a year since, either in spring or summer, made it possible to determine the total displacement of each section, as well as actual displacement compared to initial displacement.

Displacement is determined by totalling the variations of all elevation readings made since 1976, at each chaining. The values obtained for each section serve to determine its displacement compared to another section, which did not vary much from one section to another except for sections 1, 2 and 6, where they are slightly lower than the average. Average displacements vary from 0 to ± 5 cm but can reach ± 10 cm in some places. Thus it was noted that frost does not affect one section more than another.

Table 4 - Characteristics of Bituminous mixes

Characteristic	Type of Mix			
	Wearing Course		Open Graded Mix	
	Average	Standard deviation	Average	Standard deviation
Gradation				
Sieve 25 mm			100	
19 "			96	2,8
12,5 "	100		54	5,6
9,5 "	93	1,7	32	5,2
4,75 "	60	2,9	7	2,5
2,36 "	44	2,4	4	2,8
1,18 "	35	1,9	3	1,9
600 µm	27	1,7	3	1,2
300 "	15	1,1	2	1,2
150 "	7	0,8	2	1,2
75 "	3,8	0,65	1,2	0,48
Total Gradation	485	10,7	201	15,9
Bulk density	2,43	0,01	2,31	
Maximum density	2,47	0,01	2,59	0,02
Compaction (%)	95,8	1,13	74,2	3,13
Thickness (mm)	32	4,9	52,5	12,1
Bitumen %	5,55	0,25	2,30	0,35
Stability (N)	9600	885		
Flow (mm)	2,24	0,28		
Void %	1,8	0,64	25,8	3,13

Condition of the road prior to the study was evaluated by surveys made before work began. Mays tests (see Table 5) on the riding quality of the pavement demonstrated that section 2 was in good condition where as section 5, 6 and 7 were the most badly deteriorated. It is also noted that sections 1, 3 and 4 possessed about the same ride coefficient. The taking off of transversal cracks (Table 5) shows slightly different degradation. Sections 7 and 4 having the least number of cracks per kilometer and sections 2, 5, 3 and 1 follow in that order. Furthermore, the

degree of asphalt penetration of the old bituminous pavement varied between 24 and 30, thus indicating quite pronounced oxydation.

Table 5 - Condition of route 161 before evaluation

Section No.	1	2	3	4	5	6	7
Mays Test	33	21	28	30	47	42	44
Transversal cracks per km	213	152	174	139	163	182	138

Evaluation of the road revealed, when cracks were counted after the first winter, the presence of a more or less equal number of cracks on the three sections of open graded mix. On the other hand, marked reflective cracking was noted in the other sections in several different forms: transversal, longitudinal, lateral and alligatoring, however, it was the transversal cracks which were the most numerous.

Cracking

The evolution of transversal cracking is shown for each section in Tables 6 and 7 by the total number of transversal cracks, by length in kilometers and by the percentage of cracks which reappeared.

Table 6 - Evolution of transversal cracking
Route 161 from 1976 to 1983

Date of Take-off	* No. of transversal cracks per section/Average No. per km						
	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Section 7
77-03	44/63	195/72	200/132	199/80	46/16	49/17	38/13
78-11	47/67	205/76	280/184	345/138	79/28	54/19	49/17
79-03	100/143	293/109	302/199	412/165	118/42	87/31	60/21
80-03	101/144	313/116	363/239	486/194	110/39	110/39	89/32
82-04	146/209	353/151	358/236	632/253	461/163	351/124	262/93
83-05	192/274	432/161	356/234	638/255	501/178	391/139	267/95

* Transversal cracks including multiple transversal cracking and alligatoring.

Table 7 - Percentage of reflected transversal cracks

Route 161 from 1976 to 1983

	No. of transversal cracks before repair in 1976						
	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Section 7
	149	409	265	347	460	513	388
Date of Take-off	% of transversal cracks reflected at the time compared to the number of cracks prior to repairs						
77-03	30	48	75	57	10	9	10
78-11	32	50	106	99	17	11	13
79-03	67	72	114	119	26	17	15
80-03	68	77	137	140	24	21	23
82-04	98	86	135	182	100	68	68
83-05	129	106	134	184	109	76	69

It can be seen that sections with an open graded mix, base layer, had substantially reduced cracking during the first five years and subsequently, which increased inversely in proportion to the thickness of the overlay. In other words, the thicker the open graded mix, the less the reflective cracking.

Heat scarification, with or without Réclamite, covered with a conventional wearing course, did not give good results, whereas regular pavement gave passably better performance.

Figures 5 and 6 clearly illustrate these results and give a better idea of the evolution of cracking. They make it possible to determine which procedures provide the most effective control in prevention of reflective cracking. Conclusions drawn from the first four years' evaluation by Gilles Lavigne (29) vary just a little in comparison to their performance after 7 years. In effect, the thickness of the open graded layer seems to become a little more important. Meanwhile, it is evident that a certain limit is essential with regard to thickness of the open graded layer, since an additional quantity does not give better results. Preliminary repairs before applying the wearing course tends to delay the

start of cracking. Heat scarification does not always appear useful in correcting the cracking problem.

FIGURE 5
Fissures transversales réfléchies
Route 161 de 1976 à 1983

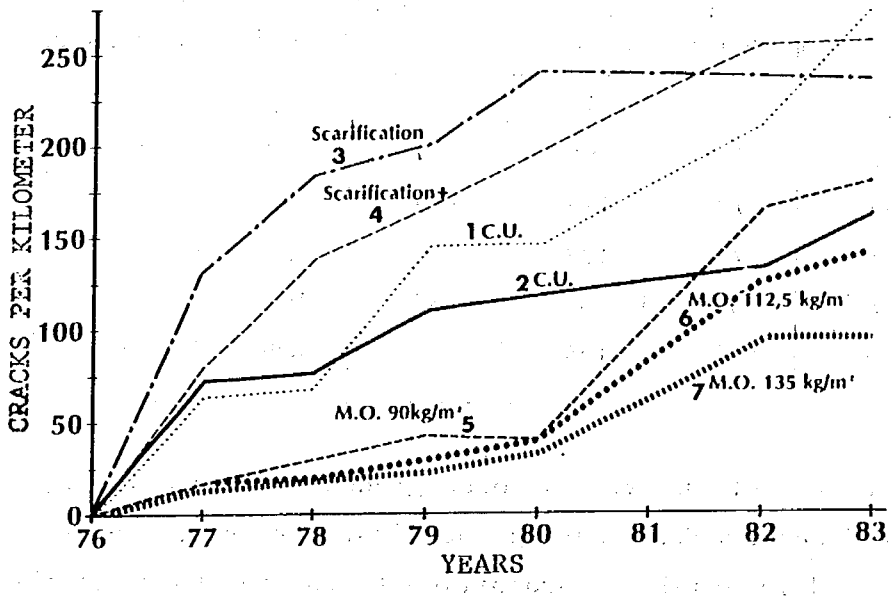


FIG. 5 REFLECTIVE TRANSVERSAL CRACKS
ROUTE 161 FROM 1976 TO 1983

In effect, fig. 6 shows that 100% of old pavement cracking has reflected through the wearing course after only 2 years in sections 3 and 4 where heat scarification was carried out. This occurred after 6 years in sections with a conventional corrective layer and 90 kg/m² of open graded mix, to reach 100% reflective cracking. As for sections with 112,5 kg/m² and 135 kg/m² of open graded mix, the percentage of reflective cracking is quite similar for both and is less than 80% after 7 years.

Rutting

Table 8 shows results of rutting measurements carried out in 1983. No measure had been taken before that year and thus the evolution of

rutting cannot be evaluated, but the performance of each of the sections after 7 years exposure to traffic is discussed.

FIGURE 6
Fissures transversales réfléchies
Route 161 de 1976 à 1983

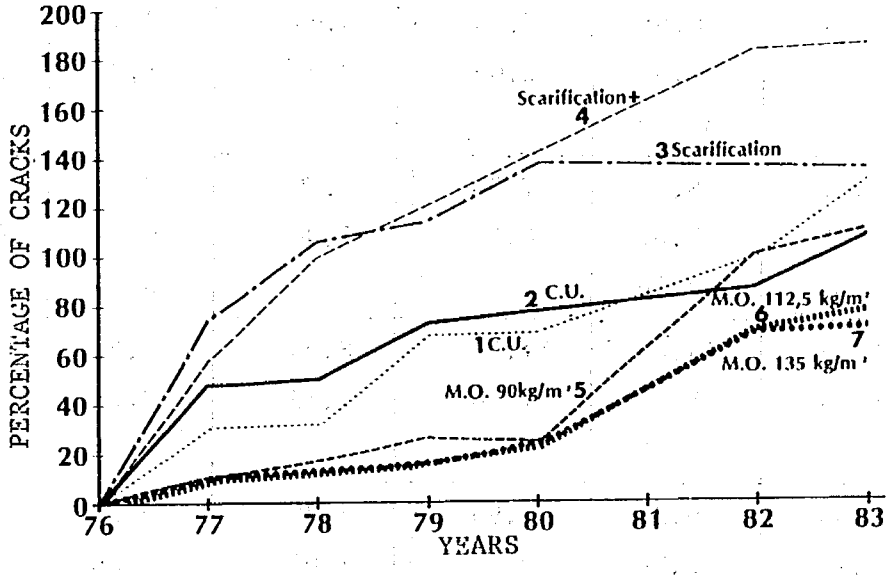


FIG. 6 TRANSVERSAL REFLECTIVE CRACKS
ROUTE 161 FROM 1976 TO 1983

Rutting is quite similar for all sections, although deformation of sections 1, 4 and 7 were slightly greater. For section 1, most likely, very severe previous rutting and inadequate correction course can explain this result. As for section 4, heat scarification probably failed to correct previous rutting and the spreading of Reclamite, followed with a wearing course, does not seem to have provided a satisfactory solution. In any event, measures taken recently on route 138 at Donnacona, clearly indicate that heat scarification only corrects 51% of the depth of the ruts or 5,23 mm of the 10,25 mm depth before treatment. In section 7, the thickness of the base layer, with an open graded mix, seems a little too high and this could cause premature and excessive rutting.

Table 8 - Rutting - Springtime 1983 - Route 161

Section	Rutting near shoulders (mm)*			Rutting near C/L (mm)*		
	20 cm G	Centre	20 cm D	20 cm G	Centre	20 cm D
1 μ	10,14	11,85	8,76	8,08	10,20	8,73
σ_n	4,18	5,39	4,71	1,89	1,64	1,49
		n= 19			n= 4	
2 μ	7,51	8,85	7,05	6,64	8,42	7,74
σ_n	1,87	2,37	2,09	0,96	1,03	1,03
		n= 19			n= 5	
3 μ	6,87	8,28	7,30	6,30	7,10	5,62
σ_n	5,16	7,62	6,62	1,00	0,52	0,91
		n= 18			n= 4	
4 μ	12,07	14,06	11,96	8,08	9,58	7,38
σ_n	6,94	8,22	7,16	1,89	2,08	1,35
		n= 16			n= 4	
5 μ	8,30	9,36	7,95	7,78	9,23	7,95
σ_n	2,12	2,36	2,25	1,01	0,86	1,03
		n= 16			n= 4	
6 μ	6,73	7,81	6,31	6,98	8,65	7,30
σ_n	2,87	2,92	2,98	0,80	0,96	1,66
		n= 16			n= 4	
7 μ	9,46	10,59	9,11	7,72	9,25	7,25
σ_n	3,87	4,00	3,60	1,98	2,18	1,77
		n= 16			n= 4	

* Rutting average for both directions

Load bearing capacity

The load bearing capacity of all sections was evaluated periodically by the Benkelman beam rebound test and the results appear in Table 9.

Generally, all these repair methods have not significantly increased load bearing capacity of the pavement, however, they were not chosen for this purpose but rather to reduce stresses transmitted from the old

pavement to the new wearing course. It would still have been preferable if the initial load bearing capacity had been higher in order to obtain improved performance after the repair of each section.

Table 9 - Benkelman Rebound

$(\overline{X}_C + 2\ \sigma)$ 0,01 mm

Section	June 1976 (before repair)	Nov. 76	Nov. 76	June 83
1	99	79	102	138
2	83	68	84	76
3	110	85	98	118
4	77	102	105	101
5	97	90	97	106
6	98	97	94	114
7	106	90	---	116

In other respects, it must be noted that the influence of temperature on load bearing capacity of open graded mixes is not necessarily the same as for dense mixes and corrections made to rebound measurements to compensate for temperature should be adjusted accordingly. It is perhaps this factor which distorts evaluation of the load bearing capacity of the sections.

Riding quality

The riding quality was evaluated with the Mays Rolometer. Table 10 shows the information collected from the seven sections since 1976.

Each of the repair methods chosen has certainly contributed to improve the riding quality, but the open graded mix gave the best performance as evidenced by results obtained for sections 5, 6 and 7.

5. General Discussion

In Arizona (26), heat scarification and the addition of 0,5 litre/m² of Reclamite has only allowed 7,4% reflective cracking on the 32 mm wearing course after 6 years. On the other hand, in Colorado (27), heat scarification and the addition of 0,7 litre/m² of Reclamite resulted in 100%

reflective cracking in the 50 mm wearing course after 5 years. In Quebec, the same procedure resulted in 100% reflective cracking of the 32 mm wearing course after only 2 years. This indicates that the colder the climate, the less effective heat scarification is for the prevention of reflective cracking.

Table 10 - *Mays riding quality in cm/km - Route 161

Section	June 1976 (before repair)	Oct. 76	Nov. 76	Feb. 77	Aug. 77	July 80	May 83
1	33	10	19	23	18	21	23
2	21	8	8	13	6	11	11
3	28	8	9	22	9	19	24
4	30	9	10	21	10	18	18
5	47	6	6	15	5	15	16
6	42	6	6	12	7	12	13
7	44	7	7	15	8	14	16

* Average riding quality for both directions at a speed of 65 km/hr. taken by the Division of Road Structures - Soils & Road Service

Petromat membrane gave excellent performance in Colorado (27) with only 2% reflective cracking in the 64 mm wearing course after 5 years. In Quebec (28), Petromat membrane did not perform so well: after 4 years, there was 47% reflective cracking in the 50 mm wearing course. Here again, a more severe climate in Quebec reduces the effectiveness of these procedures in preventing reflective cracking.

The standard method of using 50 mm of bituminous concrete, resulted in 74% reflective cracking after 5 years in Arizona, whereas in Quebec it amounted to 100% on Route 161 and 62% on Route 175 after 6 years. In this case, severe climate is compensated for by a softer asphalt which explains the very slight difference in the results.

The gravel sandwich method, Procedure B in fig. 3, used in Quebec gives results equivalent to those experienced at Trout Creek in Ontario (30) for transversal crack reflection.

FIGURE 7
Relation entre le taux de pose du revêtement et le nombre de fissures transversales réfléchies

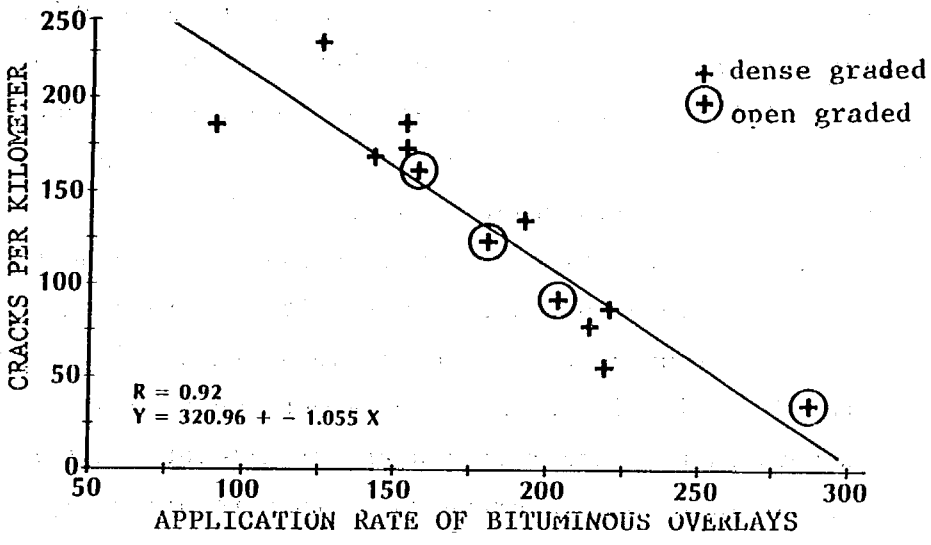


FIG. 7 RELATION BETWEEN THE APPLICATION RATE OF PAVING MIXES AND THE NUMBER TRANSVERSAL REFLECTIVE CRACKS

The use of an open graded mix to reduce reflective cracking was introduces by Tennessee in 1950 and developed by Arkansas eith the help of the Asphalt Institute (31); the mix is made with three different maximum rock sizes 76, 64 or 50 mm. Furthermore, it must be 90 mm thick and covered with a dense intermediate mix of 50 mm before the 40 mm thick wearing course is applied. According to Hinsley (32), this method has yielded excellent results in Tennessee during the last 23 years and in Arkansas for the last 10 years. In Quebec, open mixes are finer: the nominal sizes of aggregates used here are 37 mm on Route 175 and 19 mm on Route 161. Moreover, the intermediate layer is eliminated, which allows appreciable savings. Finally, open mixes were applied over old bituminous pavements, whereas in the United States they were applied

mostly on concrete. Ontario (30) started testing open mixes only in 1980, and their mix was slightly finer and denser than the one used in Quebec: in fact, the maximum size of aggregate is 16 mm; 5 to 15% of the aggregate passes a 4,75 mm sieve and 0 to 4% passes 75 mm sieve; they contain between 2,5 and 3,5% bitumen.

The effectiveness of open mixes to reduce reflective cracking is demonstrated not only in United States but also in Quebec, by the experimental sections of Routes 175 and 161 and by recent experience in Ontario. Meanwhile, since open mixes contribute to a thicker overlay on old pavements, it is important ot determine if they are effective because of their open gradation or simply because of the heavier application rate. Taking into consideration results of cracking on Route 175 and Route 161 and grouping them together in a graph (Fig. 7) according to the rate of application over old pavement, we find an excellent linear relation between the number of cracks per kilometer after 7 years and the application rate. Furthermore, this relationship does not seem independent of the type of mix, since both the dense and the open mixes from part of the same linear regression which has a very good coefficient of correlation of 0,92 and a close point dispersion.

In light of these facts, it appears that the only reason for the utilization of an open graded mix is to economize on bitumen; because its aggregates are often more expensive, its application, whitout disrupting traffic, more onerous due to deformation and stripping, its resistance to rutting is low and its greater thickness requires more expensive shoulders. Its is thus vital to optimize the gradation in view of the bitumen saving and reduced inconvenience. This would certainly bring about a modification towards a more dense gradation, even closer to that of a type MB-2 mix with maximum stone content.

In 1983, samples were taken on Route 161 in order to determine properties of the bitumen and to analyze the compaction evolution. Table 11 shows the results on reclaimed and original asphalt (in brackets).

The compaction of the wearing course had increased by 3,2% in 7 years going from 95,0 to 99,0%. This low void content of the pavement

explains the almost non existent oxidation of the wearing course despite its age of 7 years. The compaction factor of the open graded mix could not be evaluated because no measurements were taken at the time of construction. However, judging from its actual compaction of 74,2%, it is possible to state that it has not increases very much.

Table 11 - Properties of bitumen on route 161

Property of Bitumen	Wearing Course		Open-graded mix	
	$\bar{X}$	σ	$\bar{X}$	σ
Kinematic viscosity at 135°C (cSt)	(238,1) 426,4	(8,1) 46,9	(378,1) 824,3	(11,3) 63,6
Absolute viscosity at 60°C (Pa/sec)	(54,2) 212,0	(3,9) 40,1	(145,7) 1194,3	(9,1) 148,5
Penetration at 25°C	82,4 (170,6)	9,7 (4,5)	32,8 (90,8)	2,6 (2,9)

The cracking of a new pavement on a granular base is highly dependent on the type of bitumen contained in the mix as well as the environment, or in other words, the low temperatures to which it is exposed. Bituminous pavements installed on a granular base in Quebec in 1976 have produced the following number of transversal cracks per kilometer:

Area	Transversal cracks per kilometer
Montreal	88,8
Three Rivers	24,7
Quebec City	31,4
Rimouski	59.4
Sherbrooke	19,9
Chicoutimi	38,3
Average	43,8 in all

Since the average is 43,8 cracks per kilometer, a procedure which would not allow more than that number of reflective cracks after seven years would be completely effective. As shown by figure 8, only procedure of

section 7 would be as effective as a new construction in Montreal area.

FIGURE 8
Fissures transversales réfléchies
Route 161 de 1976 à 1983

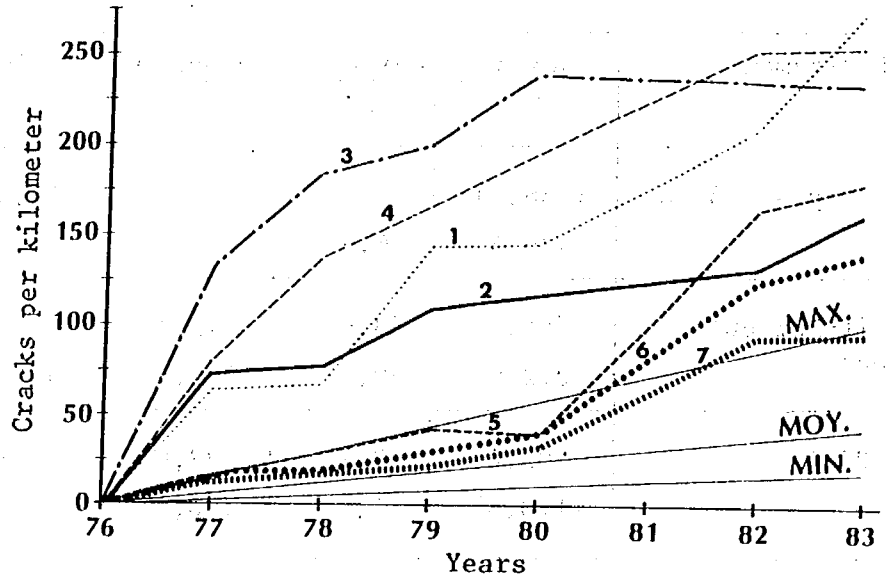


FIG. 8 REFLECTIVE TRANSVERSE CRACKS

6. Conclusion

Even though it is risky to generalize conclusions for conditions other than those we have experienced, the present data, drawn from the bibliography and from experiments in Quebec, permit us to do so with a certain assurance. Therefore, the following conclusions can be used for similar conditions or those which differ slightly from the experiments referred to herein:

1) Heat scarification and the addition of Reclamite is ineffective for reduction of reflective cracking in cold winter climates such as in Quebec.

2) An open graded mix, sandwiched between old pavement and a new wearing course is very effective if it is placed at an adequate application

rate of not less than 112,5 kg/m².

3) The application rate is more important than the gradation of the mix for reducing reflective cracking.

4) Without a doubt, the most effective mix would be an MB-2 with maximum stone content. This would provide optimum economy, placing and long term performance.

5) The granular sandwich method is very effective against reflective cracking but substantially increases the highway profile and causes placing problems in situations where work must be carried out without detouring traffic.

6) A very good correlation between application rate of the overlay and the number of reflected cracks per kilometer was established and can be used as a guide in the selection of the pavement thickness and type (see figure 9).

FIGURE 9
Relation entre le taux de pose du revêtement et le nombre de fissures transversales réfléchies

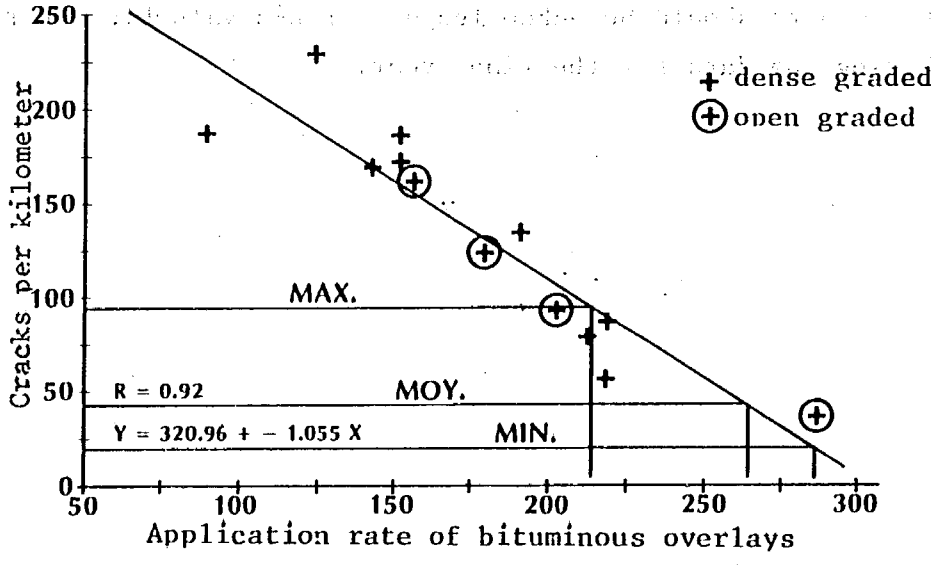


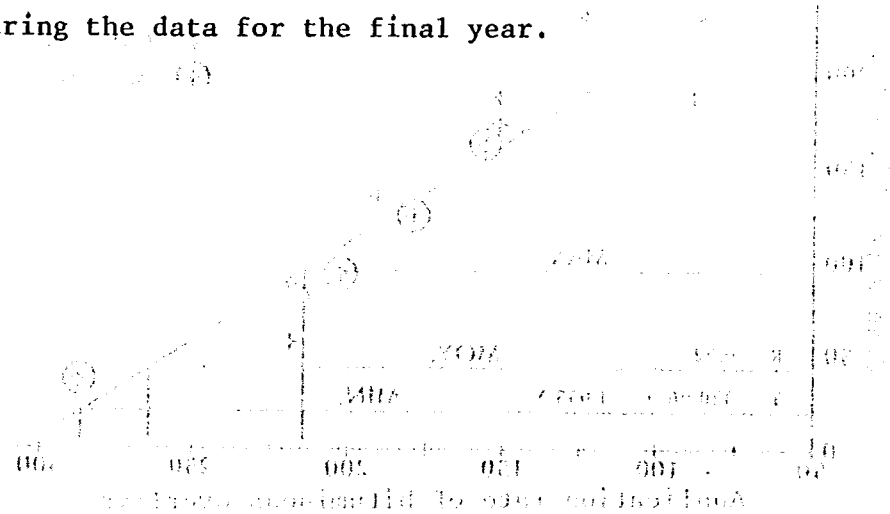
FIG. 9 APPLICATION RATE OF THE PAVEMENT RELATED TO THE NUMBER OF REFLECTIVE TRANSVERSE CRACKS

7) The conventional method using thin corrective and wearing courses is without doubt the best solution for highways where the damages are not too significant and the riding quality is not badly deteriorated. This technique could be even more effective if it were completed with filling any cracks which reappear with a rubberized material.

To summarize, there is no miracle solution to the problem of reflective cracking. However, optimization of selected procedures, commensurate with placing cost and long term performance should be made for each application. Research should continue to better understand the mechanism of reflective cracking in cold climates and also to optimize the choice of maintenance procedures.

ACKNOWLEDGMENTS

The author wish to thank Mr. Pierre Demontigny and his group for their valuable assistance in measuring the bearing capacity and evenness of the road surfaces. Mr. Gérard Moreau also deserves thanks for having drawn the figures by mini-computer. The author also wish to acknowledge Miss Francine Guillot valuable work of typing the text of this report. Special thanks to District 33 and their technician, Mr. Jean-Claude Trudel, for taking off elevations and crack measurements. Finally, Mr. Gilles Desrochers should be acknowledged for his valuable contribution in gathering the data for the final year.



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Applicability of Fracture Mechanics to Reflection Cracking

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ABSTRACT

Assuming a reasonable bond strength between overlay and cracked pavement, reflection crack growth may be predicted by means of fracture mechanics application.

This paper presents the applicability of elastic-plastic fracture mechanics, as well as linear fracture mechanics, in predicting the reflection cracking of asphaltic concrete. An experimental investigations of compliance methode was conducted in order to determine fracture mechanics characteristics of temperature-varied asphaltic concrete.

The results indicate:

- the applicability of linear fracture mechanics may not be applied both in ductile fracture stage and in brittle fracture stage.
- the applicability of elastic-plastic fracture mechanics seems to be adequate particularly in the brittle fracture stage.

APPLICABILITY OF FRACTURE MECHANICS TO REFLECTION CRACKING

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INTRODUCTION

It is well known that, because of the internal defects of materials, the fracture of brittle materials may occur with relatively lower levels than that of experimental or theoretical predictions. The brittle fracture of notched materials due to unstable crack propagations is one of the typical phenomena in fracture mechanics. The fracture mechanics principles have been applied to the solution of pavement cracking problems by a number of investigators [1,2]. These applications are based primarily on linear elastic fracture mechanics with using the stress intensity factor.

In order to apply the fracture mechanics to asphaltic concrete structures such as the overlays or resurface, the fractural properties of the material must be clearly evaluated.

In this paper,therefore, the main physical properties of asphaltic concrete concerning the fracture mechanics are discussed as follows;

1) Notch sensitivity - the application of linear fracture mechanics to a given material is useful only as long as the material is notch-sensitive. A material is consiered notch-sensitive if the net stress at failure of a notched section decreases with increasing notch depth.

2) Fracture toughness - the (critical) stress intensity factor of linear fracture mechanics parameters is called as fracture toughness and the crack-tip stress field may be described using this parameters.

3) J-integral value - J-value is the one of the parameters which describe the stress field of near crack-tip of non linear fracture mechanics.

FRACTURE MECHANICS APPLICATION

Assuming a reasonable bond strength between the overlay and the existing pavement, a stress field near a crack-tip of the cracked pavement will depend on the mode of fracture as shown in Figure 1. Von Quintus et.al.[3]

quoted that the major cause of reflection cracking is considered of holizontal movement as a result of expansion and contraction of the existing pavement from temperature changes. This fracture phenomena will be described as an opening mode failure (mode I in Figure 1).

In this paper, we deal with this mode I as a representative phenomena of reflection cracking.

For a single-edge-notch specimen in three point bending, KIC is calculated using the expression Brown and Srawley [4]:

$$K_{IC} = \frac{6\ Mc}{B\ W^2}\sqrt{\pi a}\ \ F(a/W) \qquad (1)$$

where F(a/W) is the finite width correction factor, obtained from the expression [4]

$$F(a/W) = 1.090 - 1.735\ (a/W) + 8.20\ (a/W) - 14.18\ (a/W) + 14.57\ (a/W)\ ,$$

a is the crack length, W is the specimen depth, Mc is the ultimate bending moment, B is the specimen depth and l is the span.

KIC(1) is one-parameter description of the stress field near the crack-tip. This calculation assumes a linear elastic materials, which is homogenous and isotropic. Although these assumptions are really incorrect for asphaltic concrete, investigators [1,2] have generally assumed that the approximations involved in the application of linear elastic fracture mechanics to this materials are reasonable.

The J-integral, a one-parameter average measure of non linear elastic field near the crack-tip, is investigated to overcome this problems. The path-independent J-integral was proposed by Rice [5] where the value of J in expression (2) depends upon the stress field near the crack-tip:

$$J = \int_p [\ Uody - Ti\ (\ \partial Ui/\partial x)\ ds\ ] \qquad (2)$$

where p denotes a counterclockwise path surrounding the crack-tip, Uo is the strain energy density given by

$$Uo = \int_0^{\varepsilon_{ij}} \sigma_{ij}\, d\varepsilon_{ij}$$

and Ti and Ui are the surface traction and displacement vectors, respectively.

For a single-edge-notch specimen in pure bending, the equation of the J-integral is given by [6]

$$J = \frac{2}{W - a} \int^{\theta crack} M d\theta crack \qquad (3)$$

where M is the bending moment per unit thickness and θcrack is the angle change due to the crack.

For a specimen in three point bending, this becomes [7]

$$J = \frac{2}{B\,(W - a)} \int^{\delta crack} P d\delta_{crack} \qquad (4)$$

where P is the applied load, B is the specimen thickness and δcrack is the vertical displacement of the load point due to the crack.

The critical value of the J integral, JIC, defined as the J value at the initiation of the crack extension is [8]

$$J_{IC} = \frac{2}{B\,(W - a)} \int^{\delta_c} P d\delta_c \qquad (5)$$

where δ_c is the vertical displacement of the maximum load point.

The value of JIC can be converted to equivalent plane strain stress intensity factor, KIC(2):

$$K_{IC}(2) = \sqrt{\frac{J_{IC}\, E}{1 - \nu^2}} \qquad (6)$$

where ν is Poisson's ratio and E is the Young's modulus.

KIC(2) for an elastic-plastic stress field near crack-tip may be modified as [9]:

$$K_{IC}(2) = \sigma\sqrt{\pi(a + K_{IC}^2 / (6\pi \sigma_y^2))} \qquad (7)$$

where σ_y is the yield stress.

EXPERIMENTAL DETAILS

Materials and Preparation of Specimens

Specimens were made of a dense-graded asphaltic concrete. The mix details of the specimens are all reported in reference [10]. The specimens were grooved at mid-span with a 1.0 mm wide diamond saw to produce a ratio of notch depth to specimens depth of 0.25 and 0.5 respectively.

Testing Procedure

Three specimens were tested for each notch depth and for each temperature in three point bending as shown in Figure 2. The testing was performed in instron testing machine at a rate of 100 mm per. min. The load-deflection curve was obtained throughout testing by an X-Y plotter.

TEST RESULTS AND DISCUSSION

The ultimate flexural strength, fa, the relative flexural strength, fa/fo, the critical stress intensity factor, KIC(1), the estimate critical stress intensity factor from JIC, KIC(2), and the critical value of J-integral are given in Table 1.

Notch Sensitivity

The notch sensitivity of asphaltic concrete with temperature varied is shown in Figure 3. This figure shows that the relative flexural strength, which is calculated by dividing the net stress at failure of a notched section by the ultimate flexural strength, is almost constant with increasing the relative notch depth at each temperature except near the transition point (i.e. 10 to 15 °C). This results indicates that asphaltic concrete seems to be notch-insensitive materials.

Linear Fracture Toughness,KIC(1)

Figure 4 shows the relationship between linear fracture toughness KIC(1) and temperature of the specimen. According to Figure 4, the value of KIC(1) is constant below the temperature of transition point (in this case, 10 to 15 °C). In the region up from the transition point, however, the value of KIC(1) decreases in proportion to the temperature.

The value of KIC(1) increases with increasing the relative notch depth. This is consistent with the data presented by Ramsamooj [1] or Brown [11], but not with that of Hillemeien and Hilsdorf [12].

Critical J-value, JIC

The critical value of J integral was obtained from the calculation of equation (5). Figure 5 shows the relationship between the JIC value and temperature. The effect of notch depth on the JIC value is not significant in this figure. According to Figure 5, there is a significant transition point which divides two different stages of fracture characteristics. In the region of ductile fracture stage, the JIC value ranges from 600 N/m to 800 N/m. In the region of brittle fracture, on the other hand, the JIC value ranges from 200 N/m to 400 N/m. This value in the brittle fracture region, is about a half of that in the ductile fracture region.

Refer to this JIC value of the other materials, epoxy resin (with the tension strength 6.9 Mpa) [13] , cement concrete and fiber reinforced polymer cement concrete [14] are 130 N/m, 45 N/m and 500 to 1000 N/m respectively.

Estimated Stress Intensity Factor, KIC(2)

The values of KIC(2), calculated from the equation (6), were shown in Figure 6. In this figure,a transition point is clearly distinguished. In the ductile fracture stage, KIC(2) deceases significantly with increasing the temperature. In the brittle fracture stage, on the other hand, KIC(2) increases with inceasing the temperature.

CONCLUSIONS

On the basis of the experimental investigation described above, the following conclusions may be drown:

(a) Asphaltic concrete is generally notch insensitive materials as well as cement concrete materials.

(b) The experimental value for KIC(1) indicates that within the size range studied, there is a significant dependence of linear fracture toughness upon the relative notch depth. On the contrary, the value for KIC(2) appears to be a notch independent. This implies that the application of linear fracture mechanics to reflection cracking seems to be inadequate, and that of elastic-plastic fracture mechanics appears to be applicable.

(c) JIC appears to be a promising fracture criterion for an asphaltic concrete.

(d) In case that mechanical behaviour of the reflection cracking is descrid as the displacement of opening mode (mode I), it seems to be possible to predict the stress field near the crack tip of pavement structures using the combination of two parameters, JIC and KIC(2).

(e) Preventions of the reflection cracking in fracture mechanics aspect can be enumalated as follows:

- to select materials of high-ranged JIC value
- to blunt the crack tip in order to increase the value of critical stress intensity factor.

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TABLE 1 - Flexural strength and fracture parameters

Temperature (°C)	Notch Depth (mm)	Flexural Strength (Mpa)	(fa/fo)*	KIC(1) ($N/mm^{1.5}$)	KIC(2) ($N/mm^{1.5}$)	JIC (N/m)
	0	3.07	(1.00)	-	-	-
25	20	2.96	(0.96)	22.85	20.90	893.18
	40	3.35	(1.09)	51.04	17.54	629.34
	0	4.86	(1.00)	-	-	-
20	20	5.26	(1.09)	41.56	19.21	630.93
	40	5.83	(1.20)	94.20	19.98	682.24
	0	8.27	(1.00)	-	-	-
15	20	9.16	(1.11)	71.57	36.22	899.31
	40	9.53	(1.15)	151.48	36.40	908.16
12.5	20	-		87.34	-	-
	40	-		148.91	29.16	210.05
	0	13.13	(1.00)	-	-	-
10	20	8.24	(0.63)	62.74	33.53	461.56
	40	9.52	(0.73)	147.35	25.11	258.88
7.5	20	-		73.22	-	-
	40	-		141.67	31.56	189.89
	0	10.50	(1.00)	-	-	-
5	20	9.65	(0.92)	73.25	31.81	356.93
	40	10.09	(0.96)	155.95	30.30	323.86
	0	8.60	(1.00)	-	-	-
0	20	9.08	(1.06)	69.54	27.94	245.10
	40	11.19	(1.30)	178.41	34.52	374.00
	0	8.33	(1.00)	-	-	-
-5	20	8.70	(1.04)	71.56	24.59	202.96
	40	9.80	(1.18)	143.72	25.97	226.49

* relative flexural strength

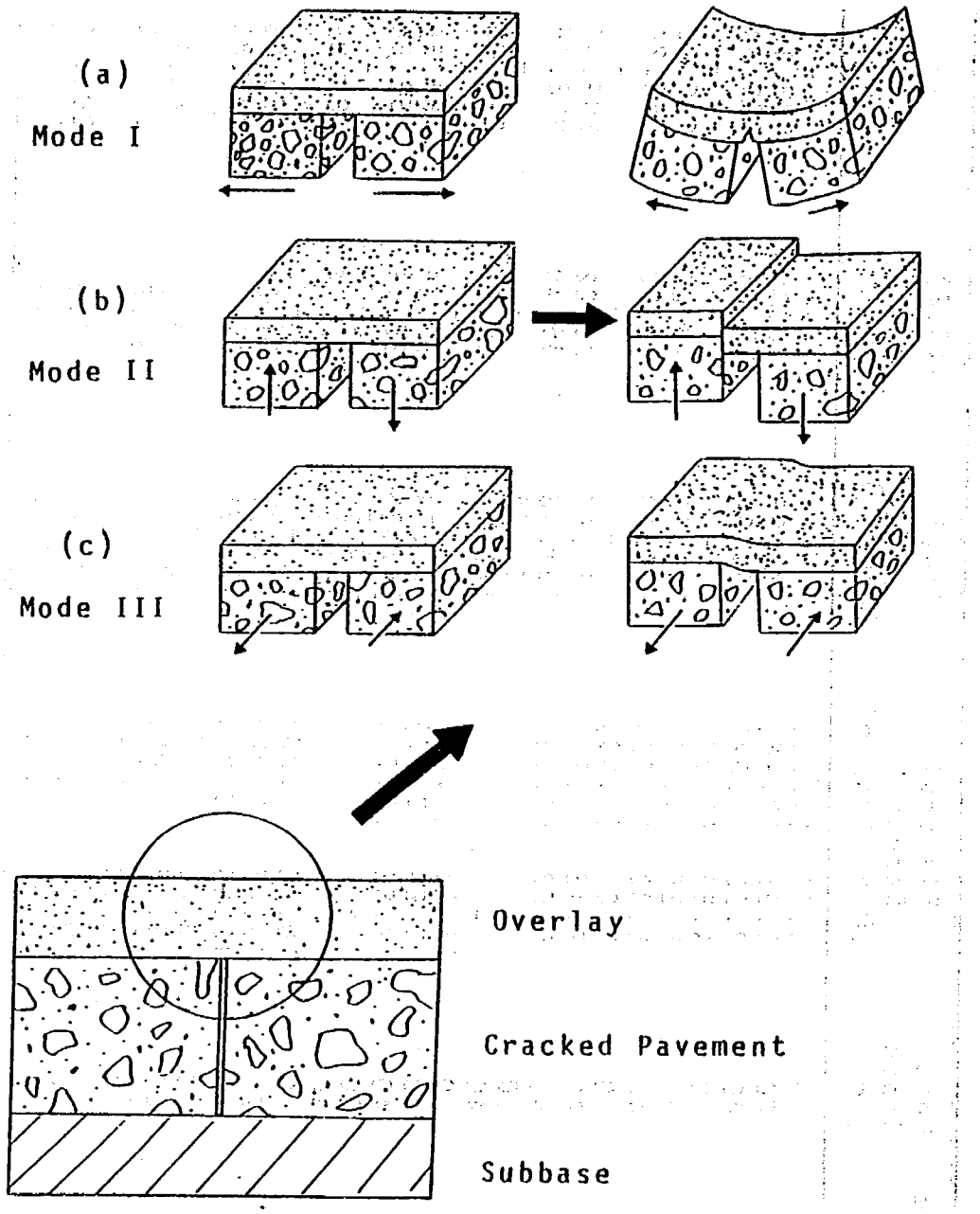

FIG.1 - Three modes of displacement considered for the reflection cracking model

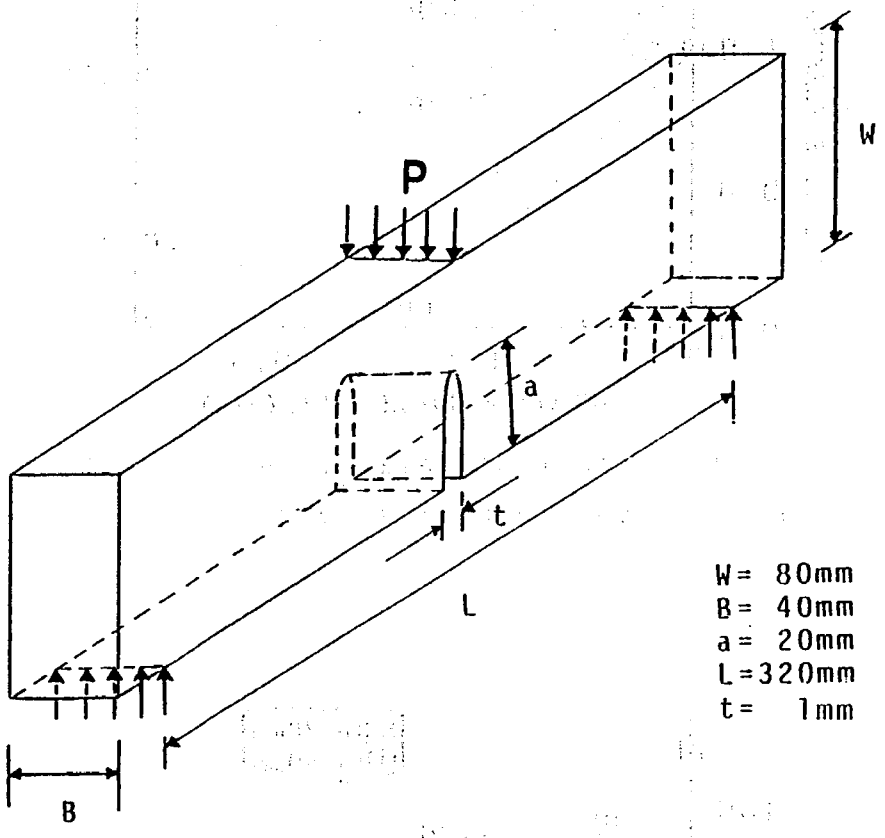

FIG.2 - Flexural test specimen

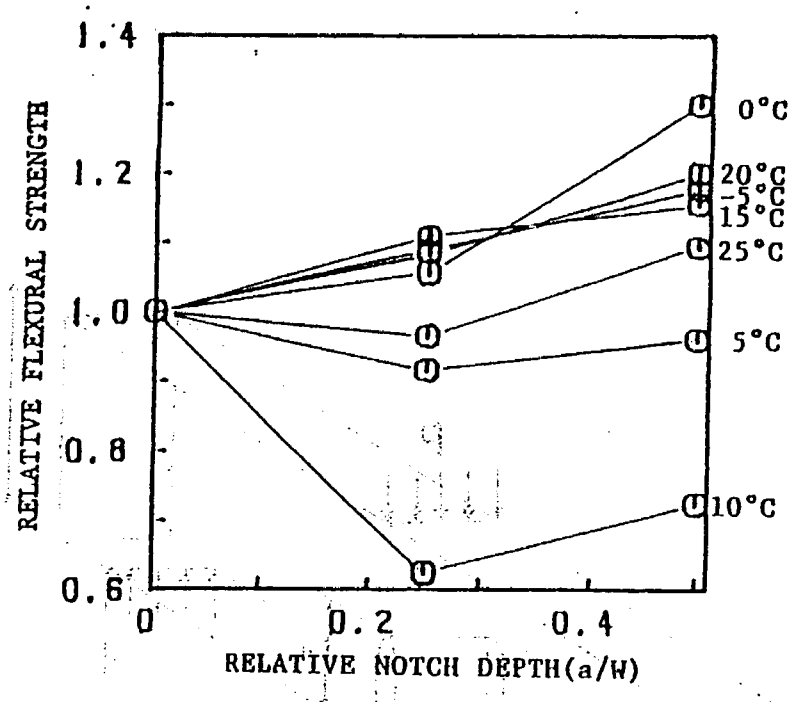

FIG.3 - Effect of relative notch depth on relative flexural strength

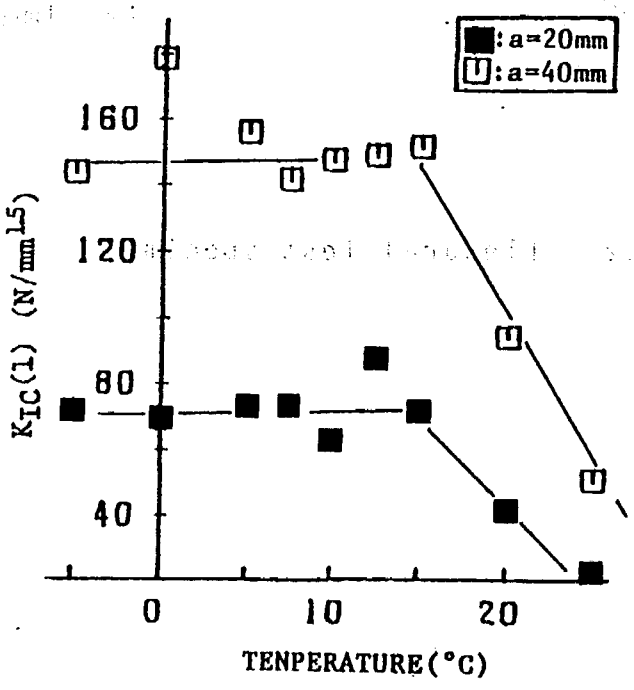

FIG.4 - Effect of temperature on critical stress intensity factor, $K_{IC}(1)$

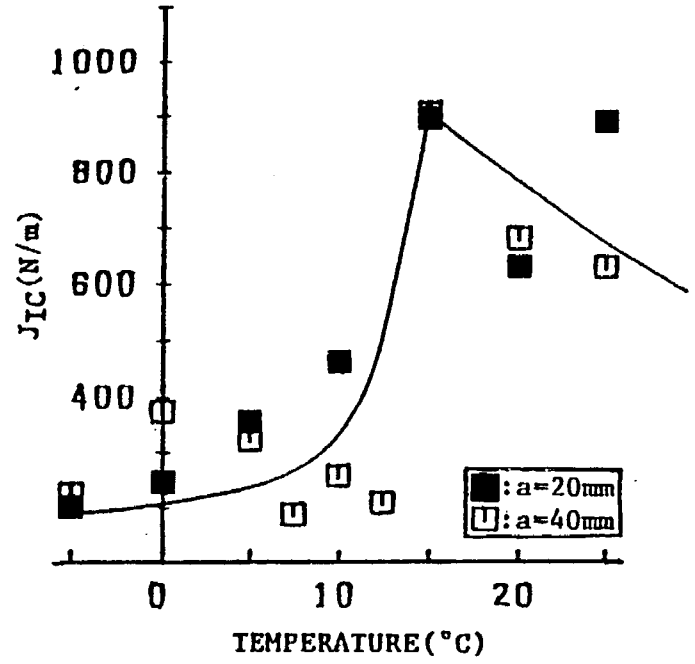

FIG.5 - Effect of temperature on J_{IC}-value

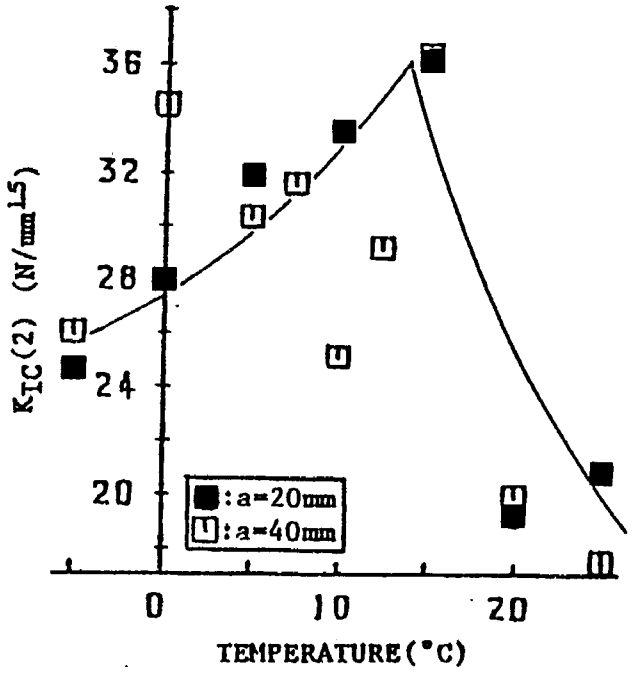

FIG.6 - Effect of temperature on critical stress intensity factor, $K_{IC}(2)$

Failure Behavior of Notched Asphalt Concrete

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FAILURE BEHAVIOR OF NOTCHED ASPHALT CONCRETE

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1. Introduction

Reflection cracks are fractures in an overlay or surface caurse that are a result of, and reflect, the crack or joint pattern in the underlying layer and may be either environmental or traffic induced. The effective preventions against reflection cracking have not been obtained, because a well developed and generally accepted description of the mechanisms and response variables associated with reflection cracking have not been established.

Reflection cracking was recognized as one of the principal forms of distress in resurfaced pavement at the 1932 annual meeting of the highway Research Board. This led to a great deal of experimentation of various techniques for the control of reflection cracking. Then reflection cracking was again recognized as being a major problem at a workshop on pavement rehabilitation in 1974 [1]. Obviously, the problem had not been solved. Recent work has shown that reflection cracking can be mitigated by three strategies-recycling the cracked pavement prior to overlay which eliminates the cracks [2], mix design such as use of softer asphalts in the overlay mix [3] and use of interlayers placed on the pavement prior to receiving an asphalt overlay [4,5].

The resurfacing course or an overlay above cracks and joints is dealt with as a notched asphalt mixture layer, because the overlay completely adheres to the existing pavement. It seems that the tensile strength of the asphalt concrete is one-half of the flexural strength, so the maximum value of the tensile strength is approximately 6 Mpa. The stress concentrating effect at the crack may cause the notched asphalt to have smaller tensil strength or strain-at-rupture than these of unnotched one.

Flexural strength, strain-at-rupture and temperature of transition point of the notched asphalt concrete were measured by means of a constant rate three point flexural test rangeing from -5°C to 25°C. The failure behavior of the notched asphalt concrete will be discussed by using a failure envelope in the case of the growth of the cracking through the overlay.

2. Method of Experiment

2.1 Type of Mixture and Specimens

The asphalt mixture prepared for the flexural test is a dense-graded asphalt concrete. The composition of the asphalt concrete and the characteristics of asphalt binder are shown in Table 1. The optimum asphalt content of the asphalt concrete is 5.3% by weight.

The dimension of specimen was 80mm in height, 40mm in width and 350mm in length. The specimens which have 20mm or 40mm of notch depth and 1mm of notch width were prepared for the flexural test. The average value of the degree of compaction of the prepared specimens was 99% based on Marshall density and the standard deviation was 0.92%. Figure 1 shows the distribution of the degree of compaction of specimens for flexural test. The degree of compaction of all specimens ranged from 97% to 101%.

Table 1/ Composition of Asphalt Concrete and Characteristics of Asphalt

Sieve opening (mm)	Persent passing	Asphalt
20	100.0	Straight Asphalt
13	98.8	60/80
5	64.2	
2.5	42.3	Pen.(25°C,100g,5sec)
0.6	26.9	: 67
0.3	15.8	T R&B : 49.0°C
0.15	8.9	P.I. : -0.9
0.074	5.8	

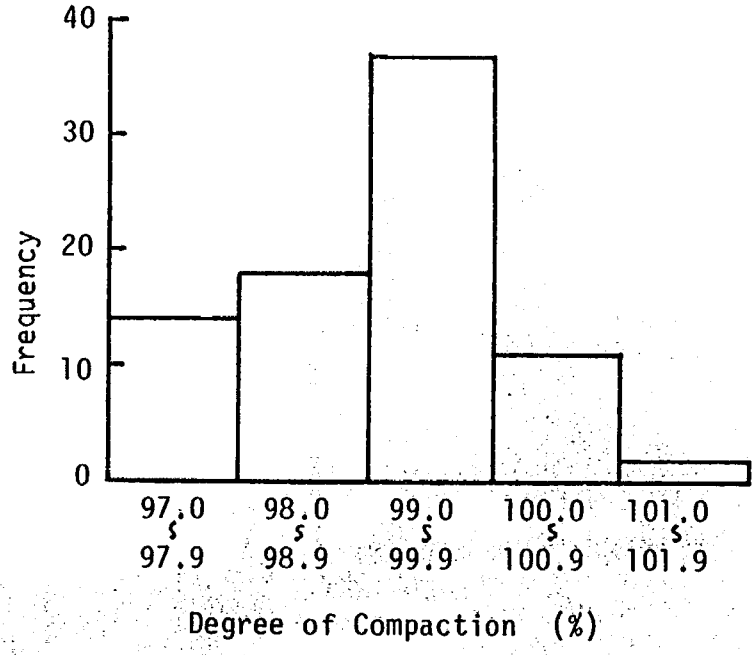


Figure 1/ Degree of Compaction of Specimen

2.2 Test Procedures

The three point flexural test in a span length 320mm was carried out at the constant speed (100 mm/min). The signal of load was stored in a disital storage ocilloscope, because the pen of the recorder did not respond to the signal of load at low temperatures. Then, the curve of load and time was recorded on the paper of Pen-recorder. The diagram of the test is shown in Figure 2 and Figure 3 shows the test apparatus.

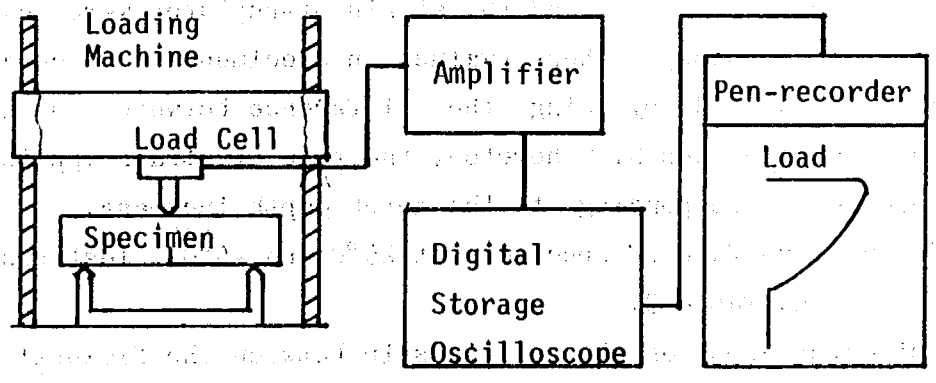


Figure 2/ Diagram of Flexural Test

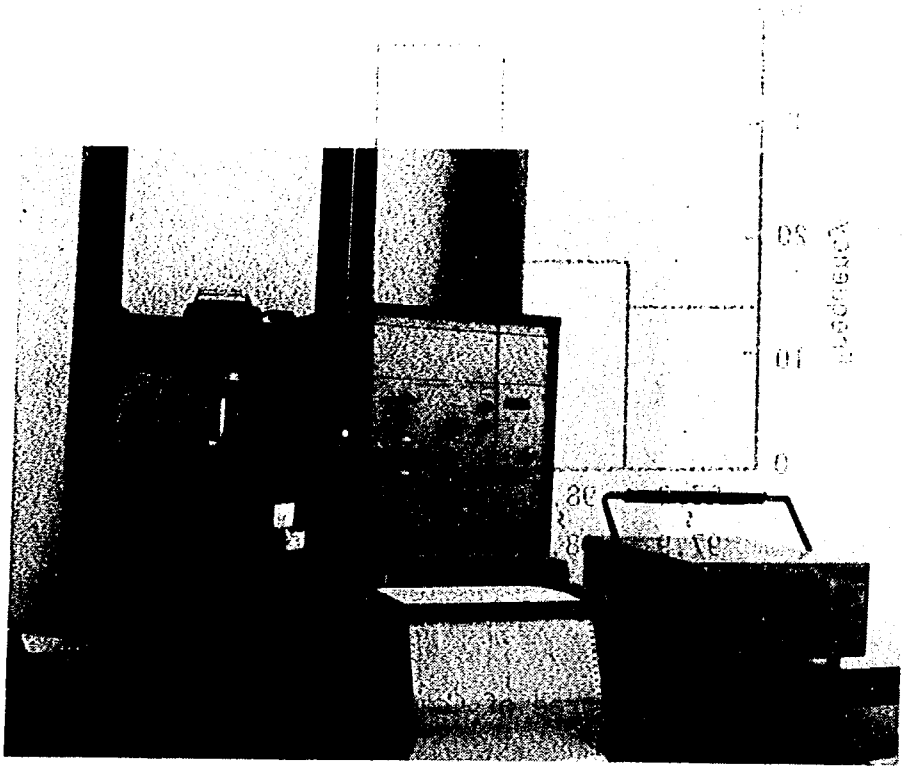
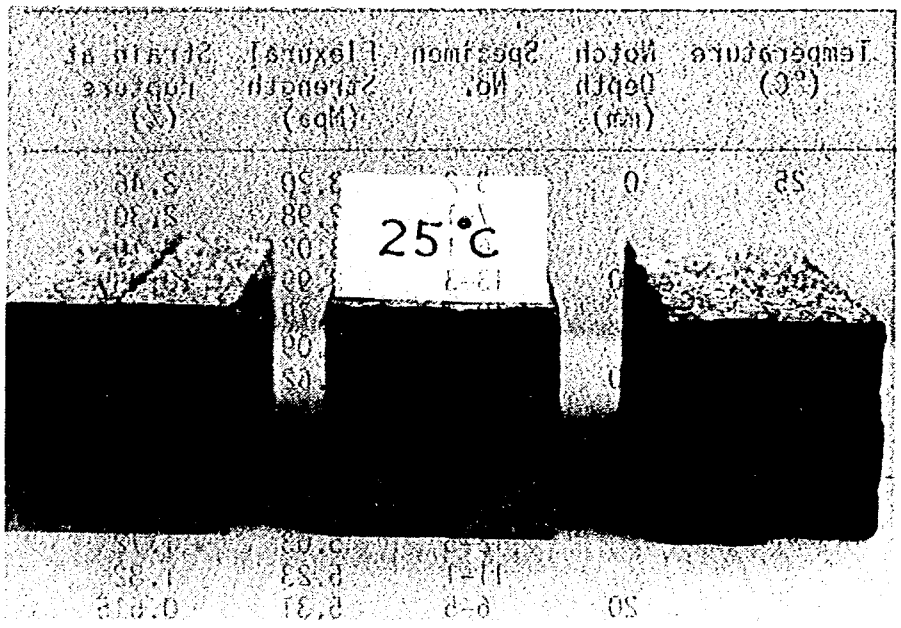


Figure 3/ Test Apparatus

3. Test Results

The peak point on the curve of load and time was defined as a rupture point. The flexural strength and the strain-at-rupture were calculated through the elastic theory. These values on specimens of notched asphalt concrete were calculated by using the difference between the height of specimen and the notch depth. Therefoe, the rate of strain applied to the specimen decreases, coresponding to the notch depth increase. Figure 4 shows the destructive face of specimens at 25°C and -5°C. Test results are shown in Table 2 and Table 3.

The effect of notch depth on the relationship between the flexural strength and the temperature is shown in Figure 5. Figure 6 shows the effect of notch depth on the curve of strain-at-rupture and temperature.



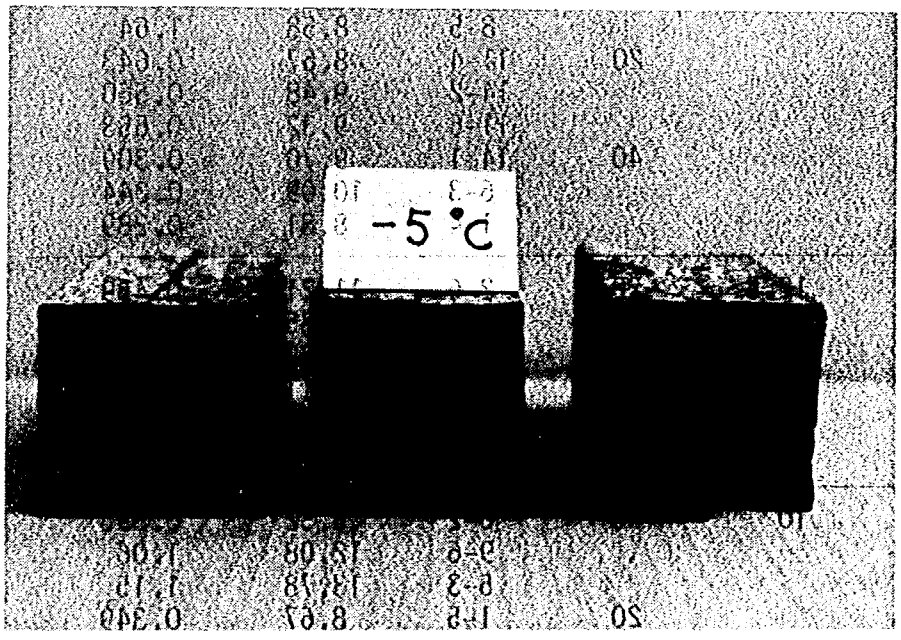


Figure 4/ Destructive Faces of Specimens

Table 2/ Test Results of Flexural Strength and Strain-at-rupture

Temperature (°C)	Notch Depth (mm)	Specimen No.	Flexural Strength (Mpa)	Strain at rupture (%)
25	0	3-2	3.20	2.46
		7-1	2.98	2.30
		6-1	3.02	2.10
	20	13-3	2.99	0.989
		10-4	2.79	0.899
		9-3	3.09	0.956
	40	3-4	3.62	0.477
		2-1	3.14	0.487
		4-6	3.29	0.444
20	0	8-6	4.31	1.68
		12-5	5.03	1.72
		11-1	5.23	1.82
	20	6-5	5.31	0.615
		5-6	5.57	0.680
		11-2	4.91	0.704
	40	5-2	5.91	0.364
		6-6	5.98	0.281
		12-4	5.61	0.321
15	0	13-2	7.80	1.38
		12-2	8.42	1.82
		8-5	8.58	1.64
	20	11-4	8.67	0.643
		14-2	9.48	0.560
		11-6	9.32	0.653
	40	14-1	9.70	0.309
		6-3	10.09	0.244
		1-4	8.81	0.289
12.5	20	2-6	11.21	0.459
		7-3	10.35	0.392
		12-1	9.32	0.355
	40	7-6	10.11	0.207
		13-5	9.58	0.190
		14-3	9.23	0.176
10	0	6-2	13.52	0.893
		9-6	12.08	1.06
		5-3	13.78	1.15
	20	1-5	8.67	0.349
		1-2	7.58	0.278
		4-5	8.46	0.325
	40	5-1	8.70	0.127
		5-5	9.49	0.151
		5-4	10.36	0.155

Table 3/ Test Results of Flexural Strength and Strain-at-ripture

Temperature (°C)	Notch Depth (mm)	Specimen No.	Flexural Strength (Mpa)	Strain at rupture (%)
7.5	20	1-1	8.68	0.312
		7-5	8.81	0.315
		11-3	8.18	0.336
	40	10-5	8.48	0.150
		9-5	8.82	0.119
		2-4	9.29	0.145
5	0	4-3	10.58	0.522
		1-3	10.16	0.515
		14-4	10.77	0.735
	20	9-2	9.24	0.321
		10-6	9.39	0.324
		3-1	10.31	0.308
	40	2-2	11.23	0.164
		4-2	9.91	0.112
		4-4	9.13	0.134
0	0	8-1	9.15	0.427
		2-5	8.00	0.425
		13-1	8.66	0.446
	20	14-6	8.08	0.292
		12-6	9.79	0.290
		10-3	9.37	0.333
	40	6-4	11.47	0.124
		13-4	11.50	0.137
		3-6	10.59	0.114
-5	0	13-6	7.76	0.359
		7-2	8.86	0.420
		9-4	8.38	0.438
	20	12-3	10.06	0.255
		11-5	8.44	0.239
		14-5	8.71	0.280
	40	9-1	8.12	0.091
		3-3	10.33	0.124
		10-1	10.96	0.113

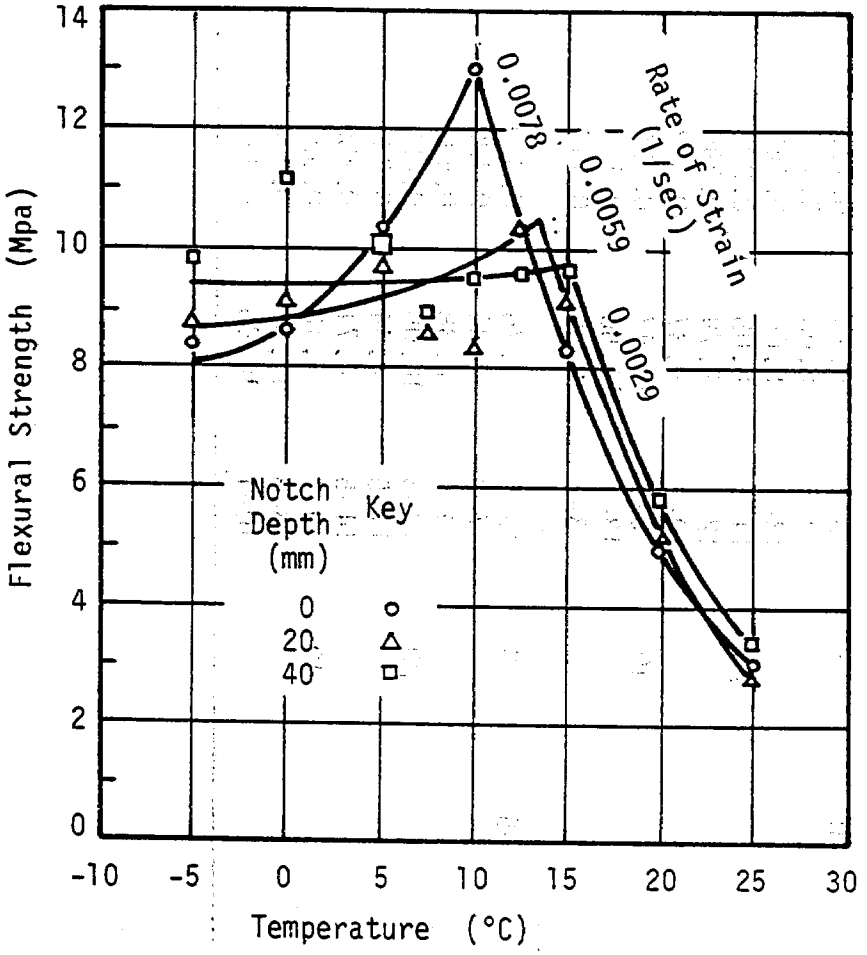


Figure 5/ Effect of Notch Depth on Flexural Strength

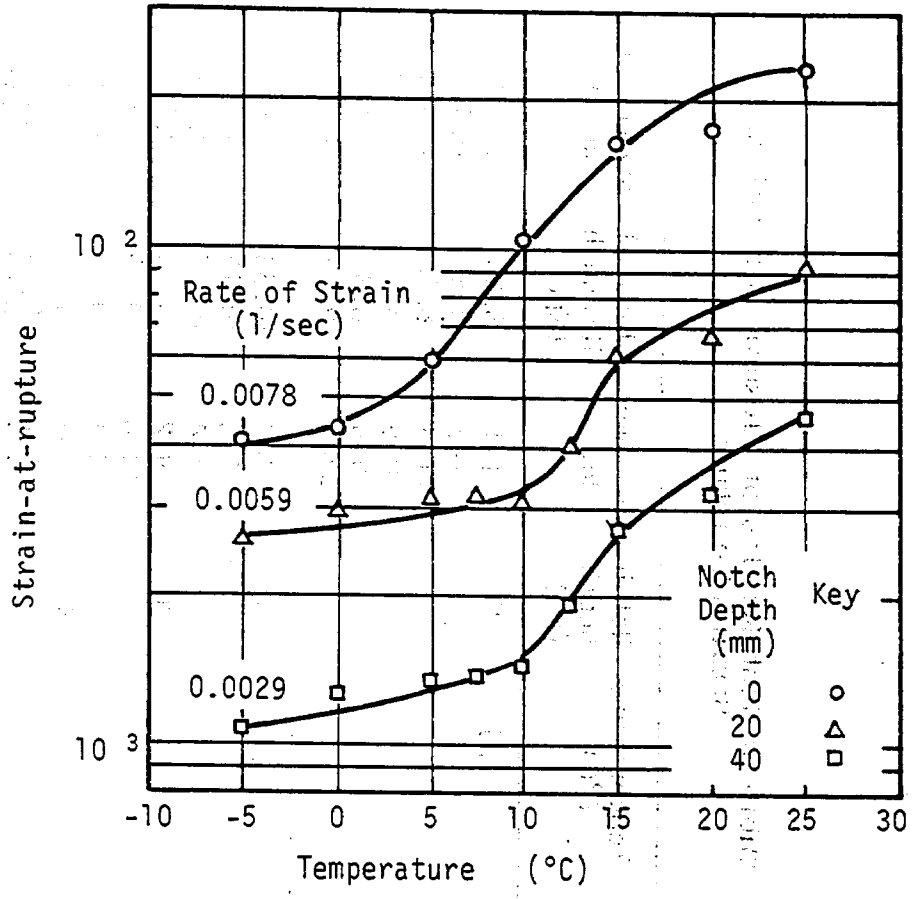


Figure 6/ Effect of Notch Depth on Strain-at-rupture

The relative flexural strength and the relative strain-at-rupture are shown in Table 4.

The flexural strength is independent of the notch depth. the deeper is the notch depth, the smaller is the strain-at-rupture.

The relationship between the notch depth and the temperature of transition point [6] is shown in Table 5. The deeper is the notch depth, the higher is the transition point, however the rate of strain may be low.

Table 4/ Effect of Notch Depth on Flexural Strength and Strain-at-rupture

Temp. (°C)	Flexural Strength (Mpa)			Strain-at-rupture (%)		
	Notch Depth (mm)			Notch Depth (mm)		
	0	20 *	40 *	0	20 **	40 **
25	3.07	2.96 (0.96)	3.35 (1.09)	2.29	0.948 (0.41)	0.469 (0.20)
20	4.86	5.26 (1.09)	5.83 (1.20)	1.74	0.666 (0.38)	0.322 (0.19)
15	8.27	9.16 (1.11)	9.53 (1.15)	1.61	0.619 (0.38)	0.281 (0.17)
10	13.13	8.24 (0.63)	9.52 (0.73)	1.03	0.317 (0.31)	0.144 (0.14)
5	10.50	9.65 (0.92)	10.09 (0.96)	0.59	0.318 (0.54)	0.137 (0.23)
0	8.60	9.08 (1.06)	11.19 (1.30)	0.43	0.305 (0.71)	0.125 (0.29)
-5	8.33	8.70 (1.04)	9.80 (1.18)	0.41	0.258 (0.64)	0.109 (0.27)

*:Ratio when flexural strength is 1 at notch depth 0 mm
**:Ratio when Strain-at-rupture is 1 at notch depth 0 mm

Table 5/ Effect of Notch Depth on Transition Point

Notch Depth (mm)	Transition Point (°C)	Rate of Strain (1/sec)
0	10.0	0.0078
20	13.0	0.0059
40	15.0	0.0029

4. Failure Envelope

The effect of notch depth on the failure envelope of the asphalt concrete is shown in Figure 7. The failure envelope of asphalt concrete has only one curve, regardless of the penetration of asphalt, the temperature and the rate of strain [7]. The failure envelope moves toward the zone of smaller strain-at-rupture, according to the increase of the notch depth.

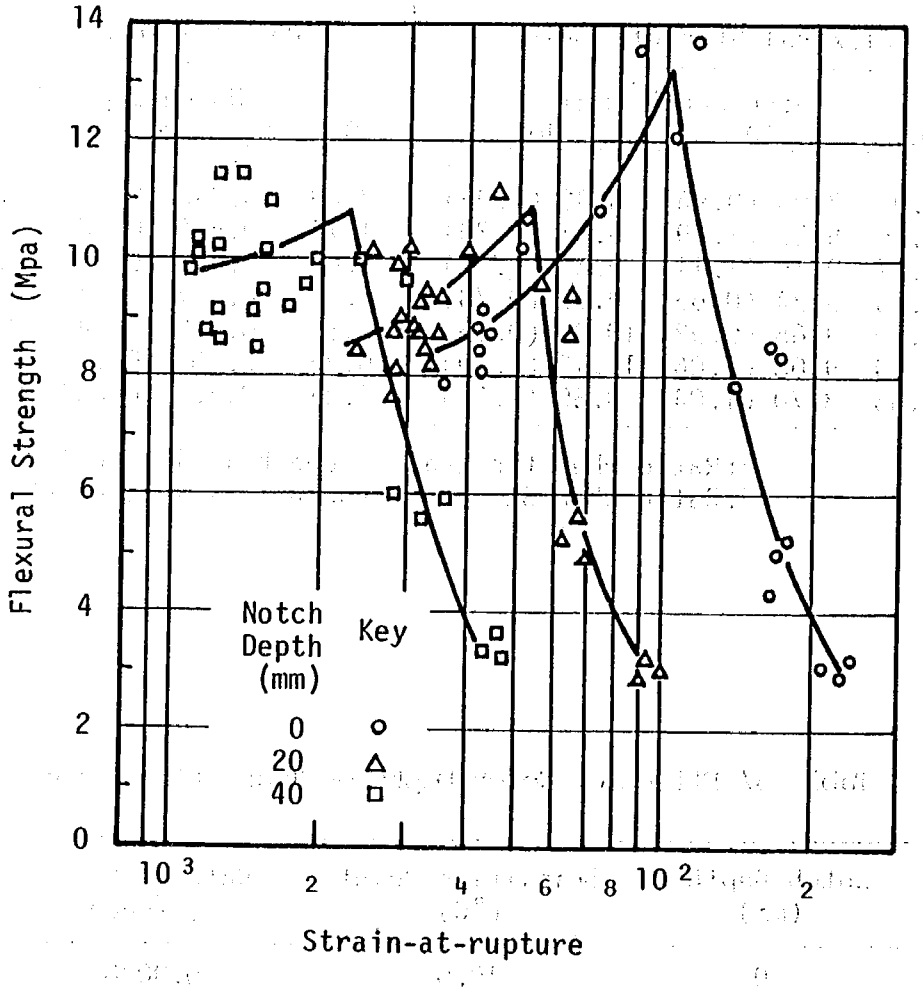


Figure 7/ Effect of Notch Depth on Failure Envelope

5. Conclusions

The major points of this paper may be summarized as follows.

1) Flexural strength of asphalt concrete at earch temperature independ on existence of notch and notch depth.
2) The deeper of notch, the smaller is strain-at-rupture of asphalt concrete at earch temperature.
3) The deeper of notch, the higher is the transition point.
4) Overlay in thick 60mm has two times strain-at-rupture as overlay in thick 40mm. The thicker is overlay, the larger is the allowed value of tensile strain.

Acknowledgement

An acknowledgement is given to Asphalt Research Group with Technical Laboratory of Taishei Road Construction Ltd who provided some brocks of asphalt concrete.

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Study on Reflection Cracks by Photoelastic Experiments

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ABSTRACT

This study, as fundamental research concerning the occurrence mechanism of reflection cracks, was carried out with the purpose of examining the stress conditions of asphalt mixture layers around joints in concrete slabs. In effect, using models of two-dimensional, two-layered structures in which the upper layers were made to correspond to asphalt mixture layers and the bottom layers concrete slabs, the states of distribution of shear stresses in these upper layers were determined by photoelastic experiments.

Normal temperature-setting epoxy resin board was used as the photoelastic material in the upper layers of these experiments, with phenolic resin board used for the lower layers. The ratio of the elastic moduli of the bottom and top layers was made 1:0.15, to model a composite pavement in cold weather.

The effects of factors such as thickness of overlay, joint width, and loading width on the stress conditions around joints were clarified by this study. Further, the effects in case bearing condition was not uniform were also examined.

STUDY ON REFLECTION CRACKS BY PHOTOELASTIC EXPERIMENTS

by

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1. Introduction

When pavements have been repaired with overlays of asphalt mixtures, there are cases in which so-called reflection cracks are produced in the overlays due to local displacement behaviors of the underlying pavements. These reflection cracks often occur at relatively early times after repairing to greatly affect the service lives of the pavements.

In the study reported here, reflection cracks produced in the overlay of an asphalt mixture on a concrete pavement were made the objects of consideration. That is, with a composite pavement such as this, because of vertical or horizontal displacements at joints of the concrete slabs, excessively large stresses act locally on the upper-layer of asphalt mixture, and reflection cracks are produced. The vertical displacement at the joint of a concrete slab is produced by traffic load, while horizontal displacement is produced by temperature and moisture content variations, and reflection cracks are considered to be due to the composite actions of these factors.

According to investigations and studies concerning reflection cracks, the frequency of reflection crack occurrence is generally increased with increased traffic volume, while it is decreased as the thickness of the overlay is increased.[1] From this, it may be surmised that the vertical displacement behavior of the pavement due to traffic load has a greater influence on occurrence of reflection cracks than horizontal displacement behavior due to temperature.

This study, in view of the above, comprises an examination by photoelastic experiments on the stress conditions inside the asphalt mixture layers around joints of concrete slabs in composite pavements subjected to loading. Since stress analyses by photoelastic experiments consider two-dimensional states, the results differ from the structures and loading

conditions of actual pavements in three-dimensional states. However, by imparting the structural characteristics of composite pavements to experimental models, even though qualitatively, the stress distribution states inside the layered structures, particularly, shear stresses which are related to cracking failure can be studied in detail by photoelastic experiments.

2. Experimentation Method

Each experimentation model, in consideration of the elastic moduli of the concrete and asphalt mixture which are actual pavement materials, was made to have a two-layered construction using epoxy resin for the upper layer and phenolic resin for the lower layer. Especially, with the epoxy resin used for the upper layer, it is possible for the elastic modulus to be altered by changing the mix proportions. In the experiments reported here the ratio of the elastic moduli of the lower and upper layer was arranged to be 1:0.15. This was a case of the elastic modulus of the asphalt mixture layer being 50,000 kg/cm^2 when the elastic modulus of the concrete slab was 300,000 kg/cm^2, and corresponds to a condition at low temperature when reflection cracks are liable to occur in actual pavements. The elastic moduli and photoelastic stress sensitivities of the materials used are as shown in Table 1.

Table 1 Elastic Moduli and Photoelastic Sensitivities of Materials Used for Experimentation Model

Material	Elastic Modulus (kg/mm^2)	Photoelastic Sensitivity (mm/kg)
Epoxy Resin	200	1.05
Phenolic Resin	1,300	-

The construction and dimensions of the experimentation model are shown in Fig. 1. The epoxy resin boards of the upper layer were in the three heights of 4 cm, 6 cm, 8 cm, while the lower layer was phenolic resin board 20 cm in height having a slit 1 cm or 2 cm in width corresponding to a joint in the concrete slab. The interface between the upper and lower layers was bonded with the epoxy resin used for the upper layer. Applied load of 96 kg and loading width of 4 cm were taken as standards,

with the width made 8 cm in part of the experiments. Loading was done in three ways as shown in Fig. 2: with the center line of the loading width directly above the slit (Loading Mode 1), with the center line 2 cm off center (Loading Mode 2), and with the center line 4 cm off center (Loading Mode 3).

The isochromatic fringe order obtained by photoelastic experiments is proportional to the absolute value of the difference between principal stresses, and is given by the following equation:(2)

$$N = \alpha d |\sigma_1 - \sigma_2| \quad \cdots\cdots \quad (1)$$

where

N: isochromatic fringe order

α: photoelastic sensitivity (mm/kg)

d: thickness of specimen (mm)

σ_1, σ_2: principal stresses (kg/mm^2)

As can be seen from Eq. (1), the fringe pattern of the isochromatic lines expresses the distribution condition of shear stresses.

3. Results of Experiments

(1) Influence of Thickness of Upper Layer

Photos. 1 to 3 comprise a series of photoelasticity photographs of the upper layer when loading experiments were conducted changing the thickness of the upper layer. Figs. 3 to 5 show the shear stresses obtained by Eq. (1) expressed as shear stress distributions at the X-X' plane along the bottom surface of the upper layer and the Y-Y' plane directly above the slit. According to the photoelasticity photographs, isochromatic lines of high fringe order are produced at the vicinity of the contact points of the corners of the slit in the lower layer and the bottom surface of the upper layer, and it is indicated that these are the points of concentration of shear stresses which are the causes of reflection cracks.

The disturbance due to existence of the slit in the isochromatic fringe pattern inside the upper layer accompanying shifting of the applied load is greater the smaller the thickness of the upper layer. However, there are regularities in the distribution waveforms of shear stresses at the Y-Y' and X-X' planes in the cases of the thicknesses, while the peak values are decreased with shifting of the applied loads. The shear stress distribution at the Y-Y' plane of the upper layer hardly changes even if

layer thickness is increased and the peak value of the shear stresses around the slit decreases in inverse proportion to layer thickness(Fig.6).

(2) Influence of Slit Width

Photoelastic experiments were carried out in case the width of the slit in the experimental model was made 2 cm in order to study the relation between joint width and reflection cracks in the concrete slab (Photo. 4). Compared with the experimental results (Photo. 1(a)) in case of slit width of 1 cm, it may be seen that the isochromatic fringe order around the slit increases as the width of the slit becomes greater.

(3) Influence of Loading Width

In the experiments up to this point the loading width was made 4 cm, equal to the thickness of the upper layer. This is a relatively small value when the actual pavement and the contact width of vehicle wheels are compared, but is a value adopted as an expedience in carrying out the experiments. Therefore, experiments were conducted with the loading width 8 cm, double the upper-layer thickness to correspond to actual cases. The applied load was 96 kg, the same as before.

The isochromatic lines obtained for the cases of slit widths 1 cm and 2 cm with the thickness of the upper layer of the expermental model 4 cm are shown in Poto. 5. The distributions of shear stresses obtained from these are shown in Fig. 8. On comparing this with the case of loading width of 4 cm (Fig. 7), it can be seen that the distribution of shear stresses hardly changes in case of slit width of 1 cm whereas in case of slit width of 2 cm, the peak value is greatly decreased. In effect, the influence of the size of slit width is alleviated by the increase in loading width, and the peak value of shear stresses around the slit is decreased.

(4) Influence of Non-uniform Bearing Condition

With an actual pavement, the difference in the bearing conditions of the concrete slabs sandwiching the joint from both sides may be considered to have a substantial effect on the stress conditions of the overlay. Therefore, a rubber sheet was laid on the phenolic resin at one side of the slit as shown in Fig. 9, and loading experiments were conducted changing the bearing conditions.

The load applied in this experiment was 60 kg, 62.5% of the cases in the experiments up to this time, and as shown in Photo. 6, a high-order fringe pattern has been produced in the isochromatic lines. The distribution of shear stresses obtained from this is shown in Fig. 10. According to the

above, the peak value of shear stress has occurred at the side of high bearing reaction. In case the bearing condition around the joint is not uniform in this way, attention should be paid to the fact that an excessively large stress acts on the overlay.

4. Conclusion

The states of distribution of shear stresses produced in the upper layers of two-layered models with slits provided in the lower layers were examined by photoelastic experiments to carry out basic examinations of the occurrence mechanism of reflection cracks. The conclusions obtained in these experiments are as follows:

a) When a loading test is performed, the peak value of shear stresses thought to be the direct cause of reflection cracks is produced in the vicinity of the slit corresponding to the joint of an actual pavement.

b) These shear stresses are decreased with increase in thickness of the upper layer. In essence, this indicates that increase in thickness of the overlay layer in an actual pavement is effective in alleviating reflection cracks.

c) In case the ratio between loading width of the applied load and the thickness of the upper layer is small, the peak value of shear stresses becomes small if the slit width is narrow. However, the influence is small when the ratio is large.

d) In case the bearing condition is not uniform, excessively large shear stresses are produced around the slit. This indicates that undersealing of the concrete slab prior to repair by overlay will be effective.

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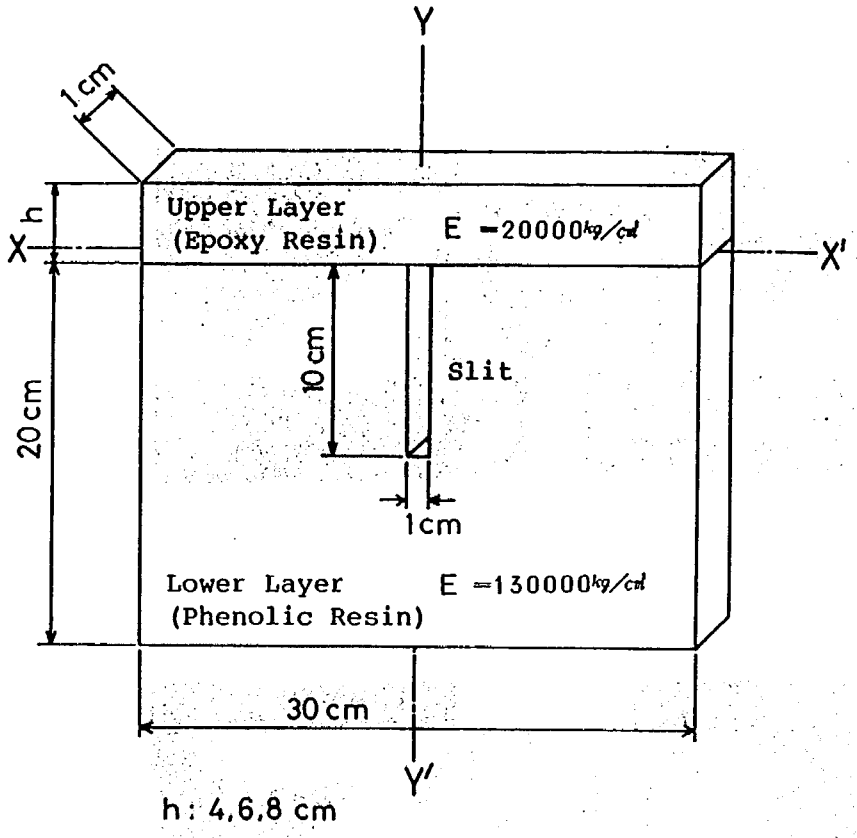

Fig. 1 Dimensions of Experimental Model

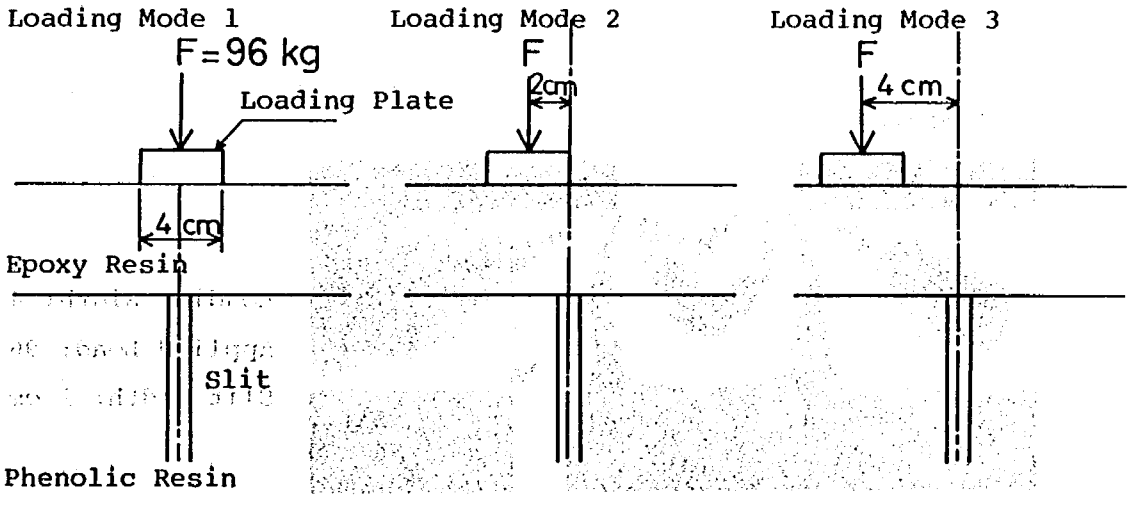

Fig. 2 Loading Locations

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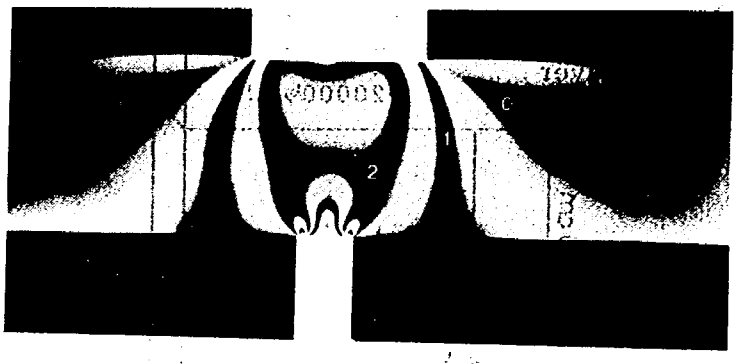

Loading Mode 2

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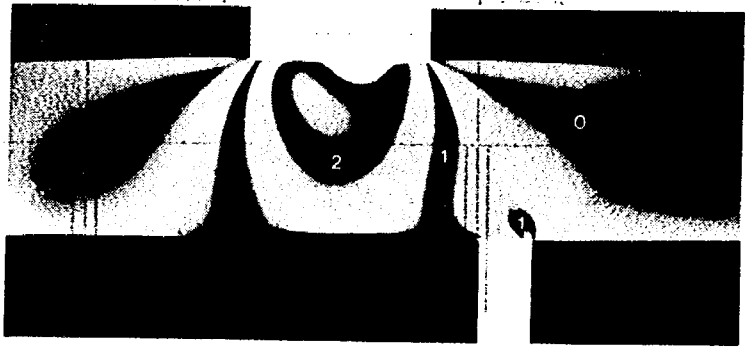
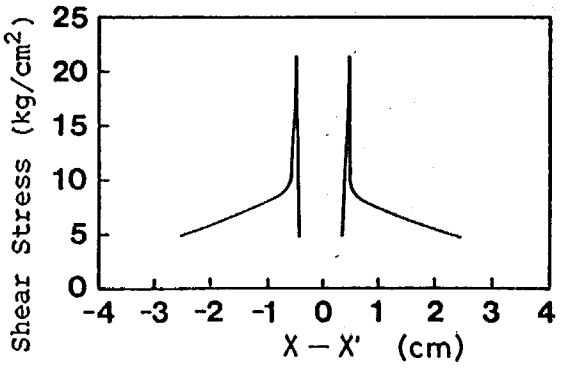
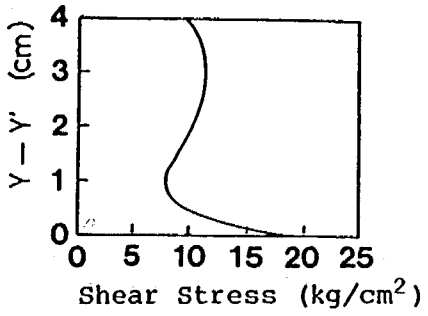


Loading Width: 4 cm
Applied Load: 96 kg
Slit Width: 1 cm

Photo 1 Isochromatic Lines in Case of Upper Layer Thickness 4 cm

Loading Mode 1





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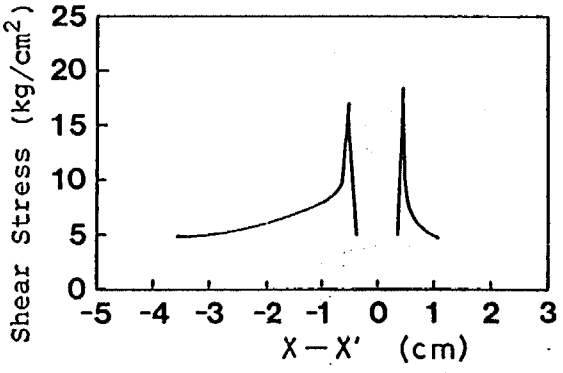

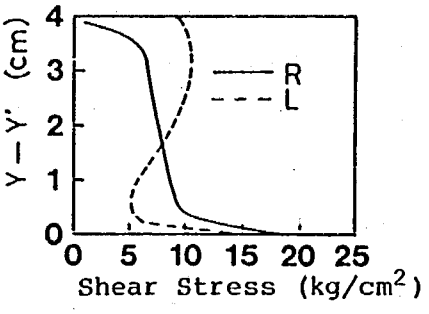

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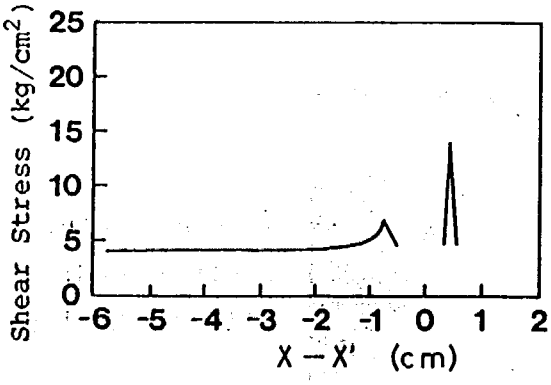

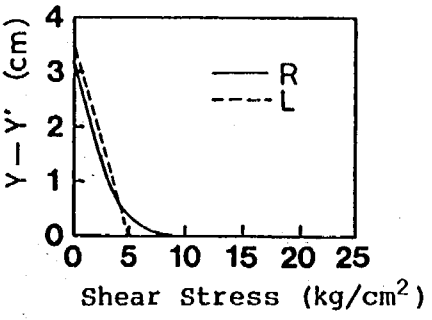

Fig. 3 Distributions of Shear Stresses in Case of Layer Thickness 4 cm

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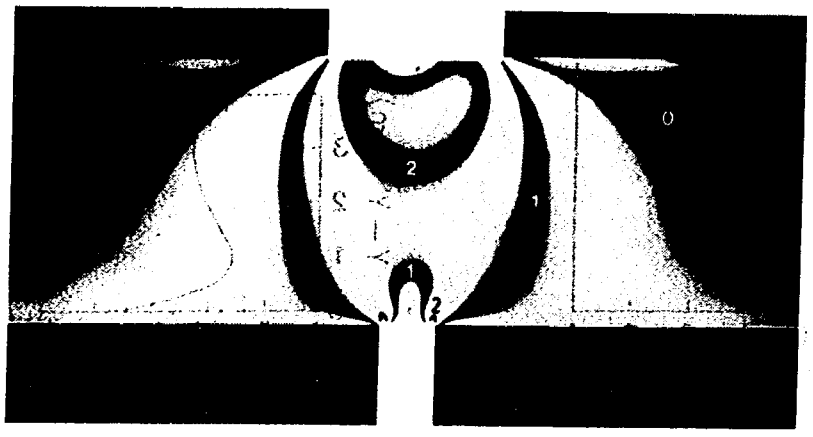

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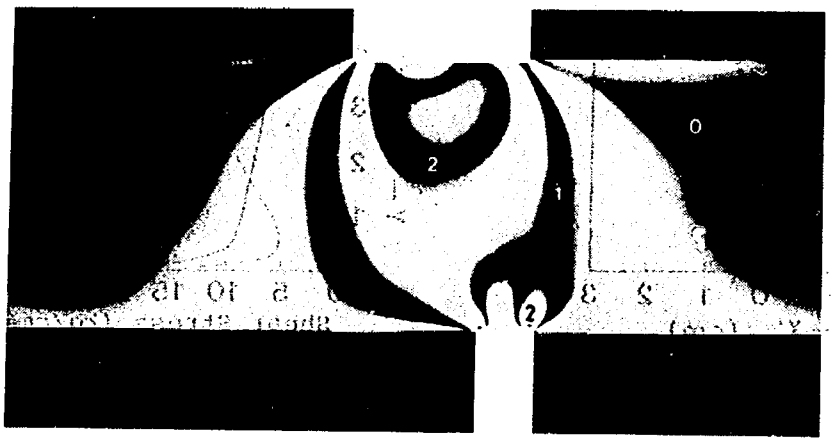

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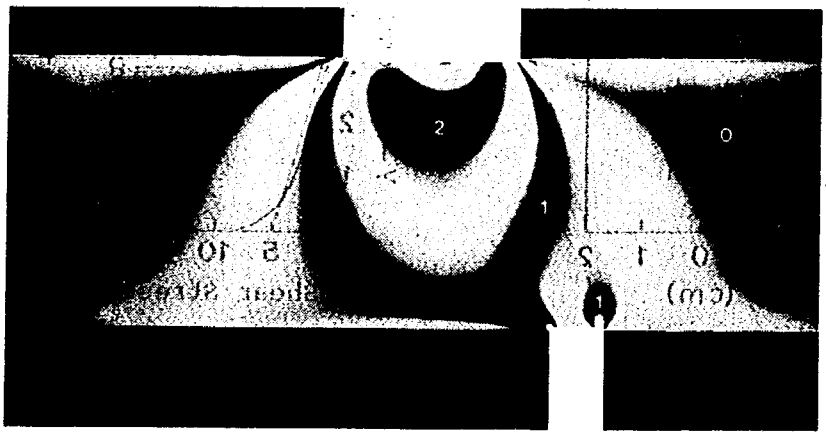

Loading Width: 4 cm
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Slit Width: 1 cm

Photo 2 Isochromatic Lines in Case of Upper Layer Thickness 6 cm

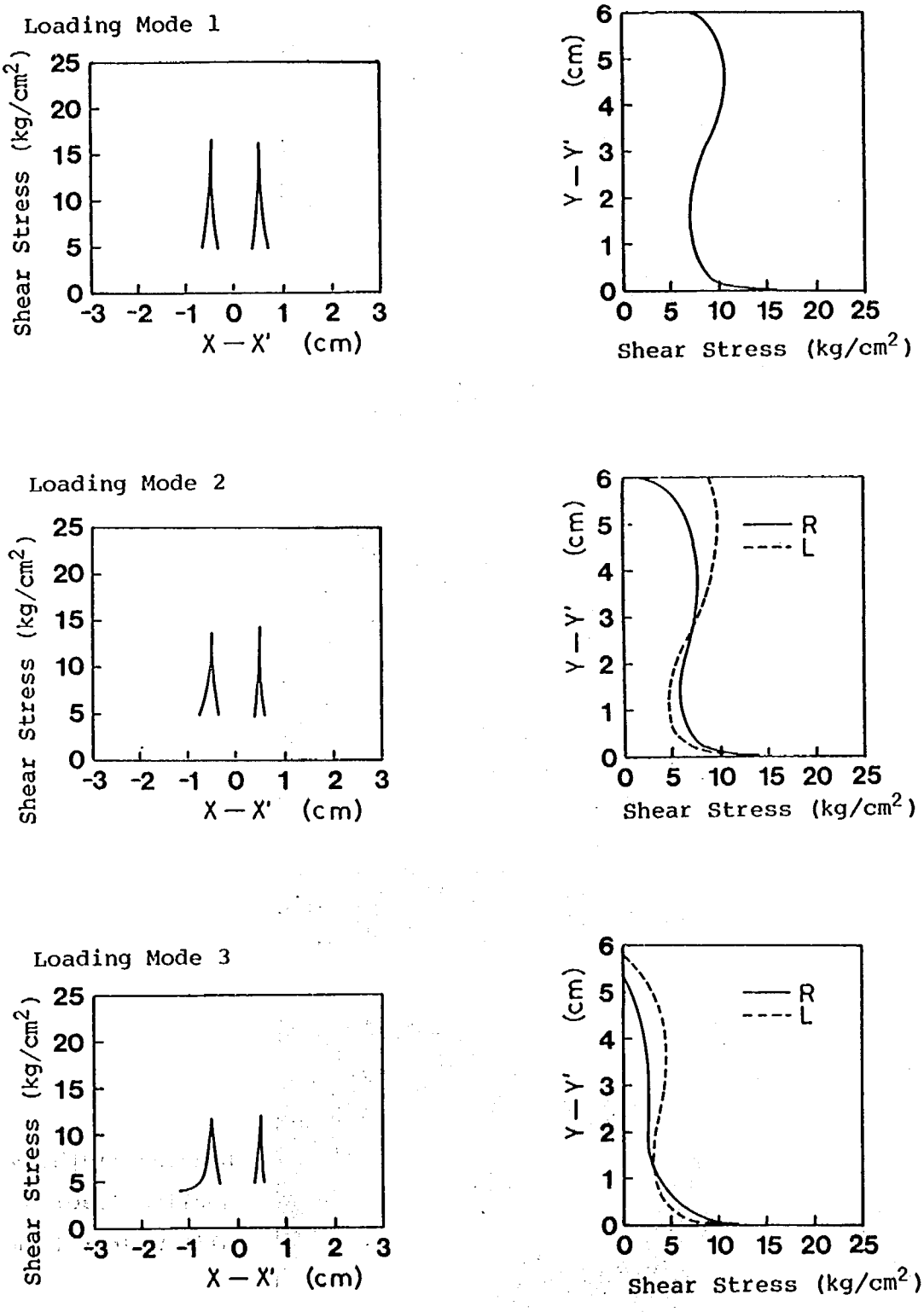

Fig. 4 Distributions of Shear Stresses in Case of Layer Thickness 6 cm

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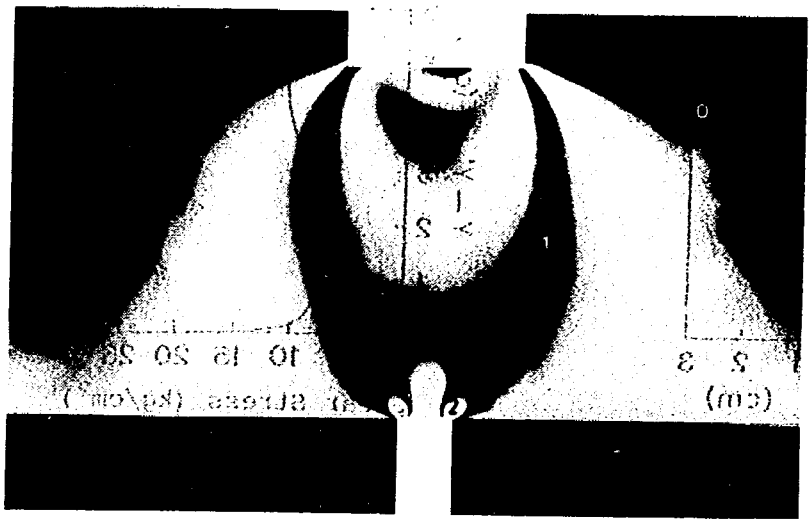

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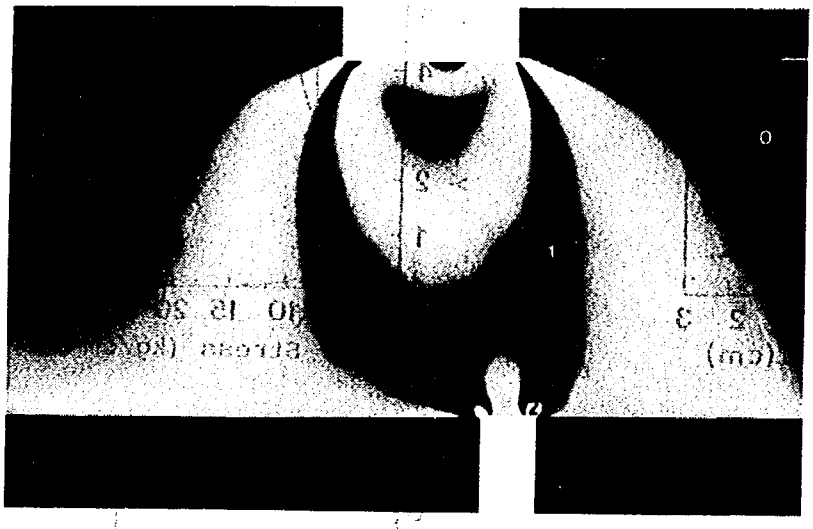

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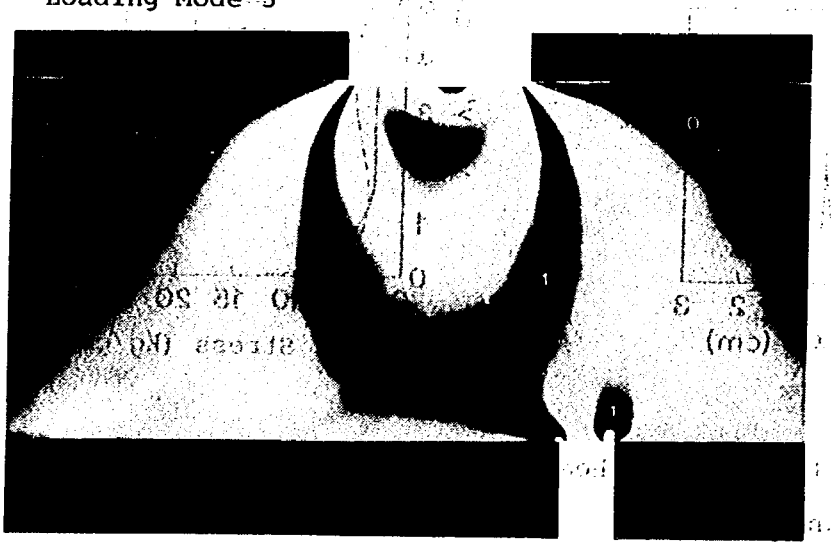


Loading Width: 4 cm
Applied Load: 96 kg
Slit Width: 1 cm

Photo 3 Isochromatic Lines in Case of Upper Layer Thickness 8 cm

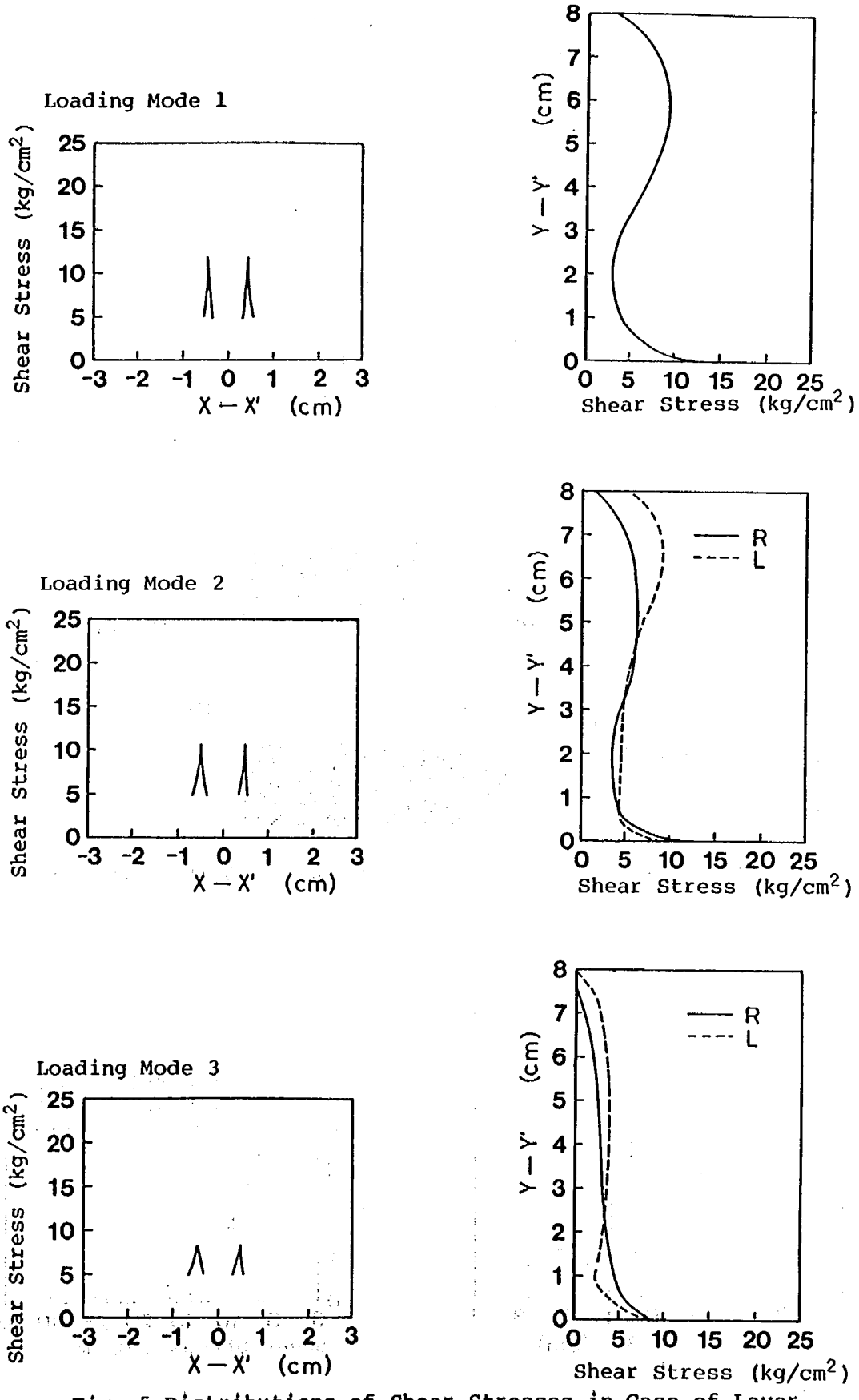


Fig. 5 Distributions of Shear Stresses in Case of Layer Thickness 8 cm

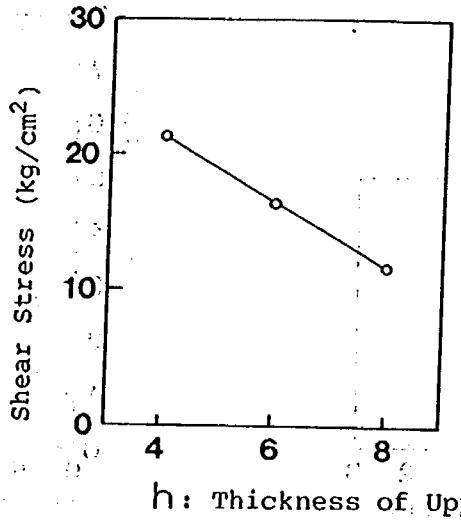
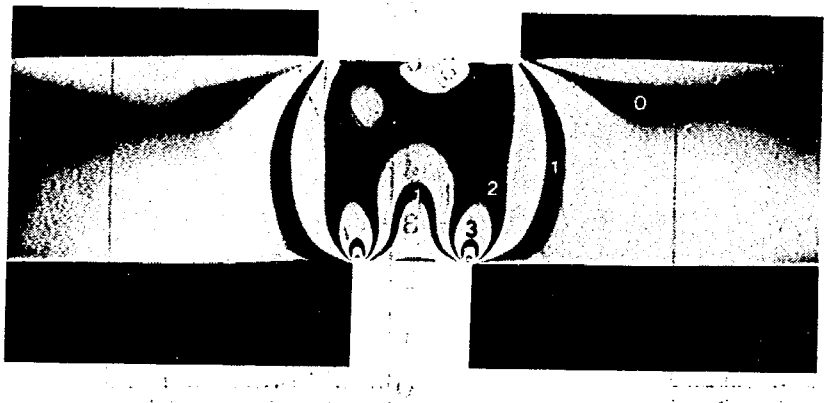


Fig. 6 Relation between Layer Thickness and Shear Stress



Loading Width: 4 cm
Applied Load: 96 kg
Upper Layer Thickness: 4 cm

Photo 4 Isochromatic Lines in Case of Slit Width 2 cm

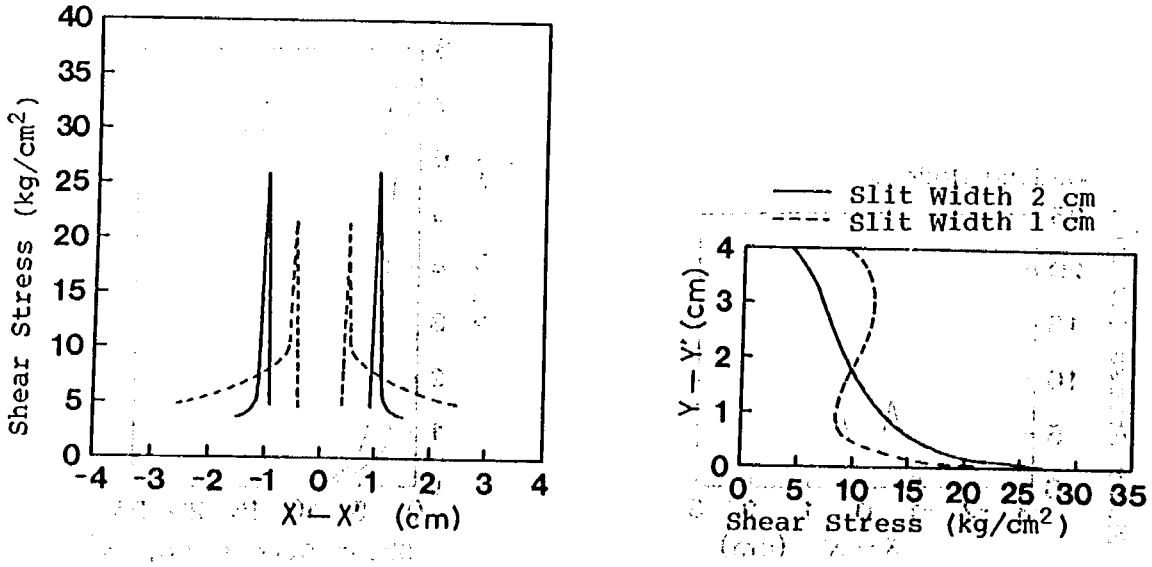
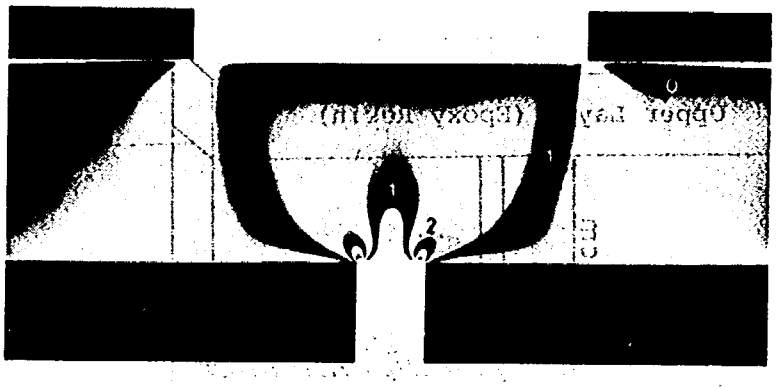
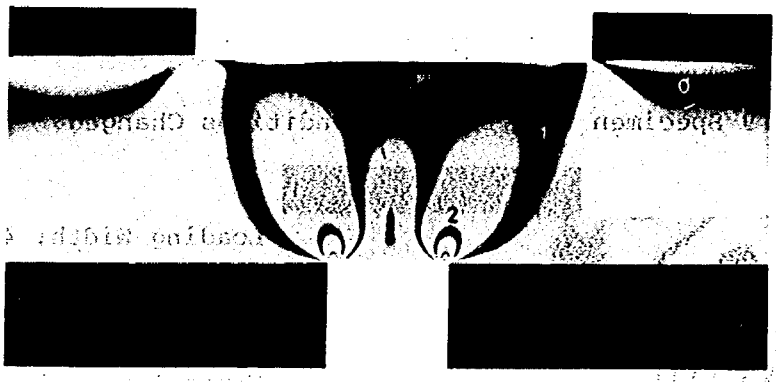


Fig. 7 Shear Stress Distribution According to Slit Width

Slit Width 1 cm



Slit Width 2 cm



Loading Width: 8 cm

Applied Load: 96 kg

Upper Layer Thickness: 4 cm

Photo 5 Isochromatic Lines in Case of Loading Width Double Upper Layer Thickness

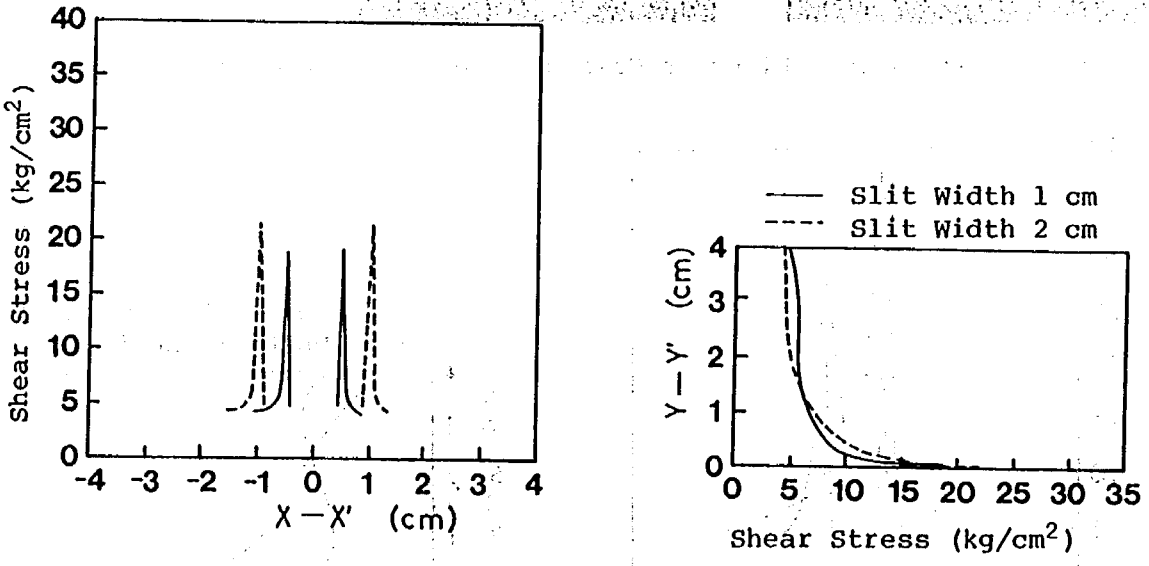


Fig. 8 Shear Stresses Due to Changes in Loading Width

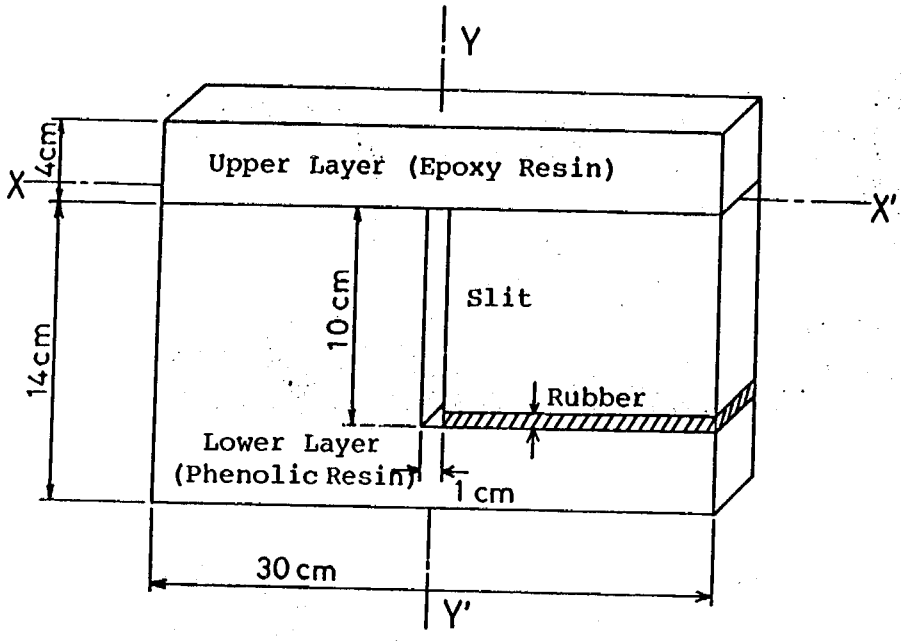
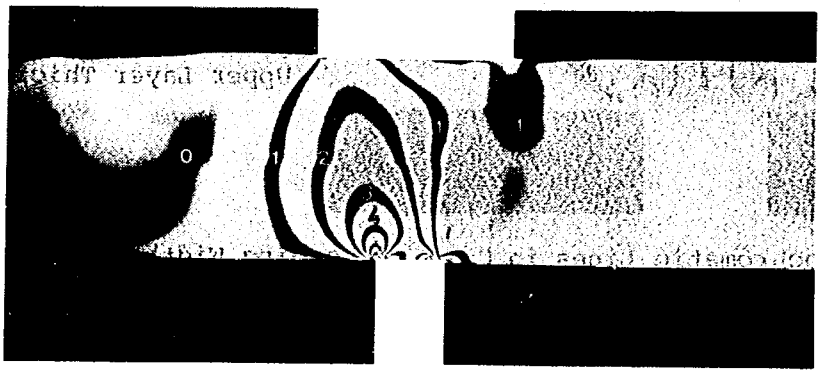


Fig. 9 Specimen with Bearing Conditions Changed



Loading Width: 4 cm
Applied Load: 60 kg
Slit width: 1 cm
Upper Layer Thickness: 4 cm

Photo 6 Isochromatic Lines in Case of Non-uniform Bearing Condition

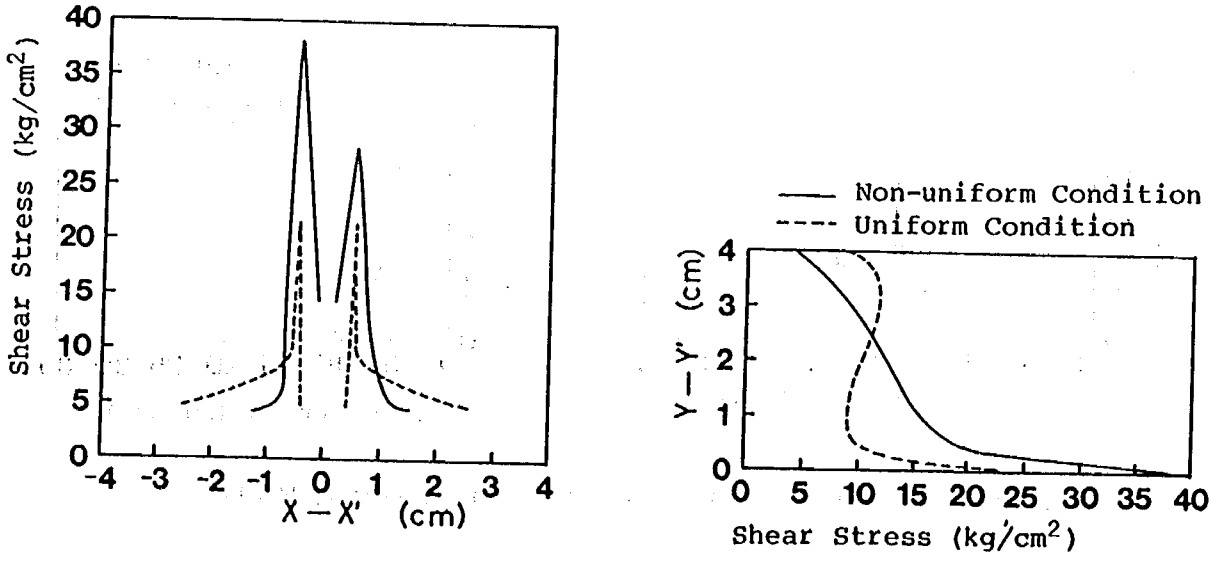


Fig. 10 Distributions of Shear Stresses in Case of Non-uniform Bearing Conditions

Distress of Asphalt Pavement of Roadways in Tohoku District and Repair and Maintenance

Mr. T. Sasaki, Mr. K. Isobe, Mr. C. Kamiya, Mr. S. Kashiwabara
Tohoku Regional Construction Bureau, Ministry of Construction

ABSTRACT

This paper presents a study with respect to the characteristics of distress of roadway asphalt pavements in the Tohoku district. First, it deals with the prevention methods of the defacement of the roadway surface that are attributed to the specific traffics and climatic conditions in the district. Second, it discusses the standardization of the repair and maintenance operations of the asphalt pavement.

The Tohoku district is located at the northernmost part of the Honshu main island of Japan, where it is very cold and snowy in the winter season, while the temperature in summer is considerably high. In winter, almost all vehicles are equipped with the studded tires which directly causes the rutting on the asphalt pavement, on the other hand, in summer the hot temperature dissolves the asphaltic materials and originates the corrugations or waves by the traffics on the pavement. Thus, it has been one of our most significant concern to clarify the characteristics and structure of the distress, thereupon, to establish the proper methods so as to maintain the roadways in the good conditions.

We have been conducting a series of roadway surface surveys by means of photography and evaluating the data thereof on the bases of the Maintenance Control Index (M.C.I.). Consequently, we have designated the roadway sections which need the maintenance operation of the pavement in the Tohoku district. We employ the adequate maintenance method for each section in accordance with the M.C.I.

As for the defacement of the asphalt pavements caused by the studded tires we have been employing experimentally following three methods since 1981:

(1) Use of the aggregate with non-continuous gradation
(2) Use of the aggregate with maximum size 20mm
(3) Application of short interval maintenance by the thin overlays

These have been recognized as effective countermeasures for the surface failure.

This report deals with above mentioned subjects, reduction of damage caused by studded tires and standardization of repair method in the future.

DISTRESS OF ASPHALT PAVEMENT OF ROADWAYS IN TOHOKU DISTRICT AND REPAIR AND MAINTENANCE OPERATIONS

by

Takashi Sasaki

Kinji Isobe

Chikahiro Kamiya

Sosuke Kashiwabara

Tohoku Regional Construction Bureau

1. The Outline of the Tohoku District and the Road Pavement

1-1. The Outline of the Tohoku District

The Tohoku District is located in the northern extremity of the main island in Japan (Fig. 1-1), and consists of six prefectures: Aomori, Iwate, Miyagi, Akita, Yamagata, Fukushima. This district has a population of about 9,570,000 and an area of about 67,000 square kilometers, and account for 8.2%, 17.7% of all Japan respectively. It is situated between lat. 37° N and lat. 42°N as shown in Fig. 1-2: The distance in the north-south direction is 527 Km, and that in the east-west direction is 218 Km. Also it is surrounded by seas on three sides. In the central part of the province, the Ou mountains range lies from north to south, and there are many other mountains. This district is mostly located in the snowy and cold area. In the winter the climatic condition are very severe: much snow and low temperature as shown in Table 1-1, Fig. 1-3. The Japan Sea side has a huge snowfall because of the wet seasonal wind blowing from the north-west, but the Pacific Ocean side has clear and dry days continually. On the contrary, in the summer the temperature fairly rises as shown in Table 1-1. Especially inland areas have high temperature day continually. The monthly precipitation varies with the season and location, but the annual mean precipitation is about 1,300 mm.

1-2. The Character of Traffic in the Tohoku District

The Tohoku Regional Construction Bureau administers major 13 routes, about 2,500 Km of all the National Highways in Tohoku District. Fig. 1-4 shows major road network in the Tohoku District. In this district, the main cities of region are scattered in a large space. Principal roads have been developed so as to connect these cities. Table 1-3 shows the breakdown of public roadways in the Tohoku District. Total road length in this district is about 140,000 Km, and the principal road such as the national highways

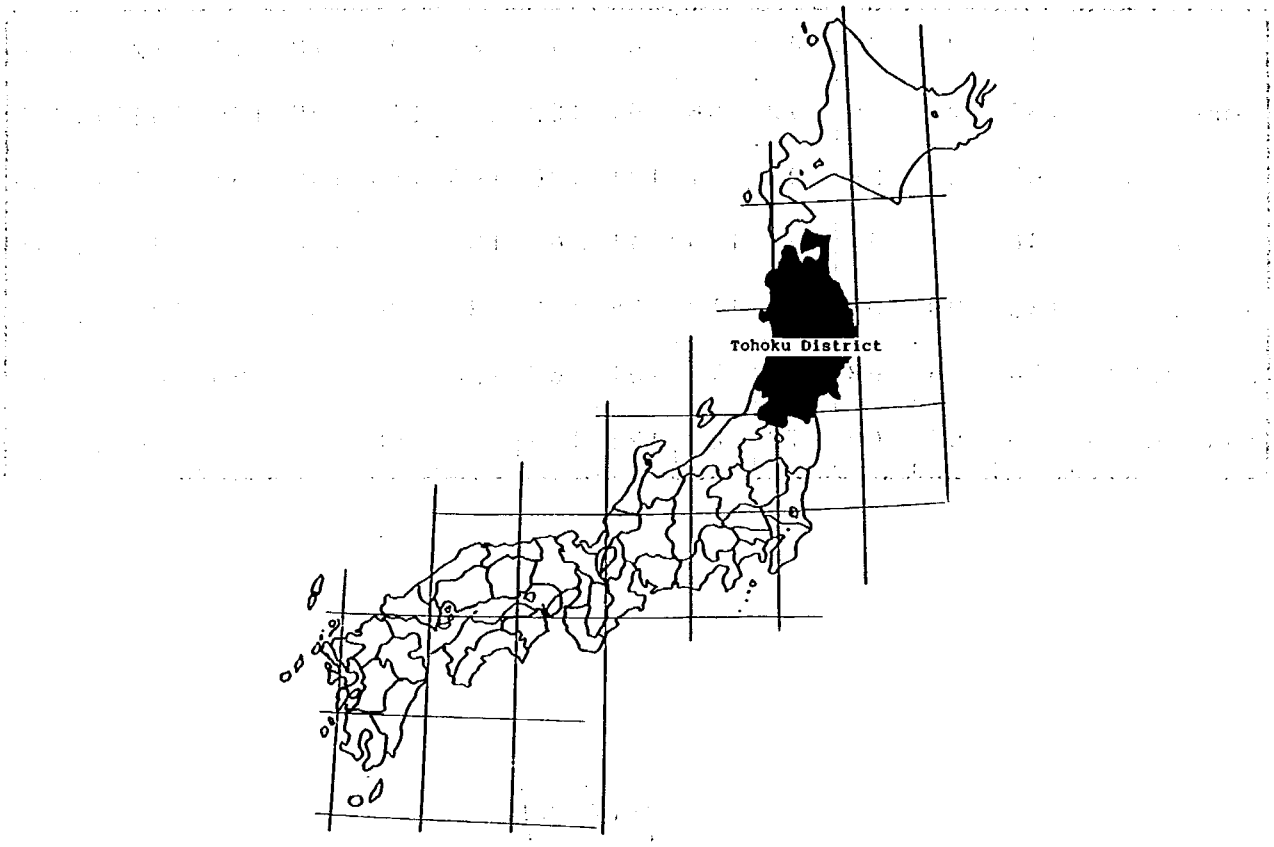


Fig. 1-1 Location of the Tohoku District

Table 1-1 The Mean Monthly Minimum and Maximum Air Temperature in the Tohoku District (°C)

	Aomori	Morioka	Sendai	Akita	Yamagata	Fukushima
The mean monthly minimum daily air temperature in Jan.	-5.1	-6.5	-2.7	-3.4	-4.7	-2.7
The mean monthly maximum daily air temperature in Aug.	27.2	27.8	28.2	28.4	29.8	30.2

Table 1-2 The Mean Monthly Precipitation (mm)

Average during 1951 -1980

	1	2	3	4	5	6	7	8	9	10	11	12	Total
Aomori	181	122	89	72	66	91	102	147	145	98	130	164	1,407
Morioka	65	57	88	97	91	121	170	165	155	108	91	75	1,282
Sendai	46	48	72	82	109	141	160	153	175	116	69	49	1,219
Akita	135	99	107	139	117	128	188	186	180	161	176	171	1,789
Yamagata	94	76	68	62	71	101	167	138	119	85	80	102	1,163
Fukushima	55	50	69	72	82	123	153	125	158	105	61	56	1,108

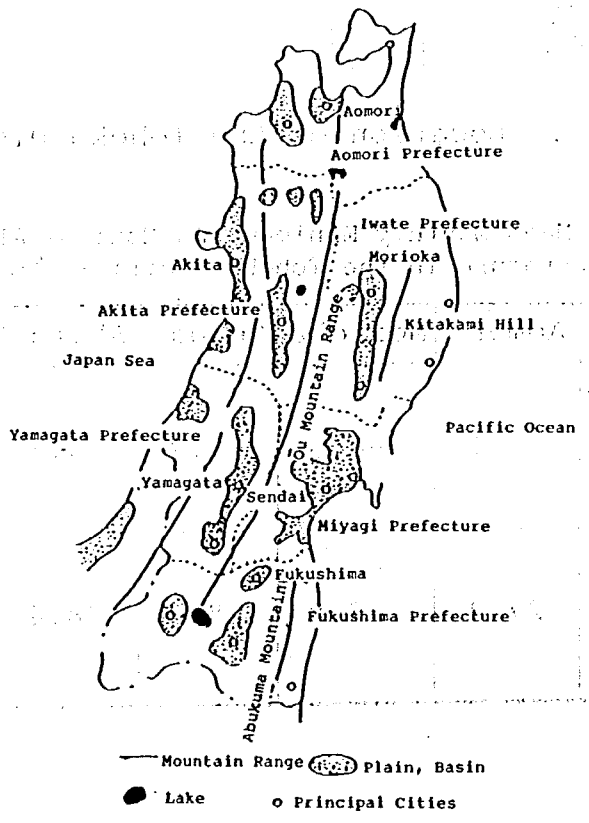


Fig. 1-2 Topography of the Tohoku District

Table 1-3 Breakdown of Public Roadways in the Tohoku District

Classify	Net Length	Length of the pavement roadway (Separatal by line no.)				Length of the non-pavement
		6	4	2	1	
Expressway	446	-	446	-	-	-
National Highway	7,369	23	243	5,924	347	832
Prefecture Road	16,288	13	67	8,964	2,045	5,199
Municipal Road	116,831	49	250	11,364	25,464	79,704
Total	140,934	85	1,006	26,252	27,856	85,736

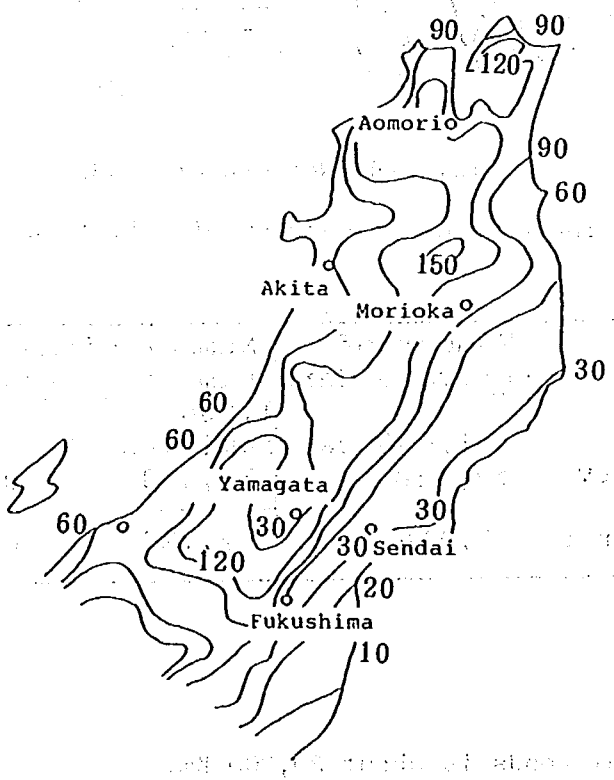


Fig. 1-3 Annual Snow Falling Days

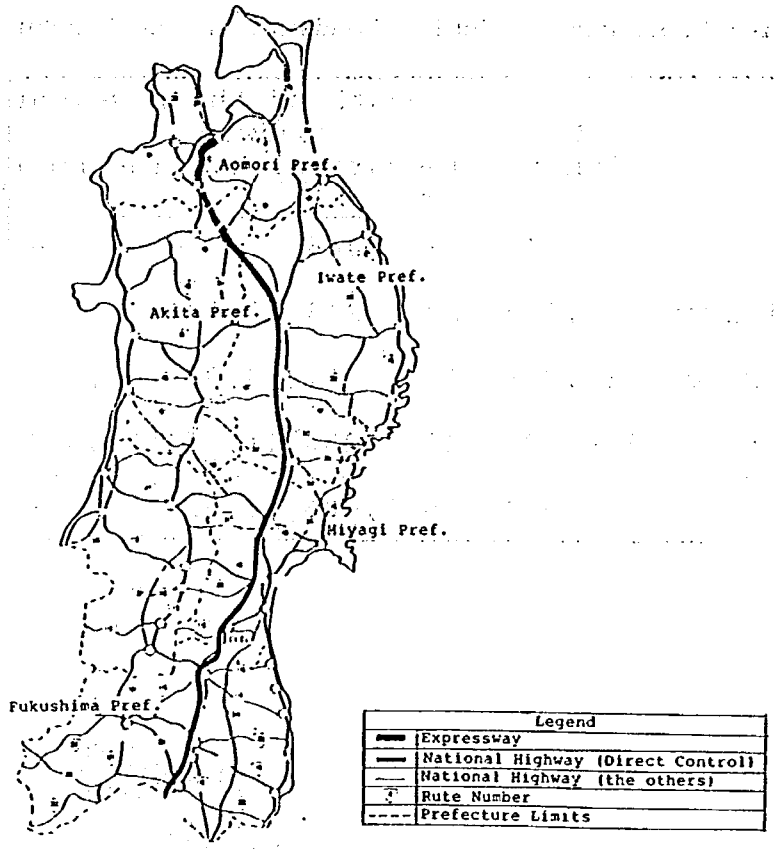


Fig. 1-4 Network of Roadway

Table 1-4 The Rate of Possession of Private Cars in Each Area

Area	Number of Private Cars	Number of Families	Rate of Posession Remark
	1,000 cars	1,000 families	car/family
Whole Country	23,138	37,453	0.618
Tohoku District	1,916	2,722	0.704

and the prefectural roads is about 20,000 Km.

In this district, the network of railroads is relatively sparce. The distance between stations are long and its service are not frequent enough. As a result, the lives of people in rural areas are very dependent on road traffic. The level of motorization is higher than the national average; most families have a car at least. (Table 1-4) In this district mean daily total freightage is 1.8 million tons and 90 % of these freight is

transported by truck between the National Capital region and the Tohoku District, or in various part of the Tohoku District.

In the Tohoku District, passengers and freight rely on vehicular traffic. Among roads, national highways generally have an important function in traffic. In and around cities, the dependency on national highways is significant. The traffic just passing through the area are congested with the internal traffic, in other words motor traffic and truck traffic are getting along with each other; therefore, the traffic on the roads has been increasing.

Table 1-5 shows the average traffic volume in the direct controlled national highways. The character of traffic flow is described as follows: there are heavy traffic volume in the cities, and the ratio of truck is large in the rural areas. The most important task in the Tohoku District

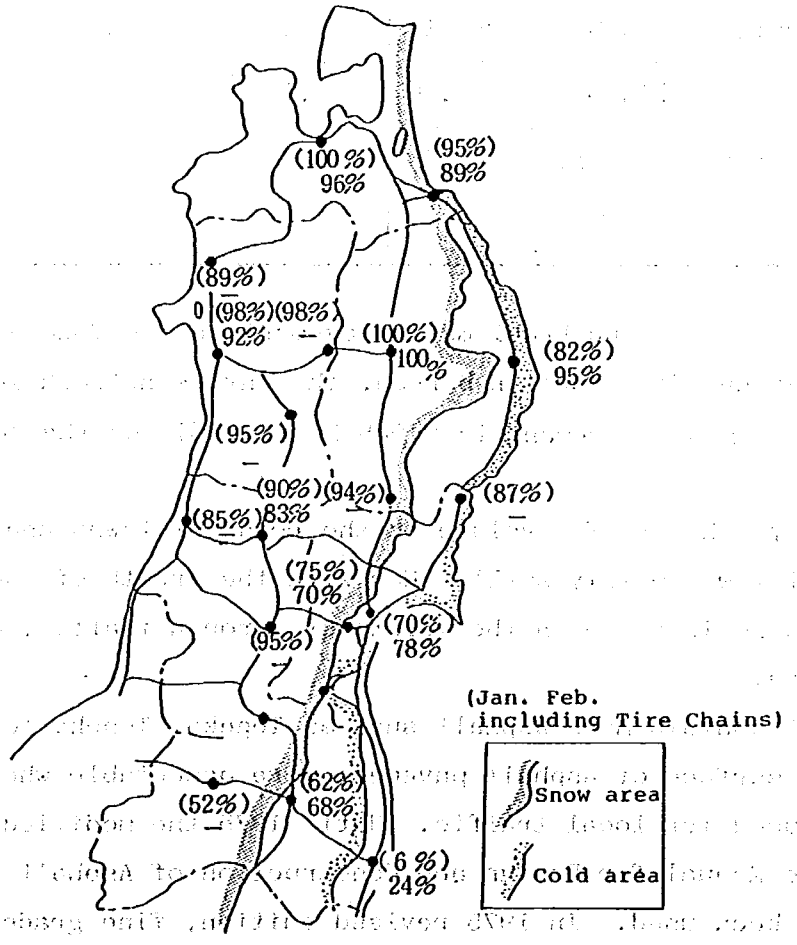


Fig. 1-5 Rate of Studded Tire Equipping During Winter

is to retain the traffic during the winter. Generally speaking, the snowfall season is from December to March. About half of the length of national and prefectural roads have secured two-lanes width by snow-removing work. Snow melting agents are spreaded over steep slope sections in the principal roads. However, during this period all vehicles equip studded tires or tire-chains in snow areas. Especially, the use of studded tires is recently increasing because of convenience. Fig. 1-5 shows the rate of studded tires equipping in major roads.

Table 1-5 Average Traffic Volume in National Highway under Direct Control of the Ministry of Construction

Place	Average Traffic Volume	Classify (%)			
		Car	Bus	Small Truck	Truck
D I D	car/day 30,202	59.2	2.4	24.9	13.5
City of other	16,909	53.0	1.5	27.0	18.5
Flat Country	11,843	46.9	1.5	27.5	24.2
Hilly Country	6,286	45.2	2.0	25.4	27.5
Total	12,878				

1-3. Characteristics and Problems of Pavement in the Tohoku District

In Tohoku Regional Construction Bureau, the entire network length is about 2,500 Km; the asphalt pavement is 2,045 Km (81.8%) and the concrete pavement is 455 Km.

Until recently, the traffic volume on the roads of towns and cities in the Tohoku District was mainly small. Now with the growth of the area there has been a large increase in the volume of through traffic, especially in the snowy area.

Mixtures that contain much asphalt such as Topeka, Topeka for cold area, etc., as the surface of asphalt pavement were acceptable when the roads were mainly used for local traffic. Until 1975 the modified Topeka recommended in the Manual for Design and Construction of Asphalt Pavement (1967 edition) has been used. In 1975 revised edition, fine graded asphalt concrete was recommended, so this asphalt concrete is being used today. Since studded tires have become increasingly popular as an aid for winter driving, rutting due to wearing has been remarkable. Road administration

needs to consider a countermeasure against wearing.

The change of surface conditions in the Tohoku District from the standpoint of the cracking ratio and rutting depth is shown in Table 1-6, 1-7. According to cracking ratio in Table 1-6, the pavement is not damaged so much as a result of maintenance. It seems that the extension of damaged section is prevented by repair. (Cracking have mainly occured in local and no low-temperature shrinking has been observed.)

Table 1-6 Length of Cracking Ratio Category

craking ratio	0 ~ 5%	5 ~ 20%	20 ~ 40%	40% ~	Total
1979	(80.9%) 1,485 km	(13.5) 248 km	(4.2%) 78 km	(1.4%) 25 km	(100%) 1,836 km
1984	(81.0%) 1,663 km	(15.5%) 319 km	(2.7%) 55 km	(0.8%) 18 km	(100%) 2,055 km

Table 1-7 Length of Rutting Depth Category

Rutting depth	0 ~ 20mm	20 ~ 35 mm	30 ~ 50 mm	50 mm ~	Total
1979	(16.6%) 304 km	(49.0%) 899 km	(27.4%) 503 km	(7.0%) 130 km	(100%) 1,836 km
1984	(21.1%) 434 km	(40.8%) 838 km	(25.7%) 548 km	(11.4%) 235 km	(100%) 2,055 km

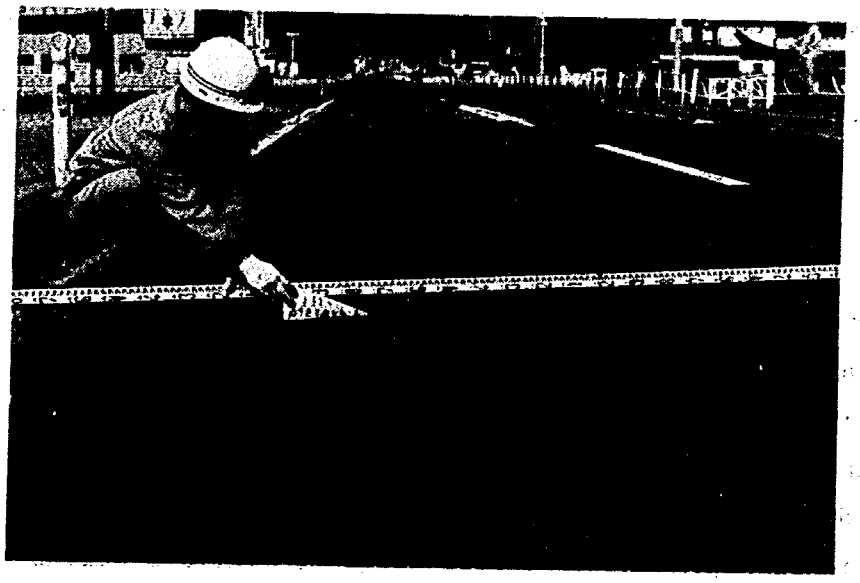


Photo-1 Wearing of Asphalt Pavement

Nevertheless, rutting is increasing in any sections in spite of repairs every year.

Pavement thickness (T_A) is satisfied to a certain degree (80% of target T_A) because of repair, therefore the occurring of cracking is relatively restricted. However, because of incresing use of studded tires the wearing on pavement surface is remarkable in spite of repair (Fig. 1-6)

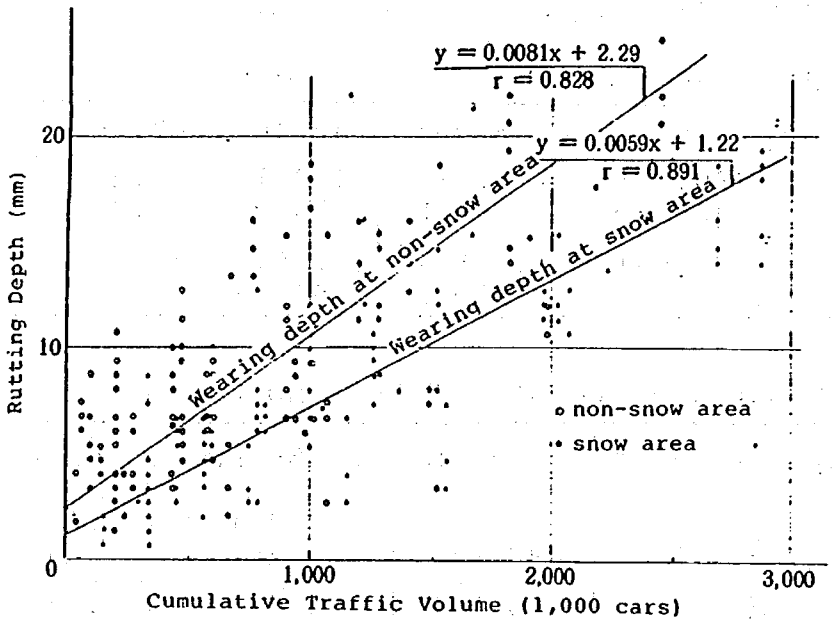


Fig. 1-6 Wearing Depth in Asphalt Pavement

In ordinary national highways administered by the Tohoku Regional Construction Bureau, the average of wearing depth reached 2.54mm and the expenses for repair amounted to seven billion yen per year. (About 10 % of the entire network)

In the meantime, the wear due to studded tires causes a lot of wear dust, and this environmental pollution has become a new social problem in the cities in the Tohoku District.

2. Countermeasure for Wear and Plastic Flow in Asphalt Pavement

2-1. Labolatory Investigation of Surface Course Mixture

In the territory of the Tohoku Regional Construction Bereau there are various climatic conditions (such as temperature, rainfall and snowfall), traffic condition, etc., depending on the locality.

Therefore, it is important to choose adequate mixtures suitable to the local conditions.

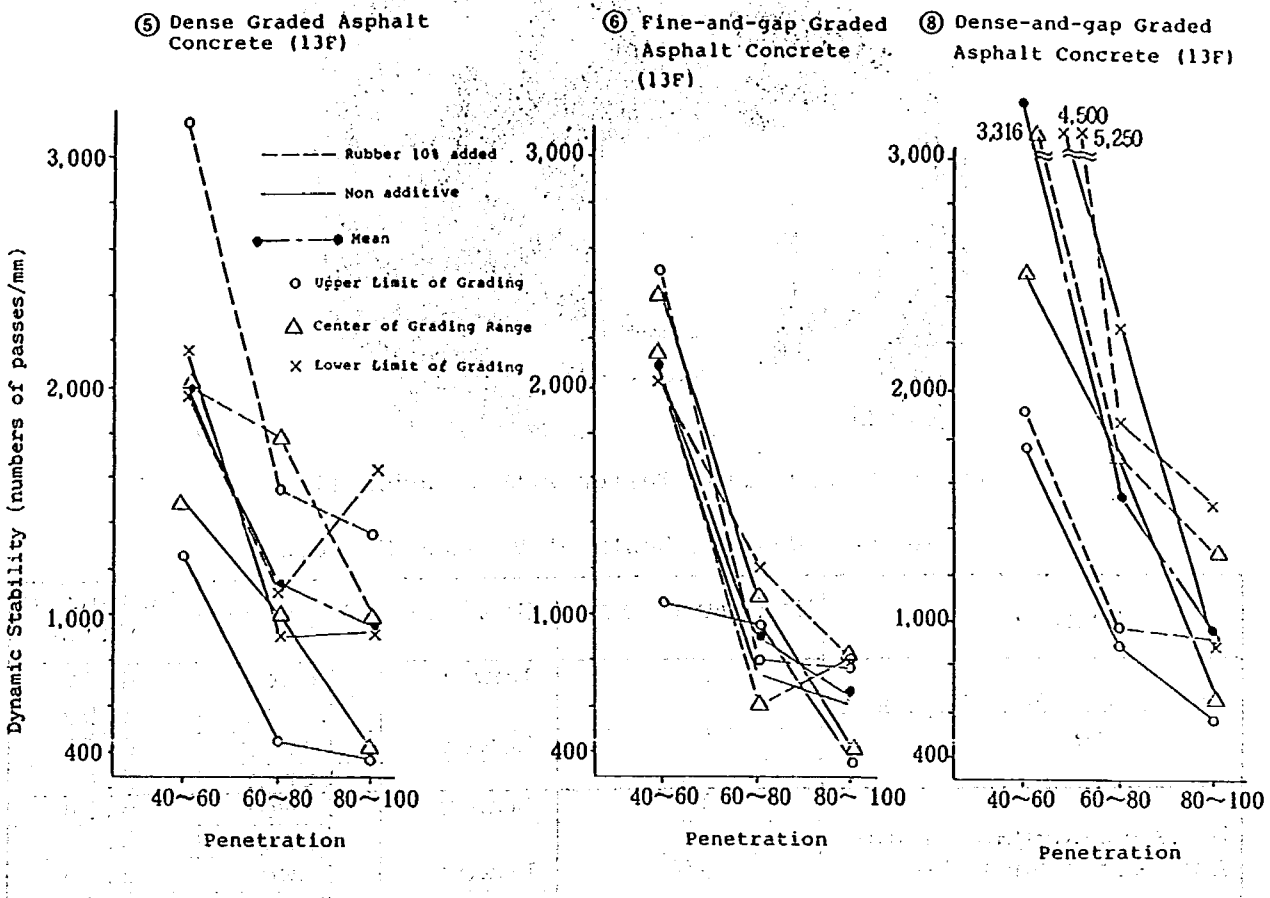


Fig. 2-1 Effect of Penetration on Dynamic Stability for Various Mixtures

In this report, several laboratory studies for various surface course mixtures used in snow areas were carried out, in order to select surface course mixtures suitable to local conditions in the Tohoku District.

The dynamic stability (Whell Tracking Test) for various mixtures are shown in Fig. 2-1.

Summary of the laboratory test results is that:

The dynamic stability inclines to increase getting toward lower limit of gradation. When the lower penetration asphalt is used, the dynamic stability increases.

Table 2-1 shows the survey data conducted by Miyagi Pref. using Rotary

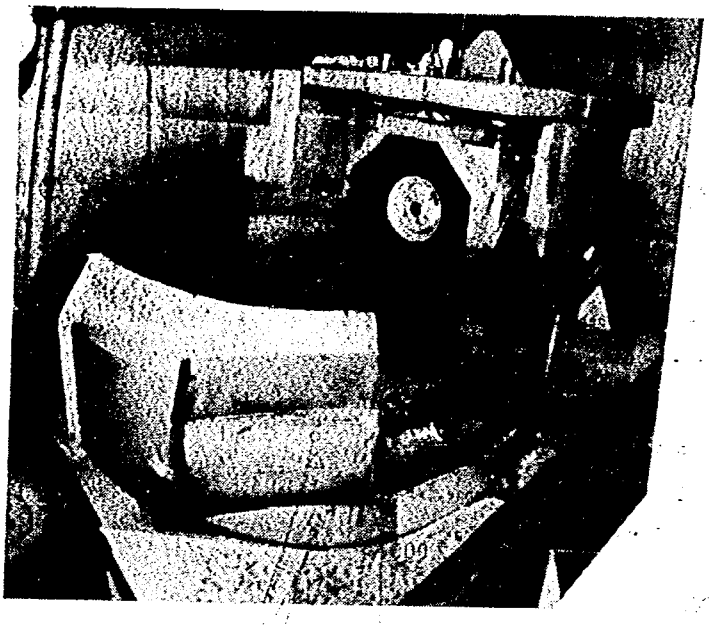


Photo-2 Ravelling Tester with Studded Tire

Table 2-1 Wearing Depth of Various Mixture

Mixture	Maximum Size	Grading	Asphalt	Measured Value
Dense Graded Asphalt Concrete	13	Lower	Lower	6.72
			Upper	5.92
	20	Lower	Lower	6.68
			Upper	5.69
Dense-and-gap Graded Asphalt Concrete	13	Lower	Lower	6.24
			Upper	5.51
	20	Lower	Lower	6.13
			Upper	5.32
Fine-Graded Asphalt Concrete	13	Upper	Lower	9.78
			Upper	9.27
	20	Upper	Lower	9.24
			Upper	8.98
Fine-and-gap Graded Asphalt Concrete	13	Upper	Lower	8.45
			Upper	7.50
	20	Upper	Lower	7.95
			Upper	7.41

Ravelling Simulation Test. (Using studded tire)

Amount of wear is measured after 20,000 passes on ravelling specimen.

For the fine graded asphalt concrete, wearing is much more and damage of the surface is remarkable.

Wearing of fine-and-gap graded asphalt concrete is less than that of the fine graded asphalt concrete.

Therefore, recently the fine-and-gap graded asphalt concrete have been widely used in the snowy and cold areas.

Compared with the dense graded asphalt concrete, wearing of the dense-and gap graded asphalt concrete used in the ordinary area is less.

Further, conserning maximum aggregate size, 20 mm top asphalt concrete is superior to 13 mm top asphalt concrete in wear-resistance. This has been demonstrated by the test for practical use.

The mix proportion and aggregate grading for each mixture are shown in Table 2-2, Fig. 2-1 respectively.

Table 2-2 Combination of Various Mixture (Example)

Material / Mixture		Coase Aggregate			Fine Aggregate		Filler	Asphalt	Total
		20 ~ 13	13 ~ 5	5~ 2.5	Coase Sand	Fine Sand			
Dense Graded Asphalt Concrete	13F		46.8		29.0	9.4	8.4	6.4	100
	20F	18.9	21.4	8.6	20.4	16.2	8.3	6.2	100
Fine-and-gap Graded Asphalt Concrete	13F		23.2	18.1	11.1	30.2	10.2	7.2	100
	20F	14.5	15.9	11.2	13.1	27.1	11.6	6.6	100
Dense-and-gap Graded Asphalt Concrete	13F		42.9	18.8	-	22.1	10.4	5.8	100
	20F	20.3	22.1	18.9	-	22.6	10.4	5.7	100

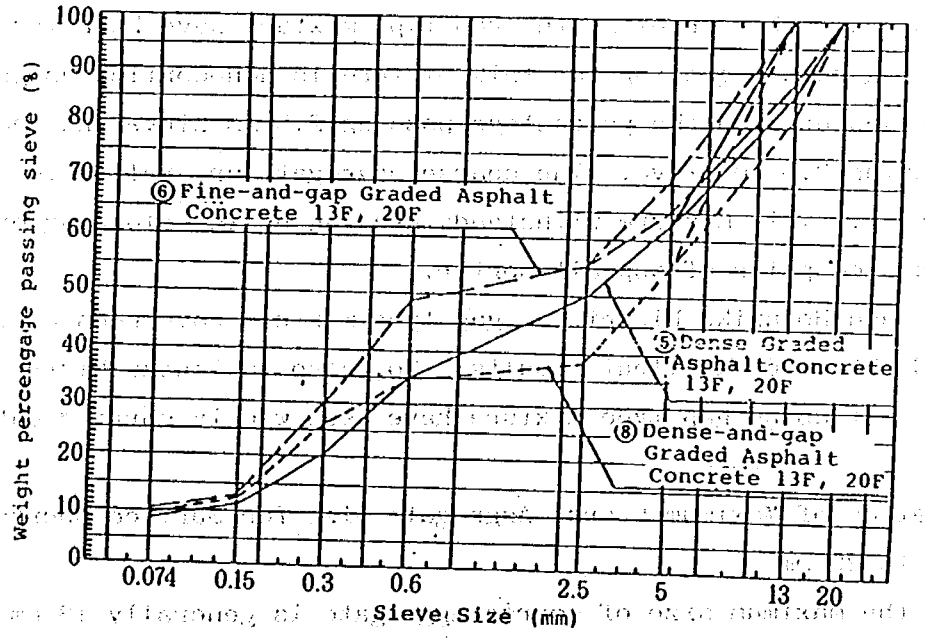


Fig. 2-1 Aggregate Grading of Various Mixture (Example)

2-2. Investigation by Experimental Test with Regard to Flow and Wearing

The laboratory tests to obtain the dynamic stability (Wheel Tracking Test) and the wear depth (Ravelling Test) have been carried out for various mixtures. Then the mixtures that obtained numerically significance in these tests have been employed.

However, with increasing use of studded tires the difference between the results of laboratory test and field observation have arised. The influence of studded tires on pavement surface is inclined to exceed the results of laboratory test, so it is also necessary to improve the testing apparatus. Therefore, the Tohoku Regional Construction Bureau noticed that it was difficult to cope with the actual condition only by the laboratory test and have carried out test for practical use during the past several years.

(1) Alteration of Mixture from Fine Graded Asphalt Concrete to Fine-and-Gap Graded Aspahlt Concrete

Fine graded asphalt concrete and Topeka that contain much asphalt and have smooth grading curve have been commonly used in snowy area. However, in heavy traffic (C,D: road classification by traffic) the flow has been remarkable in these mixtures. Also with regard to wear-resistance these mixtures cannot sufficiently withstand the studded tires aside from the tire chains. Thus, this has become a big trouble for road administration. In order to solve this problem, the gap type mixture have been investigated. The aggregate grading for this mixture is non-continuous gradation which has a gap between the percentage passing 2.5 mm sieve and the percentage passing 0.6 mm sieve. The coarse aggregate are rolled into the asphalt mortar. This mixture withstands wearing by interlocking of coarse aggregate and coupling of asphalt mortar.

In Hokkaido and Hokuriku Districts, gap type mixture have been early applied and obtained excellent results. So, also in Tohoku District, the full scale test using gap type mixture have been widely carried out since 1981.

(2) Alterlation of Maximum Coarse Aggregate Size for Surface Course Mixture from 13 mm to 20 mm

Presently, the maximum size of coarse aggregate is generally 13 mm for surface course mixture except for thick layers. However, under heavy traffic the mixtures that use former maximum size 13 mm cannot satisfactorily withstand against flow and studded tires. Thus, the application of maximum size 20 mm has been investigated. The results of ravelling test,

in which the testing machine equipped the real of studded tires, also indicated the advantage of using larger maximum size against wearing.
In the Tohoku District, experimental study using maximum aggregate size 20 mm as the surface course (5 cm) has been made in some area, the surface course mixtures suitable to local conditions will be determined in 1985.
In the case of the surface treatment such as thin overlay (3 cm), maximum aggregate size is restricted by the thickness of a layer (about 1/2.5 of thickness), so maximum size 13 mm is used.

(3) Application of Penetration 60/80 Asphalt

Lower penetration asphalt was selected, such as pen. 60/80 (partially pen. 40/60 was used in trial) instead of pen. 80/100 heretofor in use. The advantage of pen. 60/80 asphalt to pen. 80/100 asphalt was recognized from the results of the full scale test against plastic flow and wearing. Recently, lower penetration grade asphalt and hard asphalt are getting in practical use. One of the mix design condition for the asphalt concrete mixture is that relatively hard asphalt is used in order to increase the binding of coarse aggregates.

(4) Early Repair Using Thin Layer Overlay

According to the report by Public Works Research Institute concerning maintenance and repair of pavement, if the total cost of pavement is linked with maintenance level, thin layer overlays in early time are better than other maintenance procedure. Based on this concept, the thickness of overlay for repair is 5 cm and the thickness of overlay against wear is 3 cm, instead of the former two layer overlay (binder course 4 cm + surface course 3 cm). In the case of damage caused by flow, generally the cold cutting method is applied.

(5) Compaction and Construction Season

To get wear-resistance and flow-resistance surface mixture, it is necessary that the degree of compaction of mixture satisfies specifications. However, recently hard asphalt with low penetration (Pen. 60/80) and maximum aggregate size 20 mm are used. Therefore, it is necessary to consider the construction method and the season in order to satisfy the degree of compaction.

(6) Repair Method of Rutting

In order to carry out the many requered repaires on a limited budget the repair method used is to put the mixture to only the wheel tracks that has been damaged due to wear.

(7) Surface Recycling, Steel Slag Asphalt Concrete

Recently, surface recycling has come to public notice as the repair method that uses the asphalt pavement material economically and effectively. Since steel slag asphalt concrete is excellent in flow and wear, the slag asphalt concrete is experimentally applied in a few areas.

3. Systematization of Repair and Maintenance for Asphalt Pavement

3-1. Survey System of Surface Characteristics

In order to carry out the effective maintenence of pavement within the limited budget, it is indispensable to survey actual condition of pavement in the entire network and recognize the change of the condition. Tohoku Regional Construction Bureau has carried out condition survey using the photo inventory technique since 1974. Here, we introduce this system. (Fig. 3-1) This system concerns systematic survey and data analysis about surface characteristics. It has been developed to provide information for the maintenance and repair of road surface.

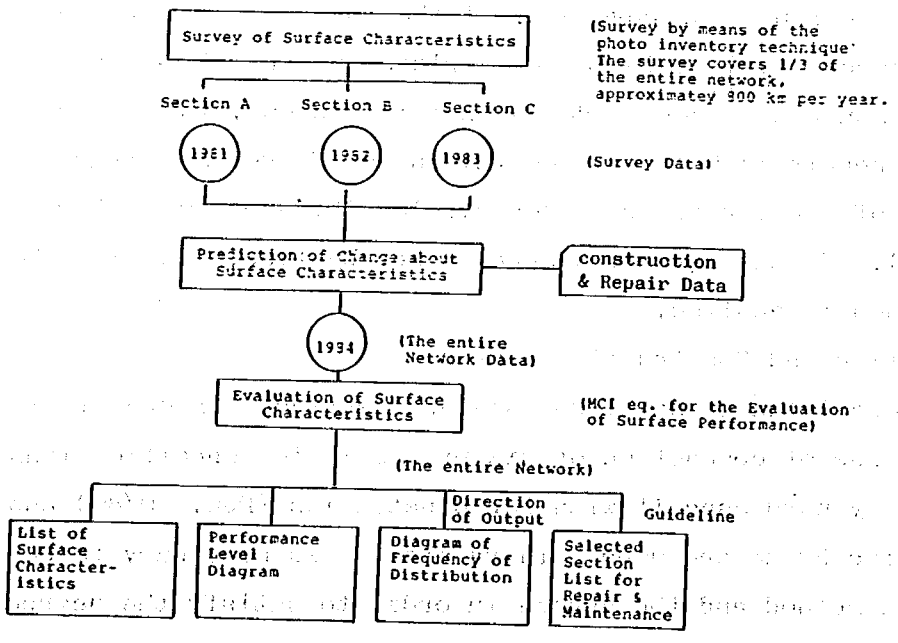


Fig. 3-1 Flowchart for Survey System of Surface Characteristics

Three items such as "Cracking", "Rutting Depth" and "Evenness" are measured as representative values of road surface characteristics. The survey covers 1/3 of the entire network per year. The survey is conducted on one representative traffic lane of each road.
Photographic measurement is made by the vehicles that install the photographing device and measuring equipment.
Data are obtained in numerical form from each unit section (100 m).

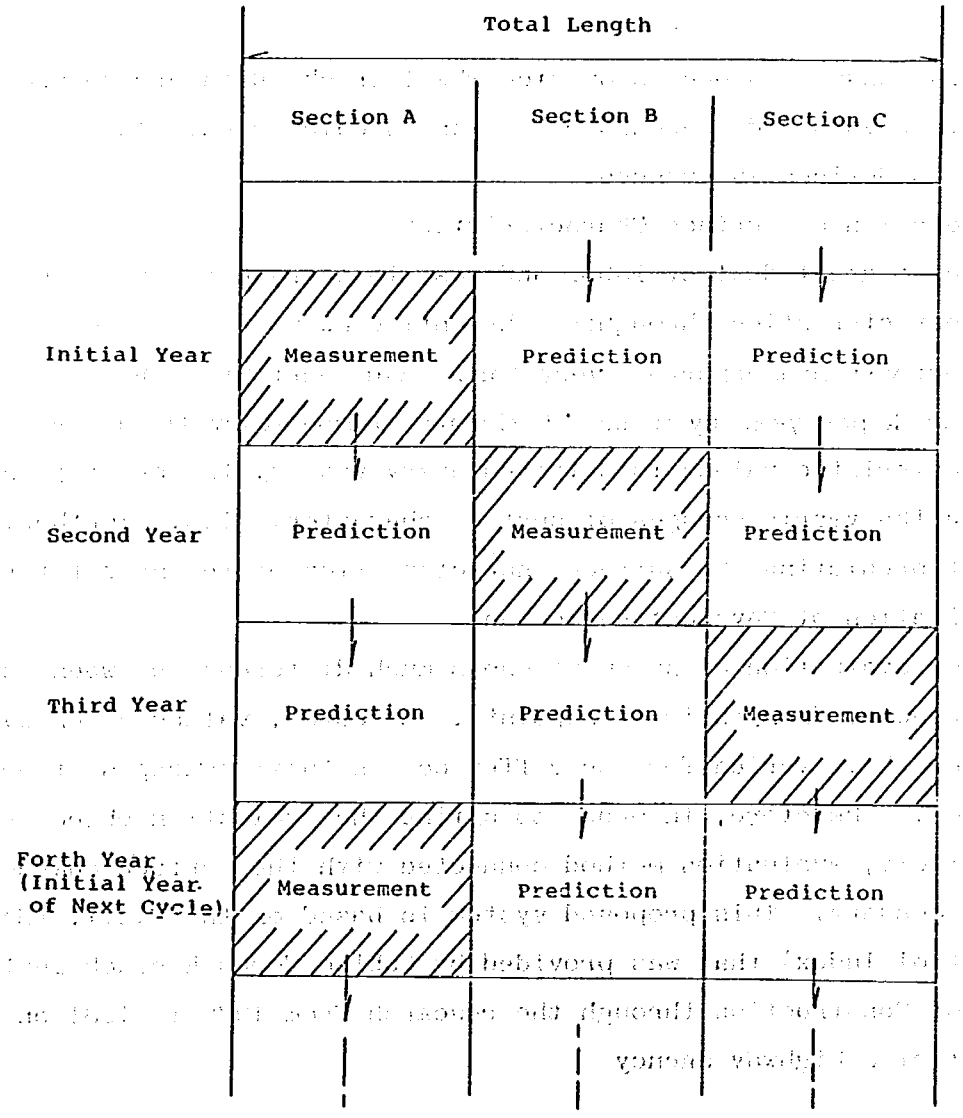


Fig. 3-2 Cycle of Photographic Survey of Road Surface

∘Cracking

Cracking area, patching area and cracking length are measured by the analysis of photographs. Percentage of cracking or degree of cracking is calculated by the following equations.

$$\text{Cracking Ratio} = \frac{\text{cracking area } (m^2) + \text{patching area } (m^2)}{\text{total area } (m^2)} \times 100\ (\%)$$

∘Rutting Depth

Rutting depth is measured in 5 cross sections at interval of 20 m per unit section by means of a photographic device. Maximum rutting depth is obtained from each cross section. Maximum, mean and standard deviation are calculated from those 5 values.

∘Evenness

Evenness is measured along an outside wheel track Data are obtained at constant intervals and standard deviation is calculated. This standard deviation is defined as evenness.

3-2. Prediction of Surface Characteristics

It requires a great deal of labor and cost to carry out the survey of surface characteristics throughout the entire network every year. Therefore, the survey is made once every three tears and cover about 1/3 of the entire network per year by means of the pgoto inventory technique. (Fig. 3-2) Each characteristic values is obtained every year by the prediction model concerning the yearly changes of surface characteristics. Further, it is capable of predicting the surface characteristics of the near future.

3-3. Evaluation of Pavement Performance

Performance evaluation of pavement shows much difference between users and highway agencies from their standpoints. Moreover, values given by a group of observers have varied for the difference in their concepts of desirable performance. Therefore, in order to define the evaluation objectively and quantitatively, evaluation method connected with the surface characteristics is necessary. This proposed system is based on the M.C.I. (Maintenance Control Index) that was provided by Public Works Reseach Institute, Ministry of Construction through the research from 1979 to 1981 on the standpoint of a highway agency.

$$MCI = 10 - 1.48C^{0.3} - 0.29D^{0.7} - 0.47\sigma^{0.2} \quad \text{.....(1)}$$

$$MCI_0 = 10 - 1.51C^{0.3} - 0.30D^{0.7} \quad \text{.....(2)}$$

where

MCI : Maintenance Control Index

C : Cracking Ratio (%)

D : Rutting Depth (mm)

σ : Evenness

Eq. (1) is basically applied to the performance evaluation. However, presently MCI_0 eq. (2) is likely used because data of evenness are not obtained from whole area. The value of MCI is calculated on a scale of 1 to 10. The more the evaluation for the road surface decreases the more the valu of MCI decreases. Generally speaking, the criteria for repair and maintenance is described as follows: (Table 3-1)

Table 3-1 Calculating Table for MCI_0

MCI_0	Cracking Ratio (%)	Rut Depth (mm)
10	0	0
5	5	20
5	20	10
4	5	40
4	30	20
3	20	40
3	50	30
2	50	50

Recommendation
MCI:
5 or less
Desirable to repair
4 or less
Need to repair
3 or less
Need to repair soon

3-4. Output Format of Surface Characteristic Data

(1) List of Surface Characteristic Data

The various data are printed out for each unit section (100 m) of the entire network. (Table 3-2)

(2) Performance Level Diagram

The various characteristics and performance levels are diagramed. (Fig. 3-3)

(3) Histogram

The various data such as cracking, rutting depth, MCI and so on are totalized in each interval, and these data can be printed out in each work office or all over the network of Tohoku Regional Construction Bureau, etc.

(1) Koriyama National Highway Work Office
(1) Koriyama Maintenance Work Branch

Kilopost	Brake	Type of Pavement Structure	B: Bridge T: Tunnel	Section Length	Measurement: Survey Year	Cracking Ratio	Patching Ratio	As: Cracking Ratio (%) Co: Cracking Index(cm/m²)	Maximum Rutting Depth(mm)	Average of Rutting Depth	Rutting Depth, M + S	Evenness	Prediction 1984: Maintenance and Control Index MCI_0 (Cracking, Rutting Depth)	Cracking Ratio	Maximum Rutting Depth	Average Rutting Depth	Maintenance and Control Index	Initial Construction Year	Rehabilitation Year	Method of Rehabilitation	Year
232•400—232•500		AS		100	S54	0.	0.	0.	25				7.1	0.	37		6.2	1962	1977	R	5
232•500—232•600	(Mark *)	AS		100	S54	0.	0.	0.	38				6.7	0.	31		5.3	1962	1977	R	5
232•600—232•700		AS		100	S54	0.	0.	0.	29				6.8	0.	41		5.9	1962	1977	R	5
232•700—232•800		AS		100	S54	0.	1.3	1.3	23				5.7	1.3	34		4.8	1962	1977	A	5
232•800—232•900		AS		100	S54	6.2	22.7	28.9	41				1.8	67.9	36		0.3	1962	1976	N	6
232•900—233•000		AS		100	S54	7.3	0.9	8.8	31				3.8	30.6	42		1.7	1962	1976	N	6
232•000—233•100		AS		100	S54	10.8	0.	10.3	29				3.8	0.	17		7.8	1962	1981	W	1
232•100—233•200		AS		100	S54	2.4	0.	2.4	26				5.1	0.	17		7.8	1962	1981	W	1
232•200—233•300		AS		100	S54	12.7	0.	12.7	29				3.6	39.2	41		1.6	1962	1976	H	6
232•300—233•400		AS		100	S54	8.6	0.2	8.8	23				4.4	32.8	34		2.1	1962	1976	H	6
232•400—233•500		AS		100	S54	1.0	0.	1.0	43				4.3	0.	17		7.8	1962	1981	W	1
232•500—233•600		AS		100	S54	2.1	0.	2.1	25				5.3	0.	17		7.8	1962	1981	W	1
232•600—233•700		AS	R	100	S54	2.4	0.	2.4	23				5.3	0.	17		7.8	1962	1981	W	1
232•700—233•800		AS	R	100	S54	2.5	0.4	2.9	39				4.0	0.	17		7.8	1962	1981	W	1

N: New Construction
A: Reconstruction
R: Overlay
H: Surface Treatment
W: Repair of Rutting Depth
C: Surface Cutting

Table 3-2 List of Surface Characteristics Data (Legend)

Fig. 3-3 Performance Level Diagram, 1984

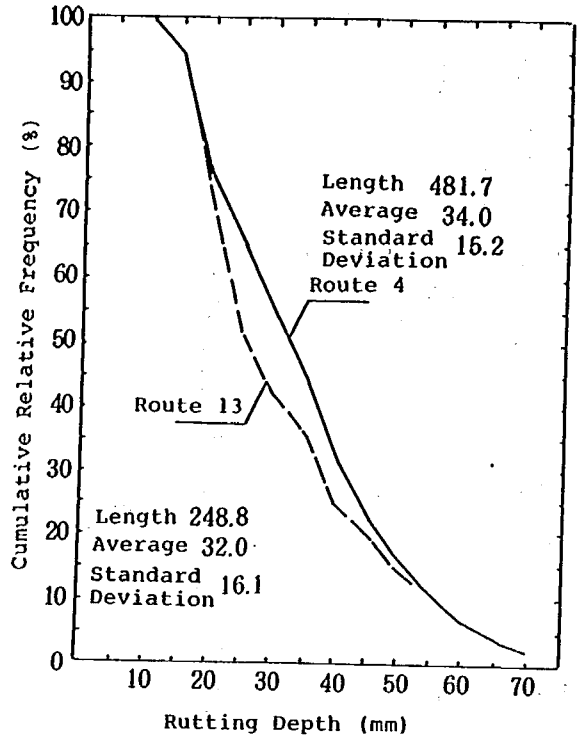


Fig. 3-4 Cumulative Relative Histogram for Rut Depth

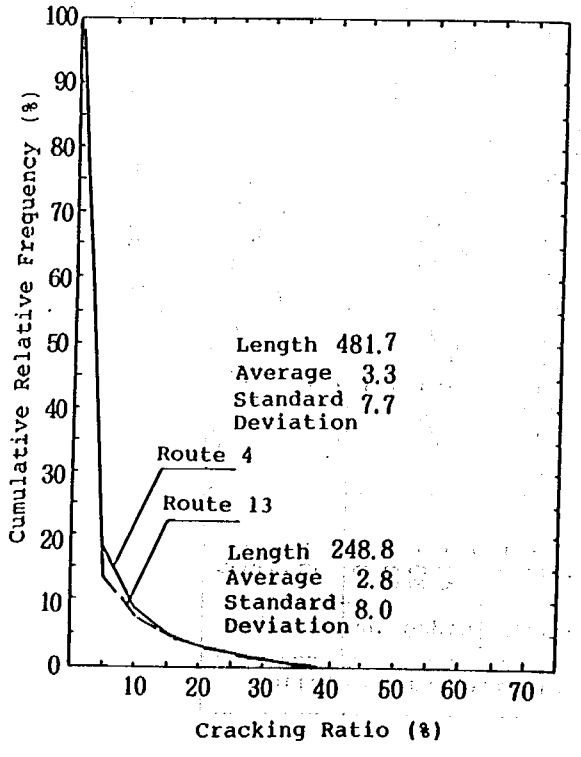


Fig. 3-5 Cumulative Relative Histogram for Cracking

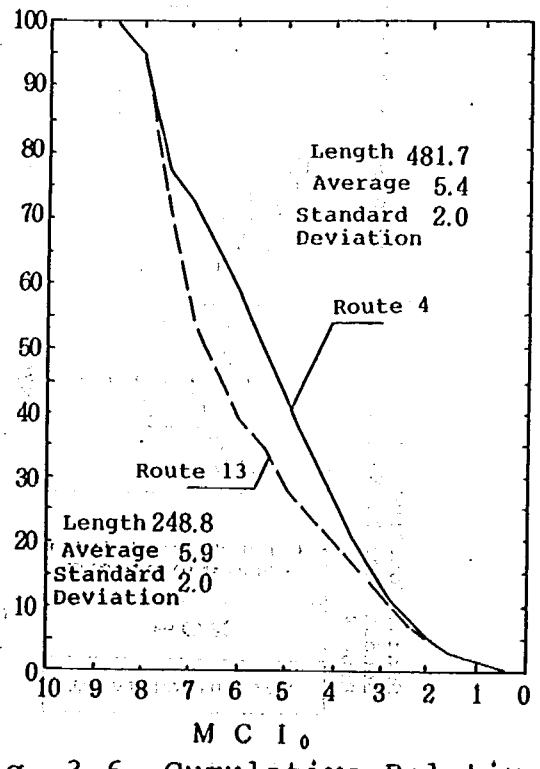


Fig. 3-6 Cumulative Relative Histogram for MCI_0

The surface conditions in the whole network are macroscopically noted from the distribution of surface characteristics. These data are useful for providing the information for the future maintenance plan of pavement surface. (Fig. 3-4, 3-5, 3-6)

(4) Selected Section List for Repair and Maintenance

After setting up the guideline for repair and maintenence about MCI, rutting depth, etc., the selected sections are printed out. Sections for maintenence are chosen based on these data considering route priority, noise, vibration, the environmental condition of the surrounding area, etc. (Table 3-3)

(5) Renewal of Construction and Maintenance Data

The survey on a certain route is carried out once every three years and data are estimated by the prediction model in the other two years. Thus during this period any information concerning repair and maintenence or new pavement construction, such as bypass, must be input as repair and maintenance data. If repairs and maintenence are carried out and there is no input, the values by which prediction model are estimated will not reflect the real condition.

Table 3-3 Selected Section List for Repair & Maintenance

Work Office 1 (Koriyama National Highway)

Work Branch 2 (Aizuwakamatsu)

Route No. 49

KP 2 ..... KP 2	Length (m)	Cracking	Maximum Rutting Depth	Average Rutting Depth	Evenness	Evaluation value (M C I o)
139 • 685 – 140 • 000	315	33.4	45	...	...	1.36
105 • 100 – 105 • 600	500	9.8	45	...	...	3.28
125 • 200 – 125 • 700	500	10.6	33	...	...	3.49
109 • 500 – 110 • 000	500	7.3	44	...	...	3.49
109 • 000 – 109 • 500	500	11.7	45	...	...	3.55
99 • 500 – 100 • 000	500	6.7	42	...	...	3.99
126 • 300 – 126 • 800	500	4.0	41	...	...	4.38
106 • 400 – 106 • 900	500	2.7	38	...	...	4.73
... Subtotal ...	3815	The average value over the interval				↓

The output is listed in a progression from the worst to the best conditions.

By using this system, we can recognize the real surface condition throughout the whole network. In the future these data will be used to provide the information for pavement rehabilitation plans such as overlay, etc. At present, this system only deals with the pavement surface. However, we intend to make a total data bank including T_A, CBR, pavement structure, repair history, etc.

4. List of Repair Method for Asphalt Pavement

Roads are facility for motor vehicle tranportation and coutinuously support heavy load. Therefore, the damage and degradation are inevitable. Especially, the pavement surface is exposed under severe conditions owing to increasing traffic volume and heavy vehicles. Further, the wear of pavement surface is recently remarkable due to the use of studded tires. This has become one of the problems of road maintenance. After the pavement performance is obtained from the survey of surface conditions, adequate repair and maintenance are applied in order to restore good performance. In this paragraph, the repair method corresponding to surface conditions are summerized as shown in Table 4-1.

5. Afterword

During the past 10 years, studded tires have become increasingly popular, moreover studded tires for the commercial vehicles have been developed at present. It is said that the effect of studded tires of heavy vehicles are several times as great as that of light vehicles. And now, the wear of pavement surface in snowy areas and the countermeasure of wear dust in the cities have become big social problems.

It is necessary for road administrator to find out immediately the surface course mixtures suitable to both wear caused by studded tires in the winter and plastic flow in the summer, in order to maintain good surface condition.

Thus, the investigation-based data in long-term derived from laboratory tests and field surveys has been continued. At the same time, the research of surface course mixture has been made by practical experiments.

However pavement performance should be determined by long-term data, this report is based on data and concepts at present.

Summary of this report are as follows:

(1) The priority of maintenance and repair for pavement surface is deter-

mined by a comprehensive evaluation, adopting MCI as the survey system of surface conditions.

(2) Based on the laboratory test (Ravelling Test using studded tire, etc.) and the test for practical use in each area, and taking wear and flow into consideration, the following methods are employed:

1 Use of fine-and-gap graded asphalt concrete in snowy areas and dense-and-gap asphalt concrete in ordinary areas.

2 Use of the aggregate with maximum size 20 mm, after the pavement thickness is checked according to Table 4-1.

3 Use of harder penetration asphalt (60/80 or 40/60)

4 Application of short interval maintenence by thin overlays

(3) As the effective repair method, the following items are considered:

1 The required degree of compaction (more than 98% of the density of laboratory test)

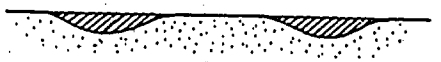
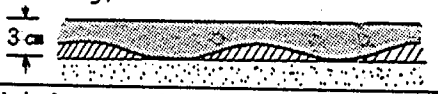
2 Repair method of rutting (to the part of outside and inside wheel track)

3 Experimental application of surface recycling, steel slag asphalt and so on.

As is mentioned in the environmental pollution problem caused by studded tires, ability of pavement to withstand wearing has a certain limit. In the future it seems that a comprehensive countermeasure is required including restricting the period for use of studded tires and developing a new tires for safe driving in winter as a substitute for the present studded tires.

In order to obtain maximum effect within the limited budget, we must continue further research and seek an adequate pavement suitable for changing times.

Table 4-1 Surface Conditions and Maintenane

	Present Surface Condition		Total Evaluation (M.C.I.)	Construction Method	Remarks
(1)	Wear (partially Rutting depth along wheel track due to studded tires, etc.	Rutting depth along wheel track due to studded tires, etc.	$MCI_0 = 5$	Repair only part of rutting depth using roadfinisher. 	Same as former laveling method. In part combined with surface cutting.
(2)	Wear (widely spreaded)	Polishing Asphalt concrete aging & scaling	$MCI_0 = 5$	Thickness 3 cm. Overlay using roadfinisher.	In case cracking is greater than 20 30%, reconstruction is partially required.
(3)	Wear (Wear & Rutting)	Occurred in part. Method (2) does not apply to this case.	MCI_0 is less than 5, approx. 4	Thickness 3 cm. Overlay using roadfinisher. (Combined with surface cutting) 	Chiefly used because of wear & aging of surfacings. In case surface cutting is acceptable.
(4)	Rutting & Cracking	Rutting depth is remarkable. (50~60mm) Methods (1) & (2) does not apply to this case.	$MCI_0 = 4$	Thickness 5 cm. Pavement thickness is yet greater than 80% of T_A or thickness is greater than 80% after 5cm overlay.	Chiefly used to shorten interval of overlay. Thickness satisfies target T_A.
(5)	Cracking & Rutting	Cracking is remarkable. (40~50%) Methods (1)-(4) does not apply to this case.	$MCI_0 = 4$	Max. 10 cm. Select pavement thickness in order to satisfy 80% of T_A.	Pavement damage caused by shortage of T_A. Overlay thickness is determined in order to satisfy 80% of T_A. Two layer construction.

In Reference to Pavement of Direct Controlled National Roads in the Hokuriku District

Mr. T. Nanbu, Mr. T. Funada
Hokuriku Regional Construction Bureau, Ministry of Construction

ABSTRACT

Hokuriku Regional Construction Bureau of Ministry of Construction control national roads which go through the areas where there is heavy snow. These papers report the actual condition about the management of road surface and repair works in these areas which are worn down hard. The area where approximately 1,000 km roads go through is under the management of Hokuriku Regional Construction Bureau of Ministry of Construction, and is prominent area of a heavy fall of snow in the world, and there are places where the volume of snow reaches 20 m depending on years. Nevertheless there are not many places where minimum temperature reaches below 0°C, and as a result of frequent snow removing, road surface is exposed in wet condition in a long term. Paving is mostly completed with asphalt and average traffic amount is about 10,000 cars a day. About 90% of full roads are two lines for going to and from, and almost all the cars fix spike tires or chain.

The situation of this weather condition and road surface and traffic cause hard defacement of paved roads. Hokuriku Regional Construction Bureau adopts gap asphalt in all the places in order to take measures for the defacement in limited budget and make efforts to get lower priced repair works as an emergent measure by developing engineering with which they full up the parts that are defaced by the prints of wheels.

Also they take photographs of road surface within the jurisdiction every 3 years, and compute barometer named MCI, and make use of it for planned management of roads.

And also they tentatively execute PC board pavement and repair work using thin layer of concrete as measures to defacement.

IN REFERENCE TO PAVING OF DIRECT CONTROLLED NATIONAL ROADS IN THE HOKURIKU DISTRICT

by

Takaaki Nanbu

and

Toshi Funada

Hokuriku Regional Construction Bureau of Ministry of Construction

The area where approximately 1,000 km roads go through is under the management of Hokuriku Regional Construction Bureau of Ministry of Construction and is prominent area of a heavy fall of snow in the world, and there are places where the volume of snow reaches 20 m depending on years. Nevertheless there are not many places where minimum temperature reaches below $0^{o}C$, and as a result of frequent snow removing, road surface is exposed in wet condition in a long term. Paving is mostly completed with asphalt and average traffic amount is about 10,000 cars a day. About 90% of full roads are two lines for going to and from, and almost all the cars fix spike tires or chain.

The situation of this weather condition and road surface and traffic cause hard defacement of paved roads. Hokuriku Regional Construction Bureau adopts gap asphalt in all the places in order to take measures for the defacement in limited budget and make efforts to get lower princed repair work as an emergent measure by developing engineering with which they full up the parts that are defaced by the prints of wheels.

Also they take photographs of road surface within the jurisdiction every 3 years, and compute barometer named MCI, and make use of it for planned management of roads. And also they tentatively execute PC board pavement and repair work using thin layer of concrete as measures to defacement.

These papers report the actual condition about the management of road surface and repair works in these areas which are worn down hard.

1. Characteristic of weather

1) Situation

As is indicated in Fig-1 the Hokuriku District is a distinguished area of a heavy fall of snow in Japan, and when we think of the heavy volume of snowfall and the largeness of population, the necessity of snow removing and protection against snow and their difficulty may be the greatest in the world.

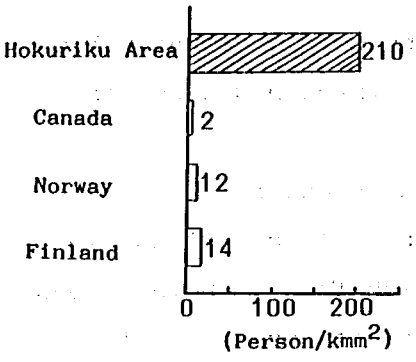


Fig-1 Population Density

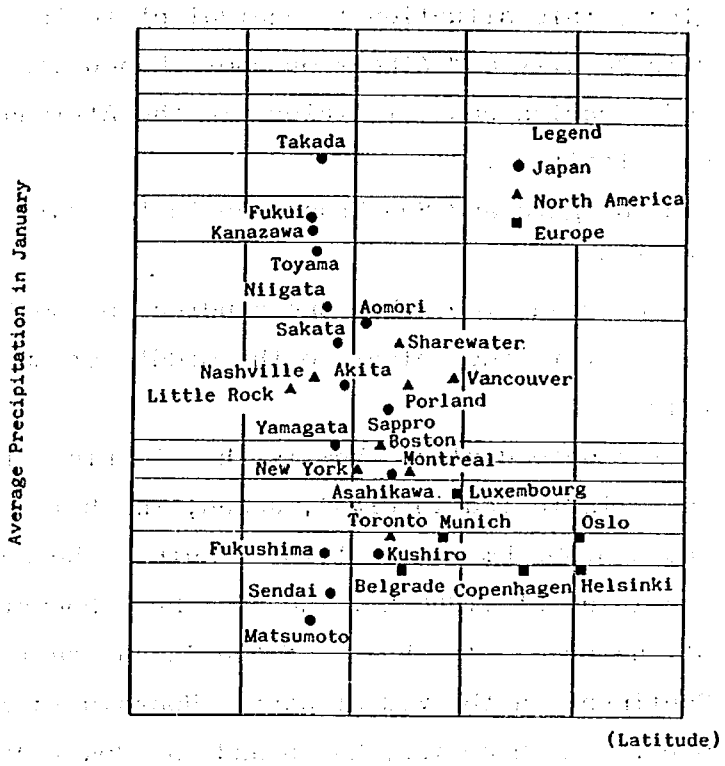


Fig-2 Latitude and Precipitation in Major Cities in the Northern Hemisphere
(From: Climate map of the world, Chronological Table of Science, 1977, ed. by Tokyo Astronomical Observatory)

In Fig-2, the relation between the average precipitation and latitude in main cities of the northern hemisphere where the average temperature in January is below 5°C is indicatad. According to this, precipitation of the Hokuriku District is extremely much.

As a result of development of technics for snow removing and protection against snow, within the area where people were forced to lead an isolated and incovenient life in winter in the past, it is possible for them to have social and econimic activities just as in terms when there is no snow. Therefore people in the area live presupposing there are traffic facilities, and expectation toward snow removing and importance of responsibilities about them have been increasing more and more.

Hereupon what position is the position of the Hokuriku like in terms of the whole world? The three prefectures in the Hokuriku District are situated in the sphere which is in lat. 38°33'N. at the northern extremity and in lat. 36°3'N. at the southern extremity and long 139°54'E. at the eastern tip and 136°14'E. at the western tip. When one turns a terrestrial

globe, this situation is equivalent to the middle of the Mediterranean. Niigata city and Athens or boot-shaped Sicily Island, and Kanazawa city and Algerian capital Algiers in the African Continent are almost on the same latitude.

Also Niigata and San Francisco in the United States are slmost on the same latitude.

In the mean time, when one studies the south along the longitude, this district is equivalent to the middle of Australia through New Guinea. (Fig-3)

In the second place, how is the District situated in Japan?

As is indicated in Fig-4, Japan, which is a strip of land, extends in about 3,000 km in the total length. The area which fronts on the Japan Sea is about 1,900 km, and the three Hokuriku prefectures are in the middle of the area. The bosom of the Japan Sea is 900 km far from the Asian Continent in the widest part. Honshyu is about 250 km in the thickest part. The Hokuriku, Kanto and the Tokai Districts are situated back to back.

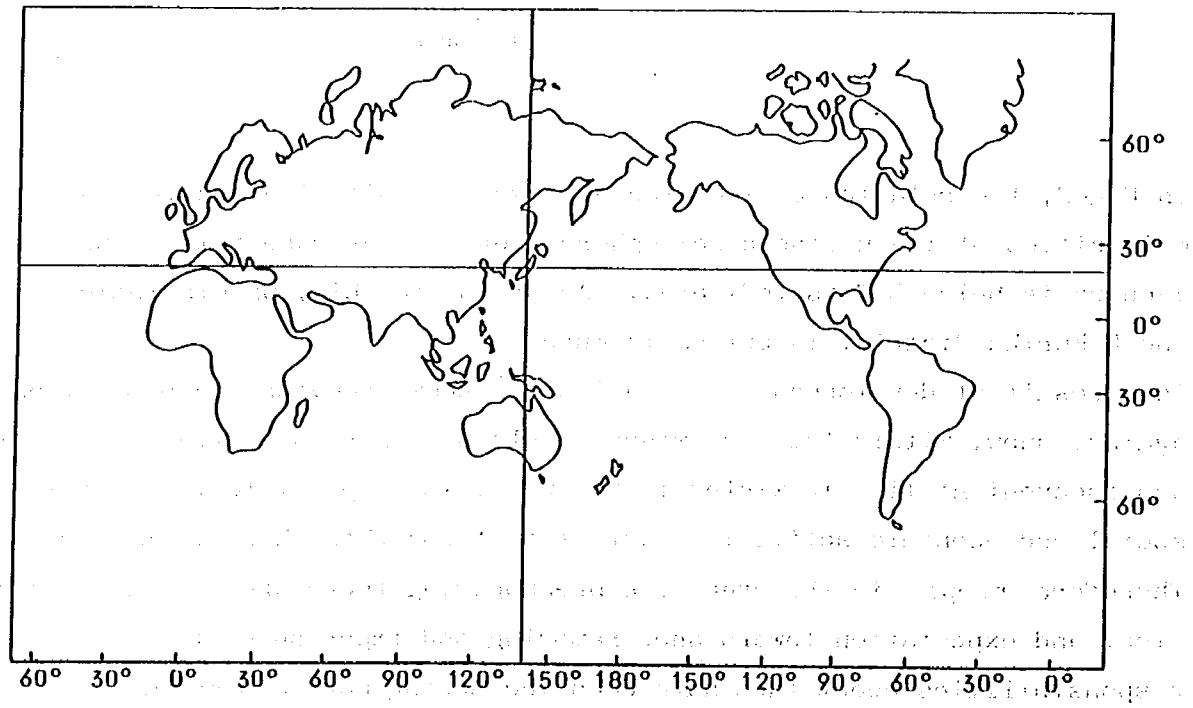


Fig-3 The location of Hokuriku district in world

2) volume of snowfall

The total size of Ishikawa, Toyama, and Niigata prefectures are 21,000 km^2, and the population of the three are 4,700,000. These prefectures are the areas of a heavy fall of snow managed by the law of special counterplanned measures to areas of a heavy fall of snow, and there are no areas where it fails to snow tremendously heavily. The jurisdiction which is directly managed by the nation is 963.5 km of 12 routes. Its actual condition of snowfall is shown in Fig-5 and Fig-6.

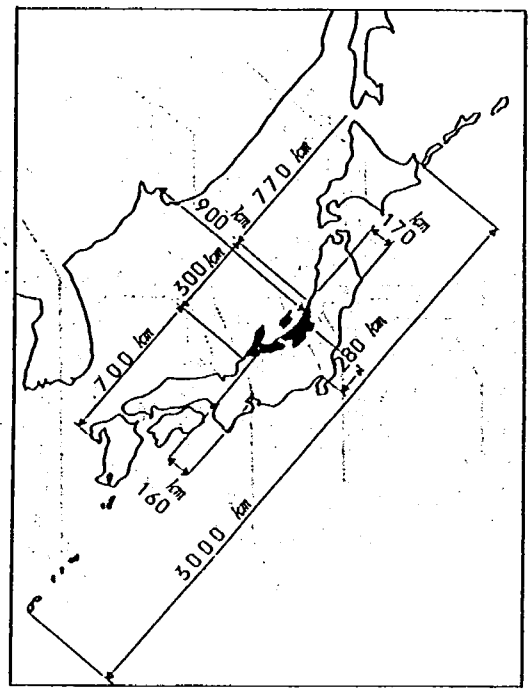


Fig-4 Distance Between Major Cities

The record of the minimum snowfall in total of a day in the past 5 years is 21 cm on Route 159 in Nanao in 1978, and the maximum snowfall in total of a day in the same years is 23 m on Route 18 on the Myoko Heights in 1983 (actually heavy fall of snow in 1984).

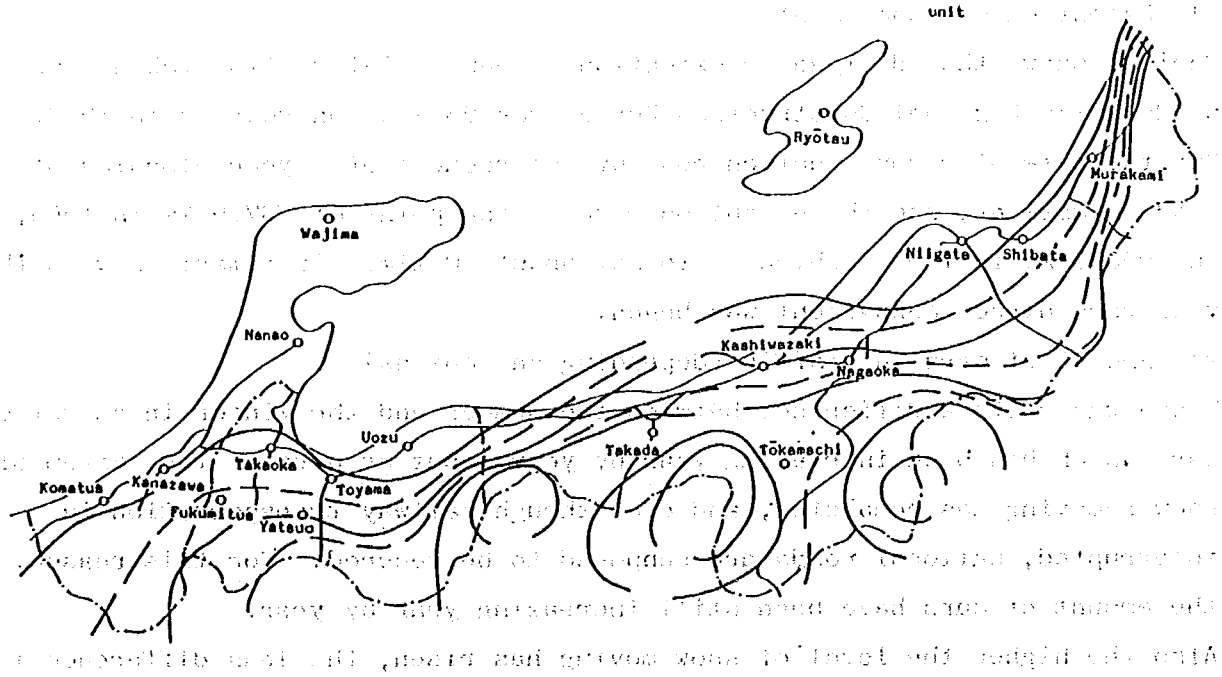
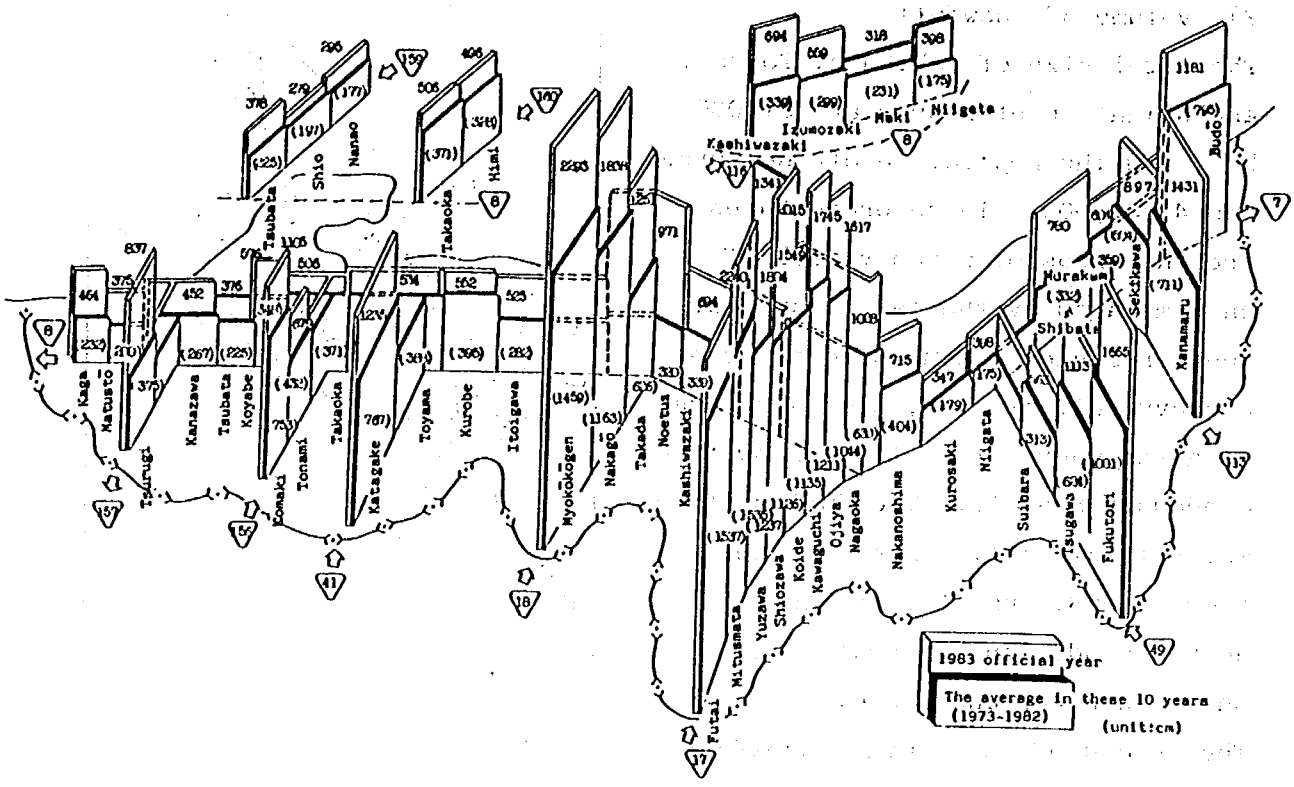


Fig-5 '84-HEAVY SNOWFALL : The distribution of heavy deepest snowfall



F9g-6 The average in these 10 years and the amount in 1983 official year

(From April 1983 to March 1984; we had heavy snowfall in 1984)

2. Conditions of management

1) Extention of management

Table-1 shows that designated extention in managerial section taken charge by Hokuriku Regional Construction Bureau was 93.7 km on Route 8 in 1958. But the rate of reconstruction rose by enforcement of 5 year planning of road adjustment, and the extention rose to the point of 767.9 km in 1968, and 963.5 km in 1983. Which is to say about 10 times in comarison with the year when direct management was begun.

2) Amount of cars in traffic (depending on seasons)

There was quite a difference between the summer and the winter in amount of cars which had been increasing year by year. But at present the system of snow removing has completed, and even though railway transportation is interrupted, national roads are supposed to be secured. For this reason, the amount of cars have been still increasing year by year.

Also the higher the level of snow moving has risen, the less difference in the amount of cars between the summer and the winter.

Table-1 Change of the directly controlled block

Year (April to next March) \ Route (Rt.)	Rt.7	Rt.8	Rt.17	Rt.18	Rt.41	Rt.49	Rt. 113	Rt. 116	Rt. 156	Rt. 157	Rt. 159	Rt. 160	total
3 3	–	93.7	–	–	–	–	–	–	–	–	–	–	93.7
3 4	–	93.7	–	–	–	–	–	–	–	–	–	–	93.7
3 5	–	93.7	–	–	–	–	–	–	–	–	–	–	93.7
3 6	24.3	116.3	–	–	–	–	–	–	–	–	–	–	140.6
3 7	24.3	203.8	25.2	–	–	–	–	–	–	–	–	–	253.3
3 8	48.7	234.7	41.3	22.2	–	–	–	–	–	–	–	–	346.9
3 9	57.2	251.9	40.8	27.6	–	–	–	–	–	–	–	–	377.5
4 0	57.0	301.3	44.8	38.8	15.2	–	–	–	20.4	–	–	11.4	488.9
4 1	108.2	364.5	100.9	39.0	27.3	18.3	–	–	20.4	8.9	54.4	11.4	753.3
4 2	107.9	367.0	100.9	39.0	27.3	30.6	–	–	20.4	8.9	54.4	11.4	767.8
4 3	107.9	367.0	100.9	39.0	27.4	30.6	–	–	20.4	8.9	54.4	11.4	767.9
4 4	107.9	366.4	100.8	39.0	27.3	31.4	–	–	23.2	8.9	54.4	11.4	770.7
4 5	107.8	365.6	102.2	39.0	27.3	63.1	–	–	23.2	8.9	54.4	11.4	802.9
4 6	107.8	365.6	102.2	39.0	27.3	67.0	–	–	26.8	8.9	54.4	11.4	810.4
4 7	107.8	365.6	102.2	39.0	27.3	67.0	–	–	26.8	8.9	54.4	11.4	810.4
4 8	107.8	365.6	102.2	38.3	27.3	67.0	–	75.4	26.8	15.0	63.0	25.6	913.9
4 9	109.7	364.7	102.2	38.3	27.3	65.4	–	77.3	26.8	15.0	63.0	25.6	915.3
5 0	109.7	362.0	102.2	37.2	33.9	66.6	26.8	77.3	28.7	15.0	63.0	25.6	948.8
5 1	109.6	361.8	102.2	37.2	34.2	66.5	25.3	77.3	28.6	15.0	63.8	25.7	947.2
5 2	109.9	361.8	102.2	37.2	34.2	66.5	25.6	77.3	28.6	15.0	63.8	25.7	947.8
5 3	109.9	361.6	102.2	37.2	34.2	66.5	25.6	77.3	28.6	15.1	63.1	25.8	946.6
5 4	109.9	361.3	102.2	37.3	34.2	66.5	25.6	77.3	28.6	15.1	63.1	25.6	946.7
5 5	109.9	361.3	102.2	37.3	34.2	66.2	25.6	77.3	28.6	15.1	63.1	25.6	946.4
5 6	110.7	361.3	102.2	37.3	34.2	66.1	25.9	77.3	28.6	15.1	63.1	25.6	947.4
5 7	111.4	361.3	102.2	37.3	34.2	66.1	25.9	77.3	28.6	15.1	63.1	25.6	948.1
5 8	111.3	361.4	110.1	37.3	34.2	66.1	25.9	77.3	28.6	21.2	63.1	27.0	963.5

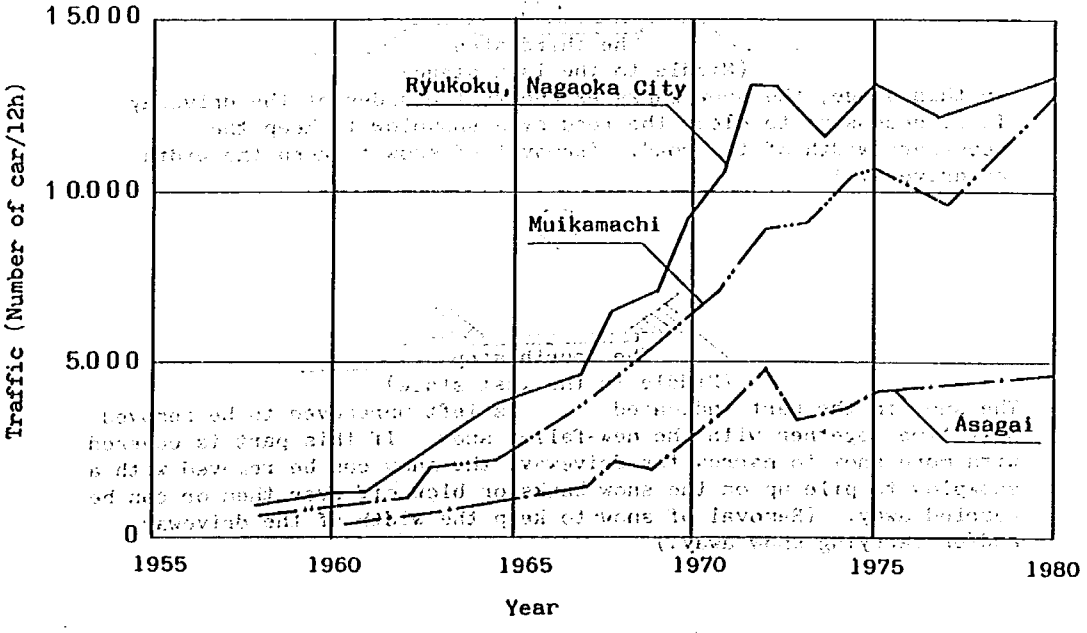
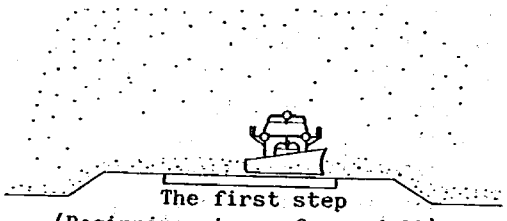


Fig-7 Transition of traffic on Route 17 (Example)

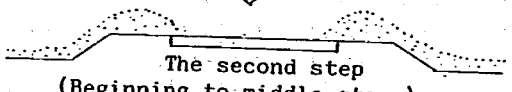
3. Snow of removing

1) Method of removing

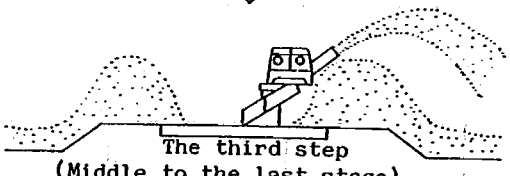
Snow removing on roads is mainly done by machinery, and its method is divided into two ways. In the first removing they swiftly remove soft snow on roads to the gutters on both sides of the road. And in the second removing they remove the accumulated snow created in the process of the first removing. Depending on the difference of operations, snow removing is divided into several ways, such as removing of new snow, removing for



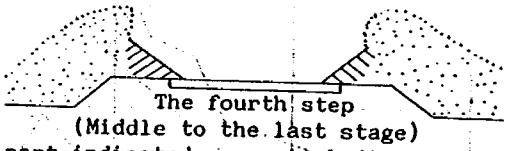
The first step
(Beginning stage of snowfall)
Remove the snow on the road to the roadsides with a snow removing truck equipped with rotary plow or with high-speed car equipped with snow removing grader. (Removal of new-fallen snow.)



The second step
(Beginning to middle stage)
Where the snow banks are less than 70-100 cm high, new-fallen snow can be blown off over the snow banks with a high-speed car. If the snow banks are higher, they should be lowered.



The third step
(Middle to the last stage)
In this stage, the snow banks extend to the sides of the driveway. It is necessary to clear the road by a snowplow to keep the necessary width of the road. (Removal of snow to keep the width of driveway.)



The fourth step
(Middle to the last stage)
The snow in the part indicated is left unremoved to be removed next time together with the new-fallen snow. If this part is covered with more snow to narrow the driveway, the snow can be removed with a snowplow to pile up on the snow banks or blow off over them or can be carried away. (Removal of snow to keep the width of the driveway and/or carrying snow away.)

Fig-8 Procedure of Snow Removal

widening the roads, removing by conveying, disposing of accumulated snow, and what not. In preliminary stages of removing, removing of new snow comes first and this is the very time that high speed is required.

Therefore frequent removing of new snow is required in order to prevent snow from hardening and at the same time road surface must be adjusted. Usually snow removing operation is done in the way indicated in Fig-8, and as frequency of new snow removing increases, the mound of snow put on both side of the road grows higher and as this result the width of the roadway grows narrow.

Therefore the third and fourth stages must be done, and as a result, the height of the accumulated mound sometimes reaches over 5 m.

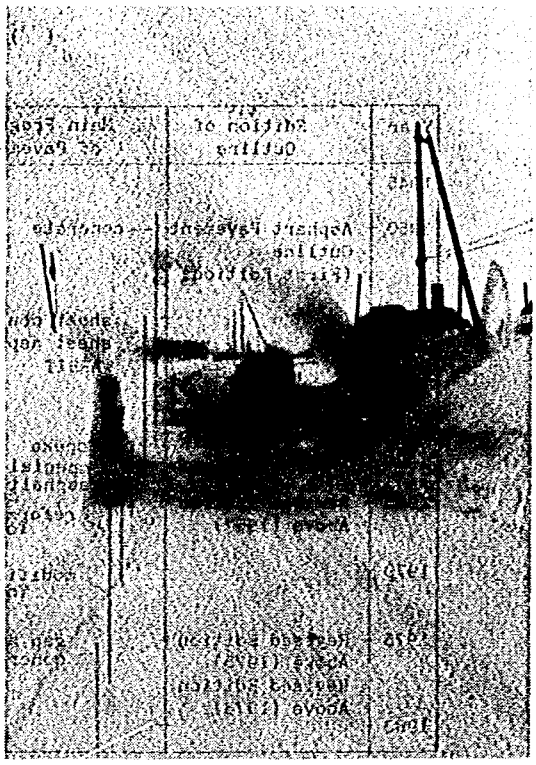


Photo-1 Snow removing for windening the road

4. The actual condition of paving

1) Process of pavement work by Hokuriku Reagional Construction Bureau

Hokuriku Regional Construction Bureau was established in 1958. In those days the number of times of road repairing work was not so many. also the production system of principal materials "macadam" was quite poor and volume was quite scanty, too. Only macadamics engineering or engineering by which seal coat on the basis of coarse-grained asphalted concrete had been adopted.

Table-2 indicates outline of process about asphaltic paving works in the Hokuriku District. Asphaltic pavement in cold districts with heavy snow is quite different from that of the Kanto District, that is, not only influenced by the weather condition, but also severely defaced by chain fixed tires or spike tires. As one measure to the defacement in about 1965, to make the biggest diameter of frame in aggregate or to use sheet asphalt from which coarse frame was reduced to some extent, or mixture produced by

Table-2 Outline-Transition of Pavement in Hokuriku District

( ) indicates year

Year	Edition of Outline	Main Progress of Pavement	Remarks
1945			
1950	Asphart Pavement Outline (First Edition)	concrete	
			First five-year plan (1954)
1955		sheet coat sheet asphalt WARBIT	Hokuriku Regional Construction of Bureau of Ministry of Construction established. 2nd five-year plan (1957)
1960	Revised Edition Above (1961)	Topeka special asphalt concrete	3rd five-year plan (1961) 4th " (1964)
1965	Revised Edition Above (1967)	reformed Topeka	5th " (1967) Begin gap asphalt concrete study
1970		modified Topeka	6th five-year plan (1970) Gap asphalt concrete tested 7th five-year plan (1973)
1975	Revised Edition Above (1975) Revised Edition Above (1978)	gap asphalt concrete	Decided to adapt gap asphalt concrete 8th five-year plan (1978)
1980			

Topeka engineering had been adopted. In those day, when frame of which grains diameter is between 20 and 25 mm was mixed in large quantity, the road surface at the time of paving used to look black, but after one winter, having been defaced by chain fixed tires of cars asphalt flied about. As the result, the whole road surface used to look white. Mixture like sheet asphalt which mainly consists of granular frame, on the other hand, was supposed to have quite strong power of resistance against defacement because it did not change its blackness very much after being put into use even after it was trodden by chain. That is the reason it had been put into use. But on close examination of the surface of used granular asphaltic concrete for several years, it developed that the parts of defacement showed a phenomenon that coarse frame jutted out of asphaltic mortar and for this the parts had increased resistance against slip. Whereupon regarding this fact as important, the authorities concern set about examining and work in 1968 to the effect that they produce mixture that has both strong resistance against slip and defacement-proof power. The method was to make the rate of mixture of frame mighty to the point that asphalt is not defaced and does not fly about, also to make the surface of paved road rugged like open grading asphaltic concrete, and mix the material with

adequate volume of asphaltic ingredient.
Afterwards through the past 8 years, it has continued to be developed and finally it was adopted as staple paving material of Hokuriku Regional Construction Bureau and has been used so far.

2) Characteristic of gap asphaltic concrete

Gap asphaltic concrete is a kind of asphaltic mixture which has discontinuous typed grading. Its diameter of frame, whose grading is from 2.5 m to 0.6 mm, is made extremely little so that it can be put within 5%, and No. 6 macadam and No. 7 macadam are plugged into asphaltic mortar in order to increase the rate of gaps of the frame. When asphaltic mortar and frame whose diameter is big are mixed adequately, resistance against slip and resistance against flowing increase, and also the rate of defacement-proof rises.

Therefore it is one of the most appropriate materials as mixture for asphaltic surface layer in cold districts with heavy snow. (which see in Table-3 and Fig-9). The following is its characteristic.

Table-3 Standard Grading

size of shifter mesh (mm)	amount through the mesh (%)
20	100
13	95 ~ 100
5	55 ~ 75
2.5	45 ~ 65
0.6	40 ~ 60
0.3	18 ~ 40
0.15	10 ~ 20
0.074	7 ~ 13

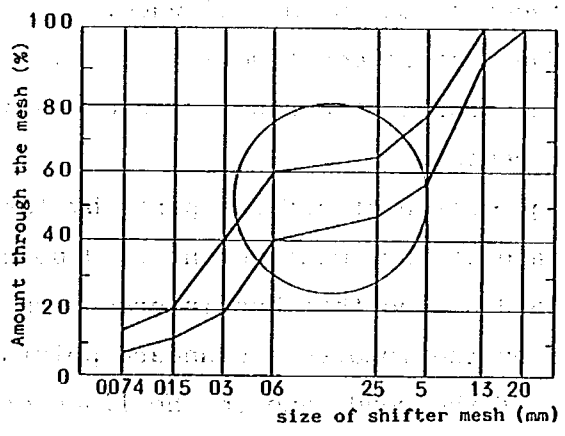


Fig-9 Standard Grading Carve

a) Defacement-proof against chain or spike tires
(hard to create parts that are deface by the prints of wheels).

Fig-10 shows the relation between 2.5 mm quantity of passage and that of defacement. Both S type and G type clearly shows that quantity of defacement increases as 2.5 quantity of passage increases. It developed that mixture which contains much granular frame like Topeka or sheet asphalt has much quantity, and the relation between both types is almost straight-like.

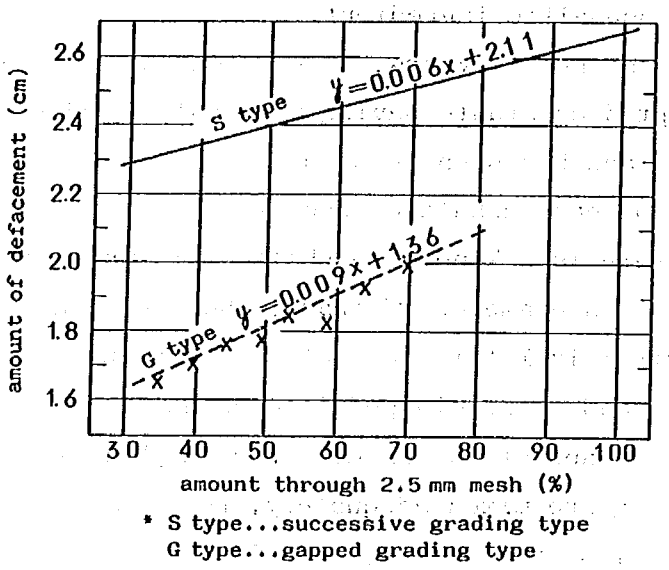


Fig-10 Amount Through 2.5 mm Mesh and of Defacement (investigated inside)

b) The paved surface is completed rather rugged and resistance against slip is high (hard to slip).

c) Destruction by waving is little (there is little parts that are defaced by the prints of wheels).

d) The cost of construction is low (10% lower than Topeka).

3) The result of pursuit of gap asphaltic concrete inquisition into defacement.

In inquisition of defaced parts by the prints of wheels caused by defacement and waving, measurement of 9 points in one kind both about a single road and a road that has intersection was done with cross profile meter based on the immovable standard points (Fig-11) which were set up on both sides of the roads and on the lines of division. Comparison of change of defacement by the prints of wheels in several years according to the average of the 9 sections is indicated in Fig-12. The discrimination of defacement and waving was done depending on terms of inquisition. The quantity of change from March to November is quantity of waving and the one from November to March is that of defacement. Quantity of defacement was computed on the basis of record of crossing measured with cross profile meter in the term before winter (November), then by calculating the difference between the section which was measured before winter (November) and the one after winter (March), and then by dividing the sectional size with the width of the road. The result is indicated in Fig-13.

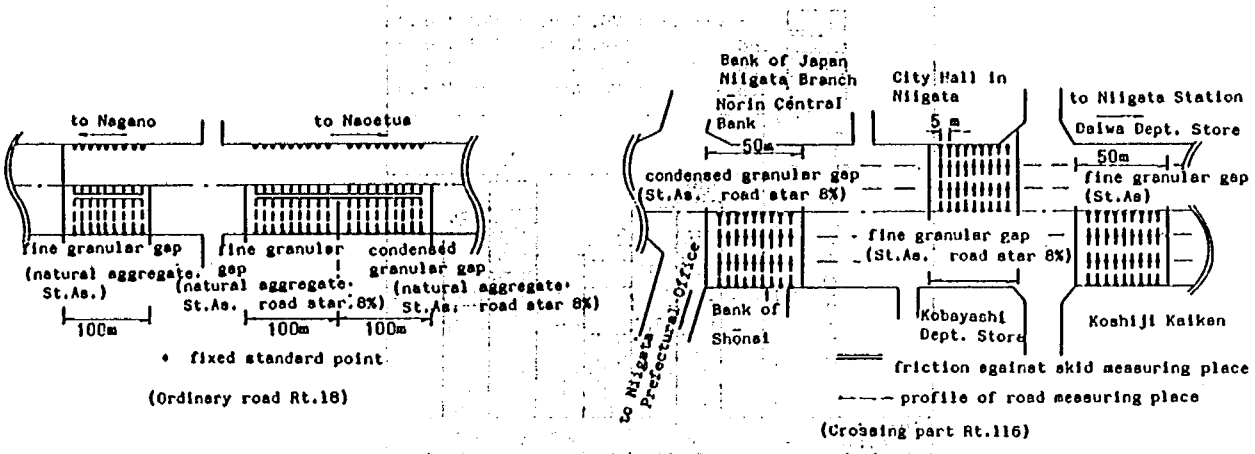


Fig-11 Places of Experimental Pavement

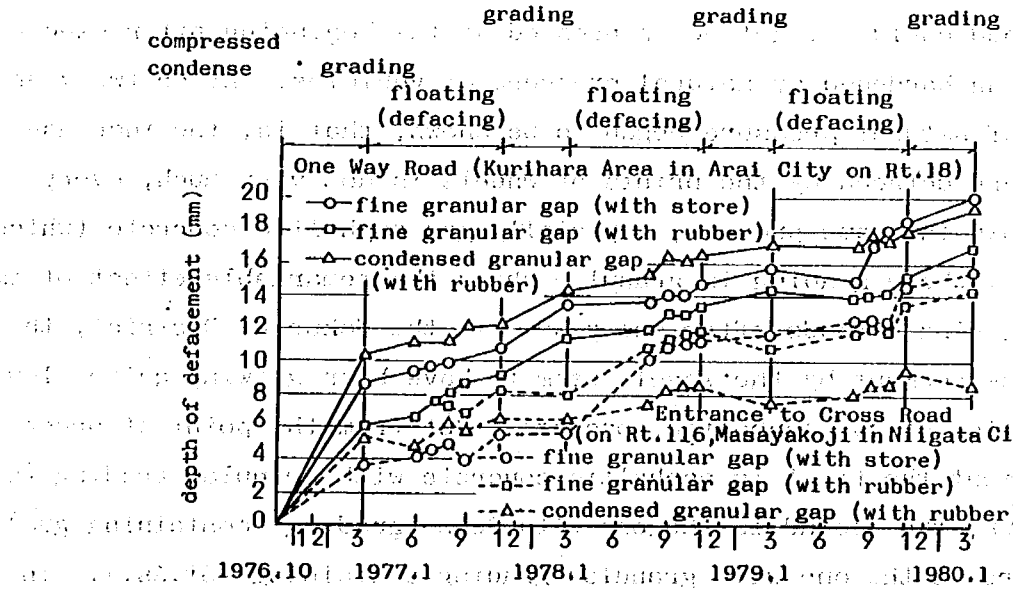


Fig-12 Duration of Defacement by Wheels

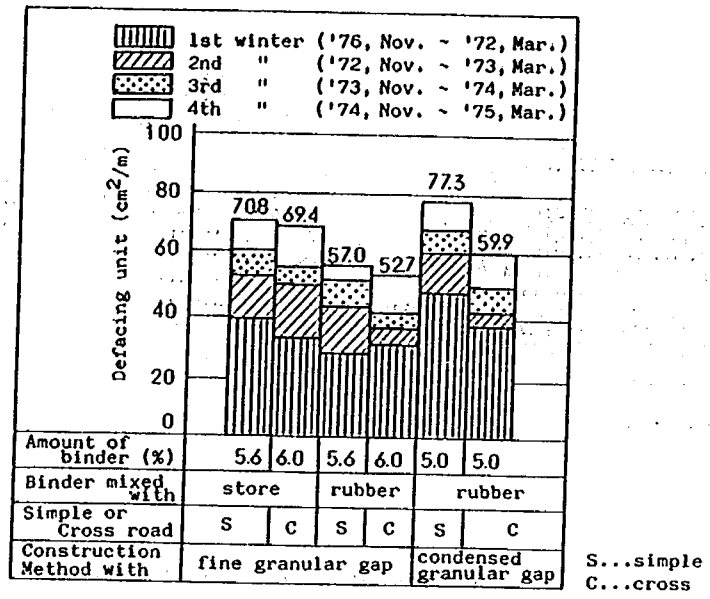


Fig-13 Cumulative Amount of Defacing

i) Parts that are defaced by the prints of wheels caused by defacement and waving.

Process of parts that are defaced is liable to be remarkable from the term of execution to the term after one year, and the rate of process after two years is apt to be slow. This tendency is considered to be the result as which the road could not get being pressed at the beginning after execution and being hardened by natural pressure of vehicles. After two years the effect of natural pressure began to be shown, that is, the increase of parts that are defaced by the prints of wheels is not very much, every specimen shows the same process, especially gap asphaltic concrete (which contains gum) whose grading is granular shows the remarkable effect of improvement on single roads after one year. In the Hokuriku District, the process of defacement by the wheels seem to have been growing quite slow after execution of gap asphaltic concrete. But from the point of order in this experiment the first is asphaltic concrete with granular grading (containing gum), the second is the one with dense grading (containing gum), and the third is the one with granular grading containing (St.As.). In comparison with defaced parts by the wheels from waving, defaced parts by the wheels from defacement is 2.1 times in case of gap asphaltic concrete with granular frame (St.As.), 1.8 times, in case of the one with dense grading (containing gum), about 3 times. That is, defaced parts by the wheels are quite larger (Table-4). On the other hand, on the road with intersection, gap asphaltic concrete with dense grading (containing gum)

Table-4 Defacing and Amount of Defacing

kind of mixture	kind of binder	ordinary road (Rt.18, Kurihara in Arai city) defacing (mm) by defacement	by fluidity	total	ranking	amount of grading (cm/m) amount of grading	ranking	entrance to cross road (Rt.116, Masayakoji in Niigata city) defacing (mm) by defacing	by fluidity	total	ranking	amount of grading (cm/m) amount of grading	ranking
fine granular gap	St.As.	13.7	6.4	20.1	3	70.8	2	4.4	11.1	15.5	3	69.4	3
fine granular gap	with rubber	10.9	6.2	17.1	1	57.0	1	6.7	8.4	15.1	2	52.7	1
condensed granular gap	with rubber	14.7	5.0	19.7	2	77.3	3	5.4	4.3	9.7	1	59.9	2

comes first, the one with granular frame (containing gum) the second, the one with granular frame (containing St.As.), the third. That is to say, defaced parts by wheels from waving is on a little larger scale than single roads.

This seems to be the result of increase of roadways and tires' pressure of big cars caused by start and stop, which is characteristic of asphaltic paving.

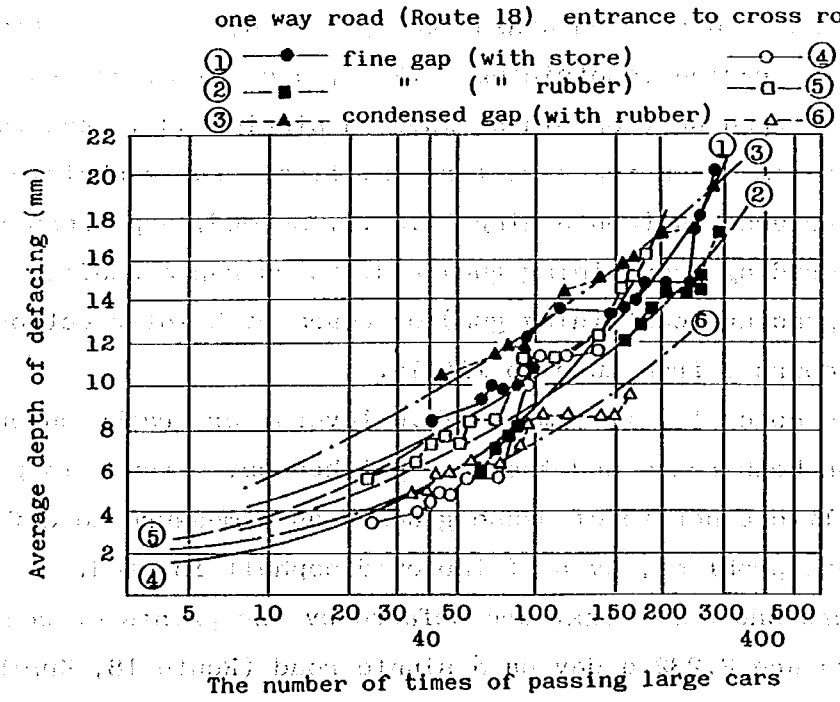


Fig-14 The Number of Times of Passing Large Cars and Cumulated defacing

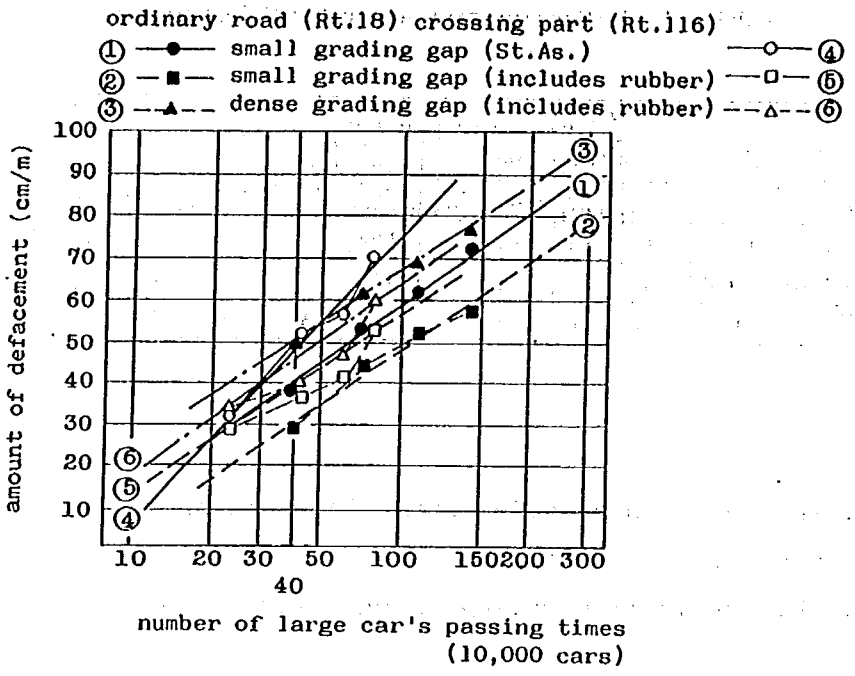


Fig-15 The Number of Large Car's Passing Time and The Amount of defacement

ii) Quantity of defacement

The total quantity of defacement in a 4 winter term is, first, gap asphaltic concrete of dense grading (containing gum), second, gap asphaltic concrete of granular grading (containing St.As.), and third, gap asphaltic concrete of granular grading (containing gum) on singleroads. And the total quantity of defacement in the same term is, first, gap asphaltic concrete of granular grading (containing St.As.), second, gap asphaltic concrete of dense grading (containing gum) and, third gap asphaltic concrete of granular grading (containing gum) on roads with intersection. This order is, of course, from much to little.

As to gap asphaltic concrete, it comes off in layers, or cracks on quite large scale, and on both roads and they are patched up. From this point of view, gap asphaltic concrete of dense grading as a measure to defacement is said to have problems, even if improved asphalt is used.

iii) Numbers of cars and parts that are defaced by the prints of wheels

Numbers of big cars are 2,232 a day on a single road (Route 18, Kurihara, Arai city), and 1,287 a day on a road that has intersection (Route 116, Masayakoji Niigata city). By March in 1980, the number of times of big cars' passage at a rough estimate is about 2,860,000 on the single road, and 1,650,000 on the road that has intersection. The relation between the number of times of big cars' passage, and accumulative parts that were defaced by the prints of wheels and accumulative volume of defacement is

indicated in Fig-14 and 15.

iv) The relation between the number of times of big cars' passage and defaced parts by wheels.

According to Fig-14, the number of times of big cars' passage was approximately 2,860,000 on the single roads, and gap asphaltic concrete (containing St.As.) of granular grading grew 20.1 mm, the one (containing gum) of dense grading, 17.1 mm. The one (containing gum) of dense grading grew 19.7 mm. On the roads that have intersection, approximately 1,650,000 big cars had passed, and gap asphaltic concrete (containing St.As.) of granular grading grew 15.5 mm, the one (containing gum) of granular grading, 15.1 mm, that is, little difference. But the one (containing gum) of dense grading grew 9.7 mm.

About gap asphaltic concrete of granular grading on single roads, the number of times of big cars' passage was about 1,650,000 at a rough estimate by the time two years had passed and defaced parts grew 15 mm. This is equivalent to the average defaced parts of 15 mm and movable stability whose number of times is 1,000 with 1,500,000 cars that are shown on the chart of data from the Public Works Lavoratory (The relation between quantity of permissible defaced parts, movable stability DS, and the number of times of big cars' passage). But in terms of the fact that about 60% of accumulated defaced parts are caused from defacement by the prints of wheels, according to this inquisition, there actually seems to be pretty registance power against flowing.

Table-5

Dynamic Stability and Frequency of Large-sized Vehicles

(Unit: 10,000 vehicles) Data from Engineering Research Institute

average amount of defacement (mm) / dynamic stability	single driveway		crossing section	
	10	15	10	15
1000	80	150	50	90
1500	150	300	90	170
2000	250	700	150	400

Movable stability of a cut sample from the actual spot was below 900 times/mm in 1980. This seems to be the result of the changes of frame and binder in years that are caused from flowing and defacement. As to roads with

intersection, 900,000 cars and 10.0 mm that is, quite satisfactory figures. But the spots that were inquired have three lines for cars in one way, that is, as the condition is different, so it is impossible to compare.

v) The relation between the number of times of big cars' passage and the quantity of defacement.

On single roads the number of times of big cars' passage at a rough estimate (the quantity through 4 winters) was 1,420,000 cars and the accumulated quantity of defacement (cm^2/m) through 4 winters was 57.0 about gap asphaltic concrete (containing gum) of granular grading, 70.8 about the one (containing St.As.) of granular grading, and 77.3 about the one (containing gum) of dense grading. And on roads with intersection, the number is 820,000 cars, 52.7 about gap asphaltic concrete (containg gum) of granular grading, 69.4 about the one (containing St.As.) of the same grading, 59.9 about the one (containing gum) of dense grading, that is, there is tendency of straight on both kinds of roads. In terms of this result, gap asphaltic concrete (containing gum) of granular grading can be said to be mixture which has resistance power against defacement. The result of inquisition into parts that are defaced by the prints of wheels and the quantity of defacement is indicated in Table-4. From the result of inquisition above, on general single roads, the quantity of defaced parts caused from defacement by the prints of wheels is about 2 times as that caused from flowing. From these reasons, it seems to be desirable to attach importance to resistance power against defacement and to produce mixture that is capable of reducing defaced parts by flowing in the Hokuriku District.

5. Damage of the pavement

Surface condition of the road is regularly examined volumetrically by the road surface survay service as well as daily patrol by eyesight. "Cracking" and "Defacement" are the factors generally used for the evaluation of the road surface condition. Cracking is the important item for the road surface survay which directly influence the durability of the pavement. Defacement of the road used to depend on the quality of the surface materials as well as its structure. When it rains, it causes the formation of puddles and, as the result, splashing of water to the sidewalk and adjacent car. Furthermore, it gives a bad influence on handling because it deteriorates the friction of the tires of driving car against the road. Therefore, defacement is an important factor to decide the timing of repair.

For the proper practice of road repair, ordinaly construction methods such

as overlay, drilling and filling of hole for pothole have to be done pricisely and effectively. In view of traffic safety and environment regulation, reasonable standard of road maintenance has to be sep up. A quick construction technique which allow the traffic have to be made also. Thus, road maintenance requires most of the fundamental techniques of road construction and repair. Besides, proper method have to be chosen within the limited budget.

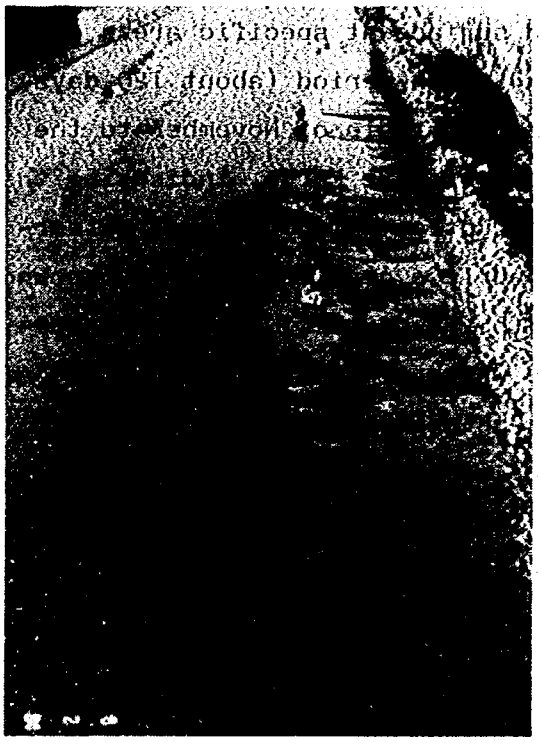


Photo-2 Road surface damage

1) External factors which cause the surface damage

i) Snowfall and road surface condition

What directly influence the spiked tire-using rate are the timing and amount of snowfall, and also the condition of the compressed snow on the road.

Figure-16 shows the snowfall diary at certain areas of Hokuriku district. Earliest snowfall was recorded on November 6, at Futai along Rt. 17, Yuzawa-machi, Minami Uonuma-gun, Niigata prefecture followed by Rt. 18 at Myōkō district, Naka Kubiki-gun, Niigata prefecture where it snowed on November 8. The first snowfall was one or two weeks earlier than it used to be there. Both districts are famous for heavy snowfall and it snows almost every day of the winter (Days of snowfall, Yuzawa : 69 days, Myōkō : 63 days).

In areas along Rt. 8 which runs through Honshu (main-land of Japan) the amount of snow is smaller than those at the two areas described above. For example, in Toyama city, they had the first snowfall on December 14, and as much as 26 days of total snowfall as it used to be.

During the winter, the condition of road surface changes repeatedly; being covered with compressed snow, sharbet-like snow or uncovered. Among those conditions, uncovered road surface is most seriously defaced by the spiked

tires or tire chains.

Figure-17 shows the condition of road surface at specific areas during 4 month period (about 120 days) from the middle of November to the middle of Next March, indicating the construction ratio of three different surface conditions. It is around 10% (5 ~ 25 days) of total road surface which really require the use of anti-skid equipment such as spiked tires and so on in average and, 27 ~ 35% (35 ~ 50 days) at most.

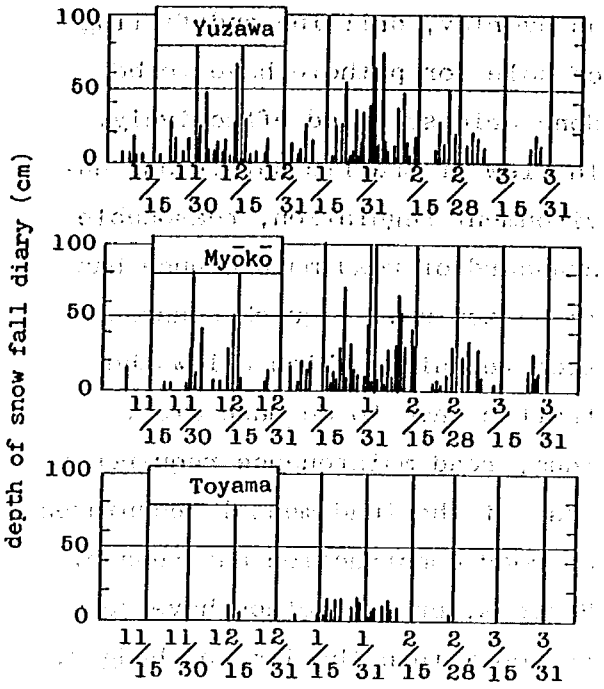


Fig-16 Snowfall diary

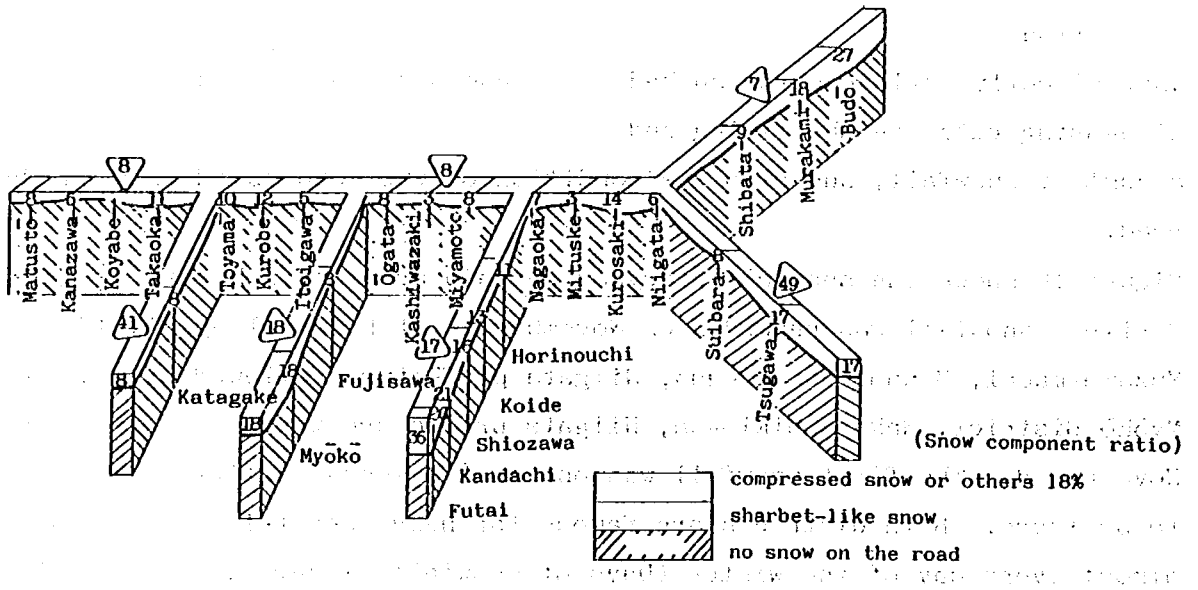
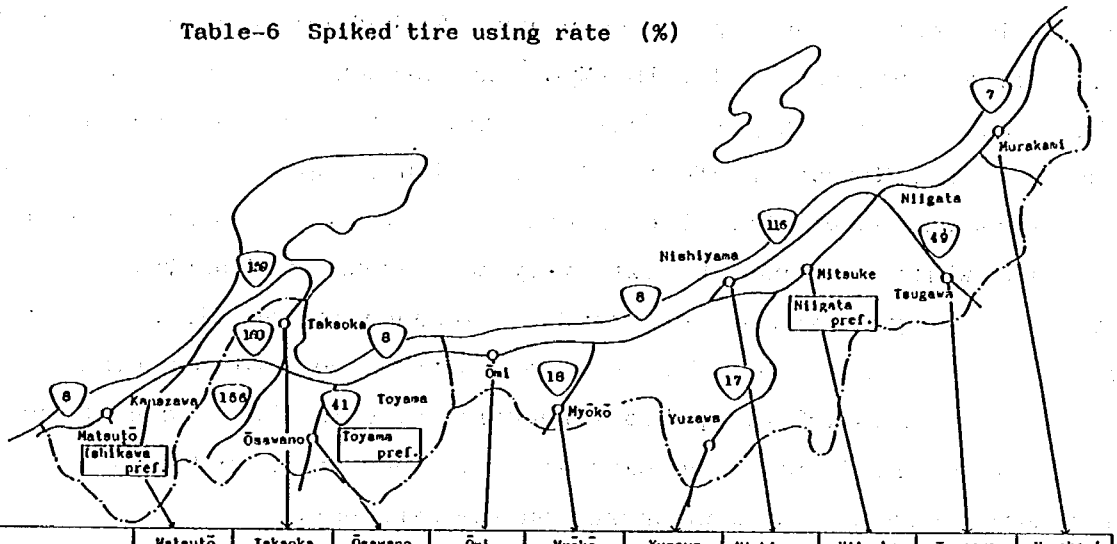


Fig-17 Road Surface Condition (four months: from middle of November to middle of March)

ii) Spiked tire-using rate

Spiked tire-using rate in Hokuriku district during the winter of 1981 to 1982 (about 7 month) was investigated and summarized in Table-6. Seasonal change of spiked tire-using rate at each area is shown in Fig-18. These table and figure indicate that it is around late November when drivers exchange summer tires to spiked tires and that the spiked tire-using

Table-6 Spiked tire using rate (%)



Number of times	Day of investigation	Hatsutō			Takaoka			Ōsawano			Ōmi			Myōkō			Yuzawa			Nishiyama			Mitsuke			Tsugawa			Murakami		
		S	L	T	S	L	T	S	L	T	S	L	T	S	L	T	S	L	T	S	L	T	S	L	T	S	L	T	S	L	T
1	56 10 28	1	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	11 25	1	2	1	0	0	0	0	0	0	0	3	1	10	2	8	8	0	4	2	0	1	5	1	4	4	0	5	0	1	0
3	12 23	38	17	34	32	13	27	45	16	40	46	17	33	52	15	41	46	6	30	31	9	25	39	10	30	63	10	39	58	16	47
4	57 1 20	54	24	49	39	16	36	60	21	54	45	32	39	75	30	60	65	8	48	56	22	42	45	16	36	67	27	59	59	28	52
5	2 17	40	20	36	37	12	31	57	2	30	40	34	38	74	36	61	65	5	42	47	7	33	42	13	35	71	33	58	45	16	37
6	3 17	7	11	8	6	6	6	4	3	3	4	1	3	34	28	33	12	6	10	5	0	3	8	0	6	10	5	9	16	14	15
7	4 14	2	5	2	1	2	1	1	0	1	1	2	1	5	2	4	0	0	0	0	0	0	1	7	3	3	5	4	5	13	6

Small car = S.
Large car = L.
Total = T.

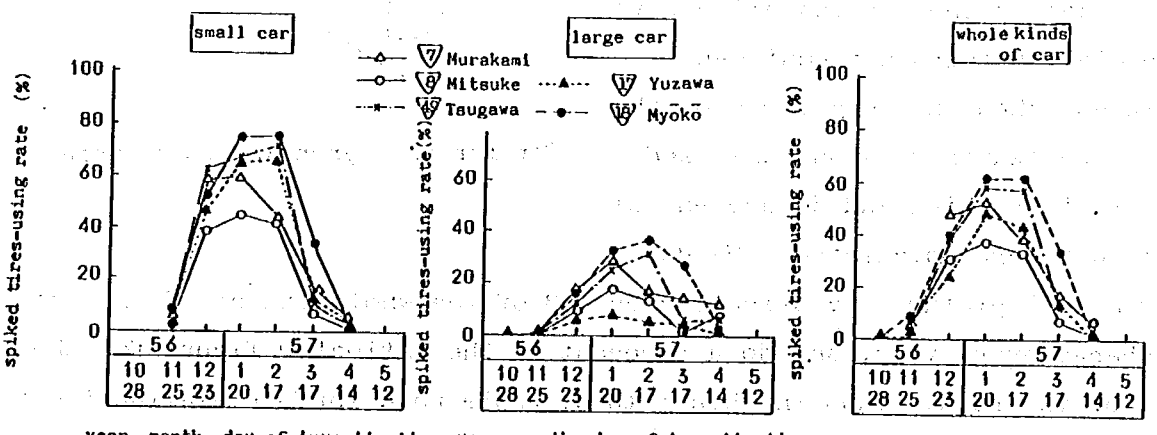


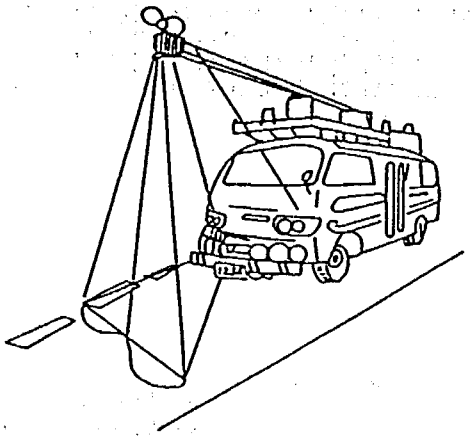
Fig-18 Seasonal change of spiked tire-using rate of each size of car at each area (over 45 snowfall days)

rate is highest in January and February. Spiked tire-using rate of the small car was highest at Rt. 18 (Myōkō kogen-machi, Niigata Prefecture) by 75%, 71% at Rt. 49 (Tsugawa machi, Niigata Prefecture) and 65% at Rt. 17 (Yuzawa machi, Niigata Prefecture). The highest spiked tire-using rate of large cars was about 30%. The spiked tire-using rate is generally high on the roads which traverse the extremely snow areas of Honshyu.
After the snowy season, drivers tend to exchange spiked tires to summer tires sometime in March. However, some of the areas of Hokuriku district showed as much as 13% of spiked tire-using rate in the middle of April, when spiked tire is not necessary at all. Such an extended use of spiked tire must cause pretty much damage on the pavement.

2) The evaluation of road damage at each area by MCI

i) Methods for the examination and evaluation of road surface condition

Two categories of road damage; "cracking" and "defacement" were examined. Pictures of the road surface, mainly of one down lane, were taken by the car installed with a special photographic equipment. Pictures taken in every 100 m of the road were examined as one unit and amount of cracking and defacement at each unit were calculated. We do this actual picture investigation on 1/3 of the whole extent of the road in charge every year, and do a forecast of the estimated figure for the rest of the road. Thus in this method, whole road surface examination is done in three years as one cycle. Figure 19 shows the diagrammatic illustration of a car installed with special photographic equipments.



Car for photographing road surface

For the proper evaluation of road damage, by which the priority order for repairing is determined, it is necessary to refer to some index numbers representing the surface condition, objectively and volumetrically as well. Variety of parameters of wide range of applicability have been proposed for years. Among them, MCI (the maintenance controll index; the 35th Technical Research Meeting of the Ministry of Construction) is employed by the

Ministry of Construction at present. In this MCI method, newly paved surface is expressed by 10 points. The score comes down as the surface grows worse. Fig-20 shows MCI and corresponding standard repair methods.

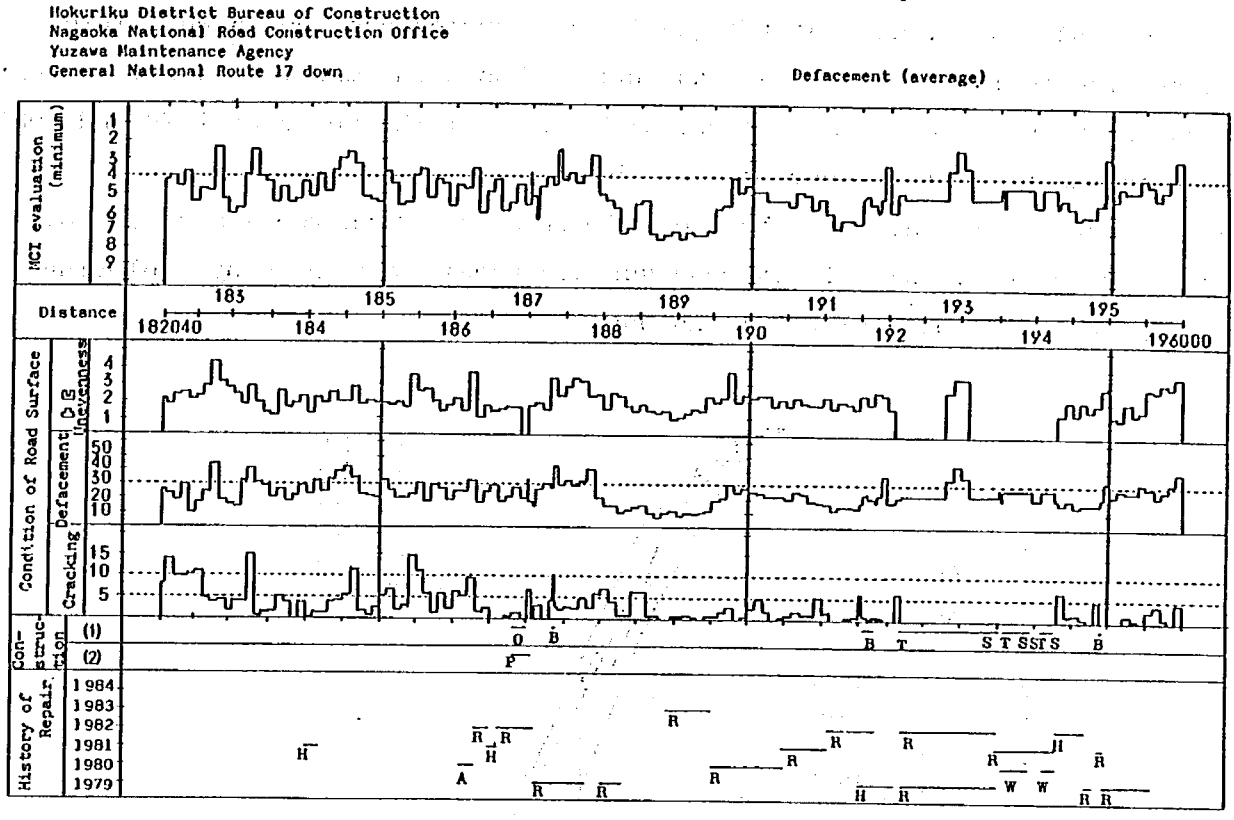


Fig-19 Range of applicability (1984)

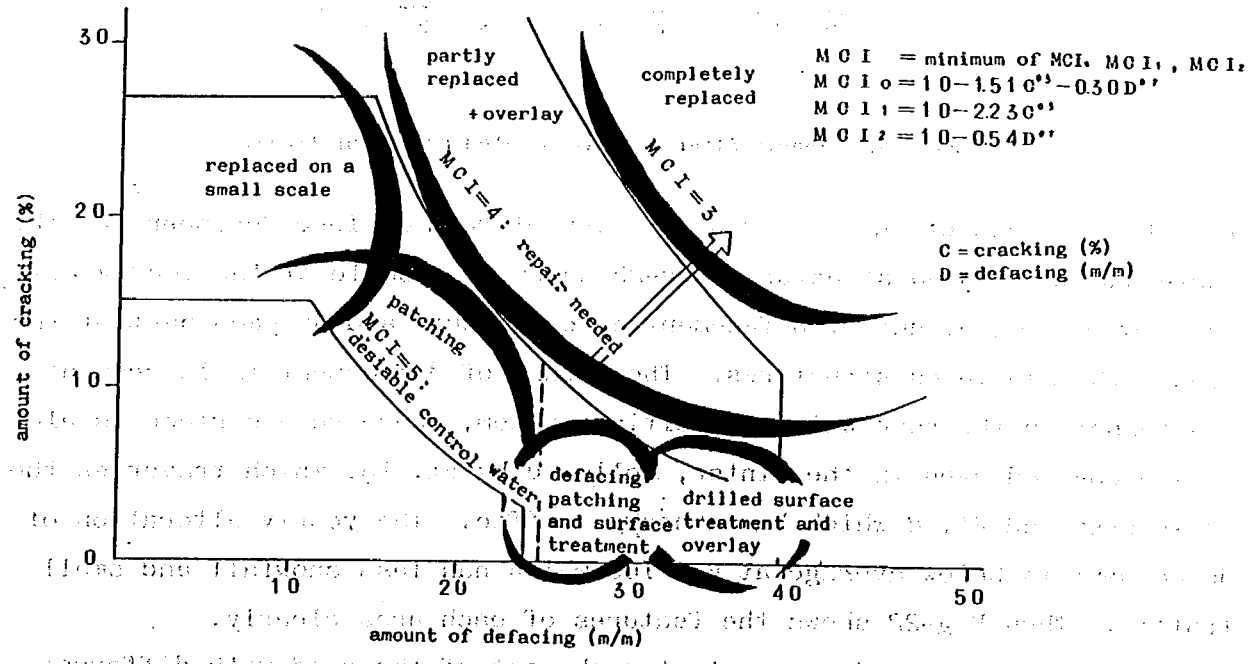


Fig-20 MCI and Standard Repair Methods

ii) Present condition of road surface and its yearly alteration

Fig-21 shows the distribution of MCI in each year. In 1975, the extent of road surface which got MCI score lower than 5 was 21%, but it became up to 41% in 1984 indicating that the quality of survice has come down. This score also indicates that the road surface has been improved to some extent when the present score is compared with those of the years of 1979 and 1981. The reasons for this improvement can be:

a) Proper repair was done, by the effective use of photographic data of the road surface. b) Timely road surface repair was tried by employing the "patching on defacement" technique.

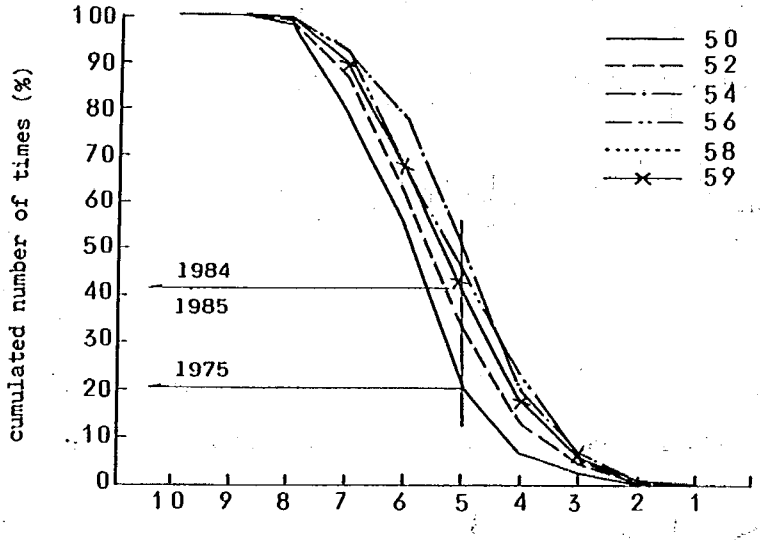


Fig-21 MCI Yearly Alteration (cumulated number of times)

iii) Differences of the yearly alteration of road surface dependent on the route and associated structures Fig-22 shows an example of the yearly alteration of the amount of defacement at each route and at each part of the road with associated structures. The amount of defacement is largest at the tunnel or the road with snowmelting system, where road surface is always uncovered even in the winter, followed by Rt. 18, which traverses the snowy area and Rt. 8 which has a heavy traffic. The yearly alteration of defacement is below average at Rt. 160 which has less snowfall and small traffic. Thus Fig-22 shows the features of each area clearly.

The average amount of defacement at each part of the road with different associating structures and the age of the road are shown in Fig-23.

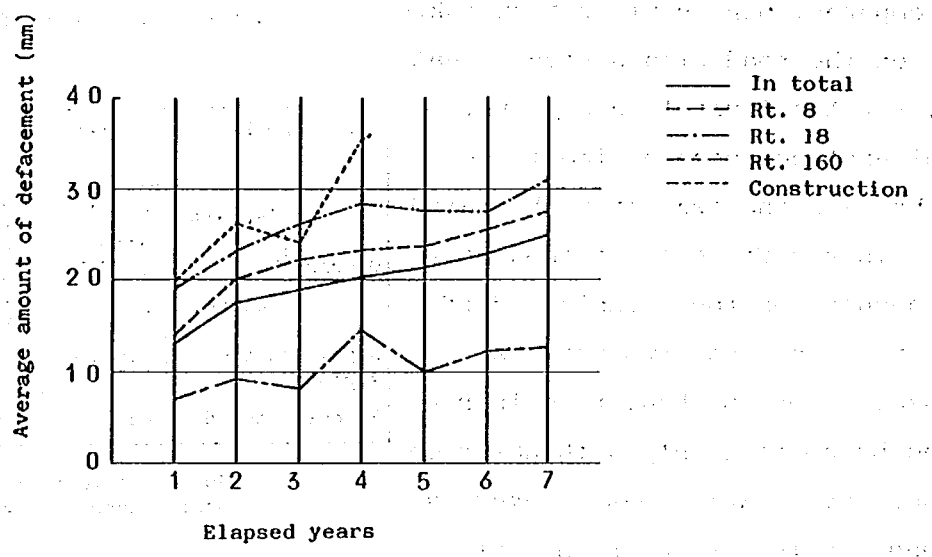


Fig-22 Relation between Amount of Defacement and Years
(By routes · By constructions)

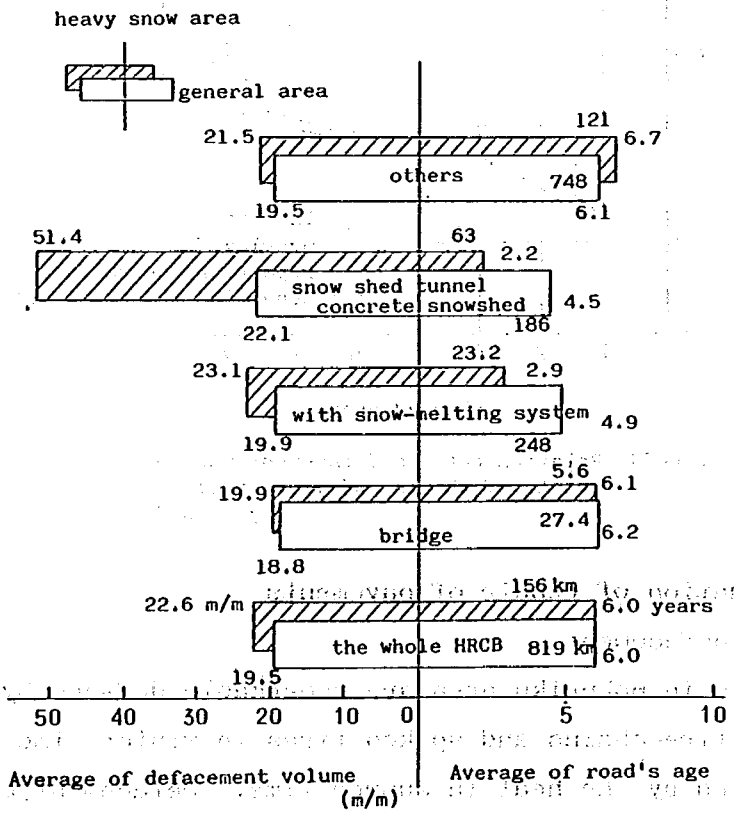


Fig-23 Volume of defacement in each spots

iv) Features of each repair method

When the yearly alteration of road surface is compared in relation to the repair method, the amount was biggest when "the patching of defacement", which is regarded as a temporal repairing method was applied. When the effect of surface-treatment and overlay, on the durability of road surface

are compared, bigger alteration takes place on the road with surface-treatment, for the first 1 to 2 years. But alterations become almost the same between the two after the third year. This difference is thought to be dependent on the technical differences between those two methods. Because, in one method, road is repaired by coating only with AS- rich surface layer, while basal layer is put underneath the AS- rich surface layer in the other method.

Table-7 Thickness of Repair

Repairing Parts		Layer	General Area	Area with much snow
Overlay	Asphalt	Surface	3 cm	4 cm
		Basic	3 cm	3 cm
	Concrete	Surface	3 cm	4 cm
		Basic	6 cm	6 cm
Repair on the surface		Surface	Average thickness 3 cm	
Patching the defaced parts (gap-sheet)				

Surface layer: Fine grading gap AS
Basic layer: Rough grading AS

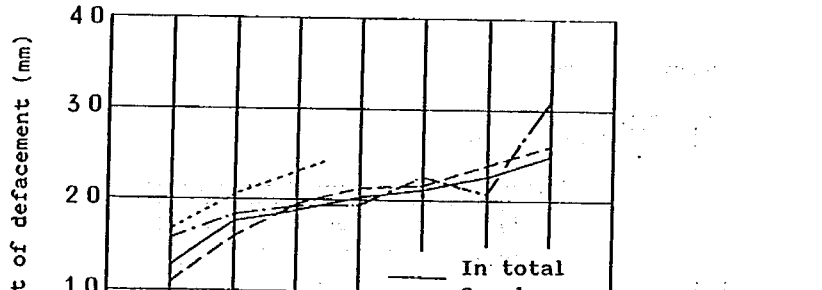


Fig-24 Relation between defacement made by prints of wheels and years past (By method)

6. Present situation of repair of pavements

1) Patching on defacement

Asphalt pavements in Hokuriku area are remarkable defaced by the prints of wheels owing to tire-chains and spiked tires in winter time and by plastic deformation caused by the heat in summer time. Defacement made by the prints of chains attached to wheels or spiked tires comes up to two thirds of all the amount of defacement made by the prints of wheels. (Fig-25) Based on this fact we have developed an easier reparing method which is excellently proof against defacement and is economical. In place of the former method of paving the surface layers, this method is to fill up only the parts defaced by the prints of wheels to restore the road to the

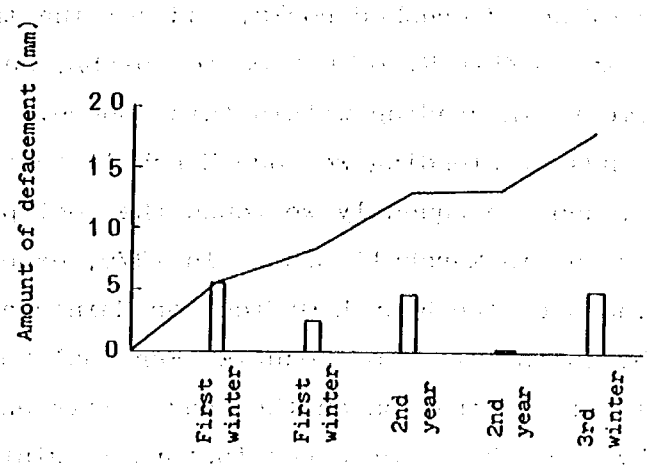


Fig-25 Yearly Alteration of the Amount of Defacement (At one place)

proper state. In this way we may keep the levelness of the road and smoothness of driving of vehicles much better and prolong the life of the roads.

We have experimented this repair method since 1978, and having proved its feasibility, its being economical, verified its applicable situations, we came to confirm the practicability of the method. We have been using this method since 1980 under the name of "gap-seat repair method".

Gap-seat is gap asphalt concrete for the thin layers with the blending newly devised taking advantage of the merits of discontinuous grading, and we had indoor experiments with it in 1978. The blending is made with the granules of 5 mm across at most as it is for thing layers, and gap rate is 10% at most considering the managerial phase in actual execution and

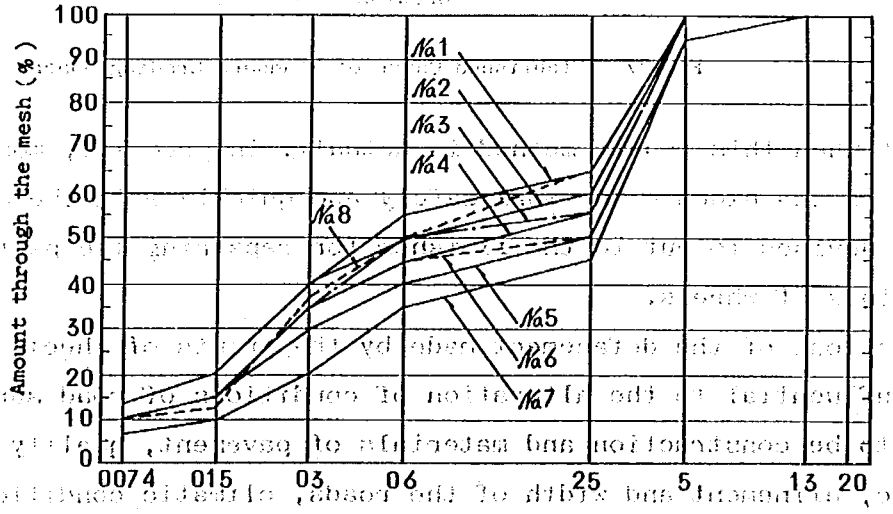


Fig-26 Combination Grading Curve

changeability of grading of No. 7 crushed rocks, and setting the amount through the sieve of 2.5 mm to 45-65%, of 0.6 mm to 35-55%, and of 0.074 mm to 7-13%, we made experiments in grading within this sphere. As shown in the Fig-26, with eight kinds of blending we made Marshall test, labelling test, wheeltracking test, and consequently we found the most proper sphere of grading and standard value in Marshall test. In 1979, experimental repairs were made at 17 places by the Branch Office for Maintenance of National Roads within the jurisdiction of Hokuriku Regional Construction Bureau, and as the result of examination on the feasibility and research on the defacement made by tire-chains or other factors in winter and by plastic deformation in summer, a grading standard was set up as shown in Fig-27.

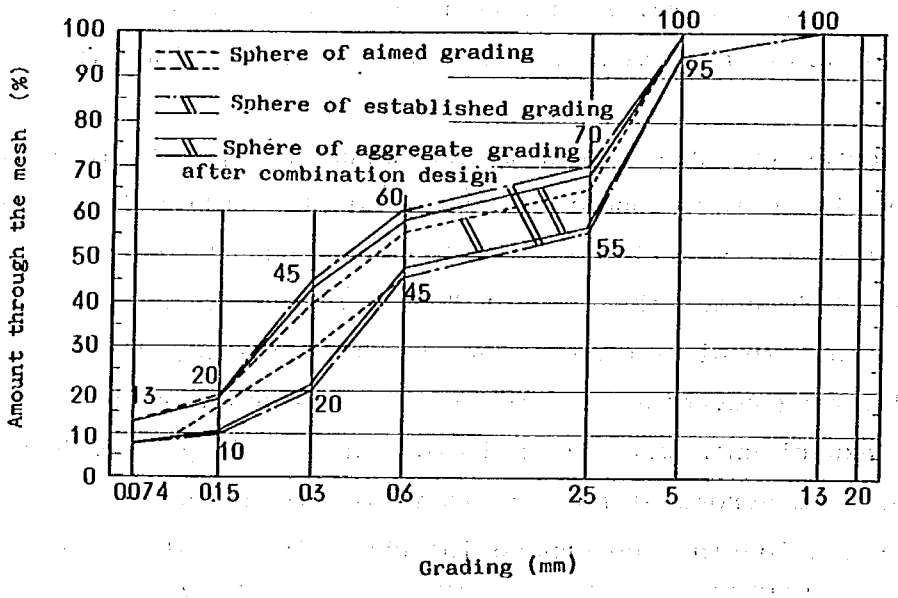


Fig-27 Established Carve of Standard Grading Sphere

At present when this repair method is actually in practice, mostly with machines, we can execute the work safely and quickly with the attachment specially devised to put to the finisher for repairing the parts defaced by the prints of wheels.

2) Projections of the defacement made by the prints of wheels

Factors influential to the alteration of conditions of road surface are presumed to be construction and materials of pavement, quality and quantity of traffic, alinement and width of the roads, climatic conditions, etc. To analyze all of these factors to establish a certain method of projections may be good for general use, but in that case we need tremendous amount of data as we have to study all the necessary data in each case; if

we can collect all the data, it should be hard to analyze the influence of each factor, and the method established should be too complicated to use practically.

In the other hand, in the actual situations, they may say, for example, "Pavement in the tunnel is much damaged", or "We have soft ground here, so subcidence or flow of the ground is much expected to happen", or "As this road is narrow, they drive on the same tracks, so that the road is easy to be defaced by the prints of wheels". As in the alteration of conditions of roads, each road has its characteristics and various factors are included in each situation, we decided to check the number of years since construction and the present conditions of the parts that have been defaced by the prints of wheels in each area based on the experiences of persons in charge of the roads. We also decided to see the alteration of defacement by years by each repair method, to work on the prospective amount of repair and usability of the method. We made investigations as stated above in about 80 cases and by forming them into groups according to the similar changes they showed, we finally could classify them into 13 blocks (Table-8) and established a method of projections as shown in Table-9. Consequently we found this method more applicable for actual situations than the former one which were only for general area and areas with much snow.

Table-8 The classification of forecast

Block No.	Terms of the object				
	Rt.	Regional devision	Construction materials	Method	Note
1	Rt. 7, 8, 17, 18, 41, 49	General area	No	Establish newly	
2				Treatment for 'the parts that defaced by the prints of wheels'	
3				Others	
4			Exist	Establish newly	Including the constructive materials of Rt.8 Kurobe, Kurosaki (soft and weak)
5				Treatment for 'the parts that defaced by the prints of wheels'	
6				Others	
7		Heavy snow area	No	Establish newly	
8				Treatment for 'the parts that defaced by the prints of wheels'	
9				Others	
10			Exist	Establish newly	
11				Treatment for 'the parts that defaced by the prints of wheels'	
12				Others	
13	Rt. 113, 116, 156, 157, 159, 160				Small traffic

Table-9 The form of forecast

Block No.	Length (100m)	Increase Quantity (mm/year)	The form of forecast Y:Quantity of 'defacement'(mm) X:Age of a road (years)	Note
1	1031	2.7	Y=7.7+2.65X	
2	370	6.3	Y=8.6+6.25X	
3	2960	4.9 2.8	Y=8.4+4.90X Y=12.6+2.80X	(Y<18.2, X<2) (Y≧18.2, X≧2)
4	18	2.7	Y=7.7+2.65X	Block No.1
5	47	6.3	Y=13.7+6.25X	Block No.2+5.1
6	121	4.9 2.8	Y=10.4+4.90X (Y:20.2 Y=14.6+2.80X (X: 2)	Block No.3+2.0
7	368	2.7	Y=7.7+2.65X	Block No.1
8	67	11.0	Y=11.9+10.96X	
9	616	4.2	Y=7.7+4.23X	
10	—	2.7	Y=7.7+2.65X	Block No.1
11	26	11.0	Y=12.8+10.96X	Block No.8+0.9
12	167	4.2	Y=14.7+4.23X	Block No.9+7.0
13	2238	1.6	Y=8.3+1.56X	

3) Present situation of repair and simulation for the future

Fig-28 shows the data of the repair actually done in the past and an example of prospective alteration of road conditions predicted with the method. It shows obviously that the increasing application of "patching the parts defaced by the prints of wheels" has checked the yearly worsening of the roads.

As for the projections, we repeated simulation for 5 years in the future each in three cases as shown in Table-10. As the result, we can say as follows:

Case 1 "Type of maintaining the status quo with increasing amount of repair"

* Amount of repair per year will be 25-28% (220 km - 250 km)
* MCI 4 under -- will be improved to the level of 1976
* MCI 5 under -- will be kept in the status quo

Case 2 "Type of repair in minimum with the amount same as in 1982"

* Amount of repair per year will be 20% (175 km)
* MCI 4 under -- will be kept in the status quo.
* MCI 5 under -- will become worse with years.

Case 3 "Type of level-down or decreasing in amount of repair"

* Amount of repair per year will be approximately 17% (150 km)
* Both MCI 4 under and 5 under --will be worse.

Table-10 Example of Standard for Repair

Case	Conditions and Repair	
Case 1	In the block where there is more than 300 m of road under 5 in MCI within 500 m distance	
	Average MCI o = 4 under Average MCI i = 4 under Average defacement = 40 mm above	Overlay
	Average defacement = 30 mm above	Repair on the surface
	Average defacement = 25 mm ~ 30 mm	Patching the defaced parts
Case 2	In the block where there is more than 300 m of road under 5 in MCI within 500 m distance	
	Average MCI o = 4 under Average MCI i = 4 under Average defacement = 40 mm above	Overlay
	Average defacement = 32 mm above	Repair on the surface
	Average defacement = 28 mm ~ 32 mm	Patching the defaced parts
Case 3	In the block where there is more than 300 m of road under 5 in MCI within 500 m distance	
	Average MCI 0 = 4 under Average MCI i = 4 under Average defacement = 40 mm above	Overlay
	Average defacement = 35 mm above	Repair on the surface
	Average defacement = 30 mm ~ 35 mm	Patching the defaced parts

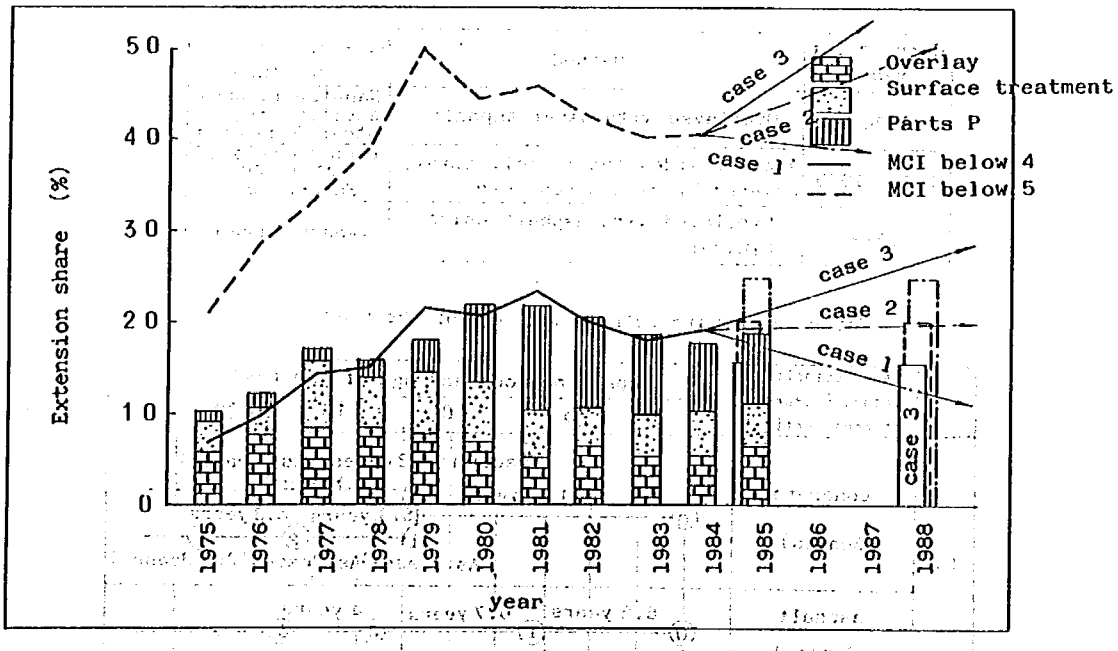


Fig-28 Amount of Repair and Alteration of MCI

As the result of what we have stated above, we can say that it is requisite to keep the amount of repair as shown in Case 1 in order to maintain the present road conditions at least. And the yearly increase of extension of MCI 5 under is due to the increase of temporary "patching the parts defaced by the prints of wheels", and it is now evident that ration of reparing the surface should become larger in the long run.

7. Concrete Pavement Repair

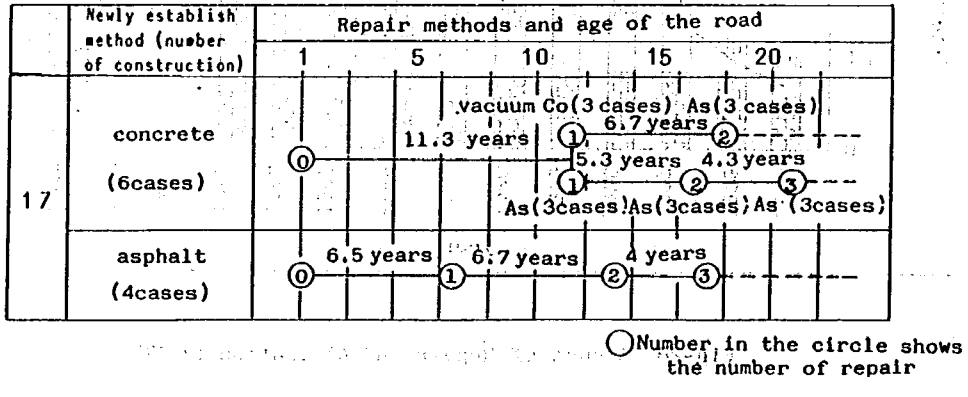
Road surface in tunnels and under snowsheds are remarkably damaged by tire chains and spiked tires. We have been reparing it by patching and overlaying with blended asphalt materials. However, having to repair so frequently, we have a lot of trouble in finding it's better countermeasure.

Hokuriku Regional Construction Bureau of Ministry of Construction did experimentaly various kinds of repairs in concrete pavements in the past, but no definite method has yet been found. The method of repair by overlaying with asphalt have mainly been used on route 17 as followed Table-11. Table-12 shows the number of the repair case and the yearly alteration. In Wanazu tunnel, concreat thin layer pavement based on overlaying with concrete was experimentally carried out and also PC pre-cast plate as replacement of concrete pavement plate was used on roads outside tunnels.

Table-11

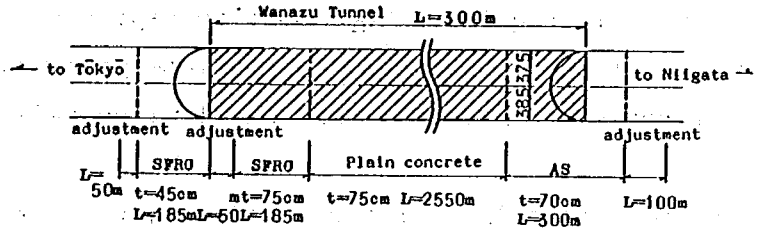
Carried out year	Method	Name of associate structure
1970 ~	Overlayed with usual asphalt	Nanatagiri-snow shelter and other 6 objects
1972 ~ 1975	About insides the tunnel, totaly covered by vacume concrete	Each tunnels of Futai and Funanosawa
1979	Overlayed with asphalt using Synopal	Wanazu tunnel

Table-12 Repair methods and age of the road

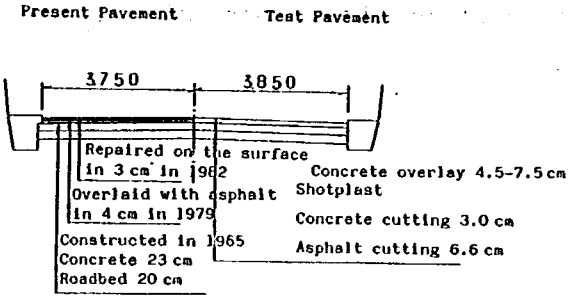


Wanazu District

Pean

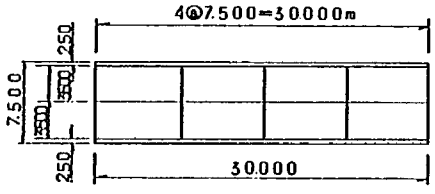


Section



Shimojima District

Pean



Section

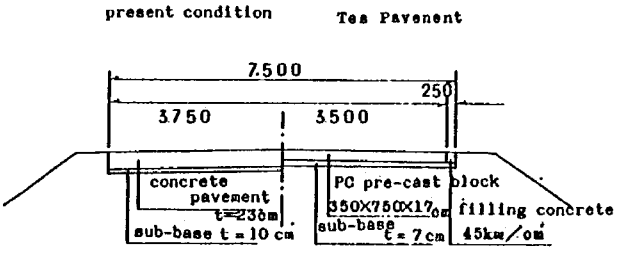


Fig-29 General view of Tes Pavement

As a result of this measure, we found the better way of repair for concrete pavement that is strong against defacement by wheels.

Post Script

It is obvious that it requirs a huge cost to repair roads judging from their actual condition. Therefore, appropriate preservation and effective utilization of roads are needed within the limited budget considering the total cost.

In maintaining and controlling roads in future, we would like to follow up the invertigation of variation for years in each place and utilize it in designing pavement structure for further development. We also hope to continue studying road materials and easy repair methods are strong enough against defacement by wheels and make efforts to rationalize construction of pavement.

Verification of Pavement Design Procedures Used in Saskatchewan

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ABSTRACT

The opportunity to compare the performance of a pavement to what was expected to occur is usually done by modelling the materials in a lab, in a test track, or in an in-field accelerated test.

The analysis of past performance to current or proposed standards over the lifetime of a pavement usually requires a considerable number of assumptions on traffic, material placement, material properties and environmental conditions.

The study cited in this paper was the development of a model used to analyize the effects of various truck hauling scenarios on pavement performance that could be used to reduce the associated pavement cost. The performance model was developed by evaluation of asphalt concrete stiffness, subgrade strengths, historic traffic and current pavement condition. The process produced stress-strain and deflection characterization of the pavement structure using the ELSYM-5 elastic layer analysis system.

Rutting/fatigue models were produced and comapred with field conditions. The results from the models, are compared to the existing design procedures for these subgrades and traffic volumes. The study compares the developed model, the Asphalt Institute, AASHTO P_T-25 and the Saskatchewan Design Procedure (modified Shell) required thickness with the actual thickness constructed.

The conclusions reached are that current North American design practices reflect what can be expected to occur over the design life of a pavement structure.

VERIFICATION OF PAVEMENT DESIGN PROCEDURES USED IN SASKATCHEWAN

by

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I INTRODUCTION

This paper is our excerpt of the analysis of Saskatchewan Highway No. 9 from Esterhazy to the American Border at North Portal, North Dakota (Figure 1). The mandate of the study was to analyse the performance of the existing highway from that analysis to make an engineering judgement on the effects of varying loadings, seasonal variations in haul schedules and the introduction of new types of haul units.

This paper is the description of the analysis of the existing pavement properties, and traffic to define the variables to be used in the ELSYM-5 elastic layer anaylsis system. This produced a model for further analysis on new loading configurations. The study also compared the existing pavement performance to the Asphalt Institute[2] AASHTO P_T-25,[3] Saskatchewan and Japan.[17]

The analysis is concentrated on Control Section (C.S.) 9-2 to 9-6 which are full depth asphalt concrete pavements. C.S. 9-1 was not included since it is to be reconstructed in the near future to

standards similar to C.S. 9-2 to 9-6. Sections on Highways 80 and 22 are not included as these are granular base pavements and would have required separate analysis. These sections are relatively minor in length and the performance of these sections can be inferred from the analysis of the full depth sections.

C.S. 9-2, C.S. 9-3 and C.S. 9-6 have been studied in detail. C.S. 9-4 is similar to 9-3 and C.S. 9-5 is similar to 9-6. This allows the extension of the data developed in detail to be used in these sections.

The project approach used is shown in Figure 2. The study included as built construction records and study of current literature to estimate asphalt conrete and subgrade strengths.

A review of the potash haul records and traffic data for the regular commercial traffic permitted an estimate of the traffic that has travelled on these control sections since they have been constructed in the late 1960's and early 1970's.

ELYSM-5 (Alborn, 1971), a computer program developed at the University of California, was used to evaluate the stresses and strains in the pavement structure on C.S. 9-2, 9-3 and 9-6. The calculated stresses and strains allowed for a comparison of predicted performance and actual performance. The performance

model was developed on a Commodore C-64 computer using the data from ELYSM-5 which itself required a larger, main-frame computer for its execution.

From the model developed, historical traffic and material properties, comparison of the different design procedures could be made with some confidence.

II ASPHALT STIFFNESS

The more sophisticated design procedures (Chevron, Shell and ELYSM-5) require an estimate of the asphalt stiffness. Since no test data is available on the stiffness of the Asphalt Concrete on Highway No. 9, the stiffnesses are estimated using the procedure outlined by Van der Poel (1954).

The stiffness varies with pavement temperature and as pavement temperature varies with depth, so does the stiffness. In this study the pavement is considered in two layers, a surface layer 75 mm thick (3") and the base layer 100 - 140 mm thick, (4-5 1/2") which varies from control section to control section.

The temperatures have been calculated using Barber's Solution[4] based on weather data from Broadview, Saskatchewan. Table 1 contains a resume of the weather data used. Figure 3 and 4 indicate the temperature of the pavement at various times of the day for each month. Figure 3 shows the temperature in the middle of

the surface layer and figure 4 shows the temperature in the middle of the lower layer. It can be noted that the extremes are much less pronounced in the lower layer, and that the extremes occur later in the day.

In this study, the pavement stiffnesses are calculated for 5.6°C (10°F) intervals such that the number of hours that the pavement is between say 4.4°C - 10°C (40°F - 50°F) can be compiled for each month. A summary of these results is shown in Table 2.

To limit the number of computer analyses the base temperatures have been grouped for each surface temperature interval such that when the surface temperature has been determined, the temperature of the base layer is set at representative values typical for all months. Table 3 indicates how the data from Table 2 has been reduced to a more general form.

In order to calculate the stiffness of the asphalt concrete, the stiffness of the asphalt itself must be established. It is known that as a pavement ages the viscosity increases and the penetration decreases. Furthermore, it has been established that the rate of aging varies with depth as the most severe aging occurs near the surface. A review of the literature indicated that it is difficult to evaluate aging beyond these generalities.

Data on the aging of asphalts is limited under similar environmental conditions but estimates can be made from previous studies on C.S. 22-02. MacLeod (1971)[9] reports on values of ring and ball temperatures and penetrations for cores taken 3 years after paving. Data is also available on cores taken 14 years after paving for the recycling contracts on C.S. 22-02. This section of highway is in the same environment and location as Highway No. 9, therefore the data was used for this study.

The limited data available does seem reasonable when compared to aging data found in the literature.[1,6,10] Table 4 shows aging rates that have been reported in similar climates. Since different asphalts were used in these studies, the percentage change from the original properties has been used for comparison.

There are conflicting views on the importance of depth on the rate of aging. Kennepohl (1981)[8] suggests that there is very little aging in the lower layers of full depth asphalt pavements. This is due mainly to the fact that air cannot get at the asphalt films in the lower layers due to dense nature of the overlying courses. Others McLeod (1978)[11] indicate that the retained Pen at 25°C is about 45% of the original for the base and about 35% for surface courses. Since there were no real guidelines to follow, the assumption has been made for this analysis that the underlying layers aged only at one-half the rate of the surface layer. The

error in this assumption is not as serious as it would first appear since the temperature variations throughout the day in the lower layers are much less than the surface layers; as such there is less variation in the stiffness of the lower layer on a given day.

Table 5 shows the values of the penetration index (P.I.) and the temperature at ($T_{R\&B}$) penetration of 800 pen (0.1 mm) at 800 pen. used in the study for year 1 and year 10. In the ELSYM-5 analyses data for year one were assumed to be appropriate for the first four years of pavement life. The values at year 10 are indicative of the last four years (1978-1982) of the pavement life. A first attempt at evaluating the parameters for "mid life" conditions (5-8 years) involved interpolating the P.I. and the $T_{R\&B}$ at 800 pen. The results were non-conclusive since for certain conditions the pavement was stiffer for the "mid life" case than the "young" and "old" pavements. Since both the $T_{R\&B}$ at 800 pen and P.I. were interpolated at any rate the procedure was changed such that the stiffness itself was interpolated between the "young" and "old" pavements, as more confidence could be placed in the values of "young" and "old" asphalts based on the previous discussion.

Once the bitumen stiffness has been calculated, the mix stiffness can be calculated using the following equations:

$$S_m = \left(S_b \left[1 + \left(\frac{2.5}{n} \right) \left(\frac{C'_v}{1 - C'_v} \right) \right] \right)^n$$

where S_m = mix stiffness kg/cm^2

S_b = Bitumen stiffness (kg/cm^2)

$n = 0.83 \log 10 (4 \times 10^5 / S_b)$

$$C_v = \frac{\text{volume of aggregate}}{\text{volume agg. + volume asphalt}}$$

$$C'_v = \frac{C_v}{(1 - (\text{Air voids} - 0.03)}$$

The mix stiffnesses have been calculated for two different cases. The first case used the data available from the laboratory compacted specimens and the maximum theoretical densities. This information was available from the surfacing log for each control section. However, in actual practice, the air voids in the pavement, as placed, are generally higher than the laboratory compacted specimens. The stiffnesses were then recalculated using 6% air voids which is thought to be more representative of the actual pavement as constructed. As will be seen later in this report, the resulting higher stiffnesses due to the higher air void contents yield unrealistic predictions of pavement behaviour for the asphalt cement stiffnesses used in this study.

Table 6 contains the mix characteristics used to calculate the mix stiffnesses.

Tables 7, 8, 9, 10 and 11 show the stiffnesses of asphalt concrete for each control section and at the two different air void contents. "young", "mid life" and "old" stiffnesses have been calculated for short loading times (10^{-2} sec) which are appropriate for moving vehicles, Schmidt (1975).

The stiffnesses contained in Tables 7, 8, 9, 10 and 11 span a wide spectrum of asphalt aging and mixes. The assumptions required to define these stiffnesses for each control section or mix cover the range of stiffnesses that could be expected under the study conditions.

III SUBGRADE STRENGTHS

Subgrade strength has be evaluated using three methods. The first method simply reviewed the construction logs to find the values used for the initial pavement design. The second method utilized the Saskatchewan Design Manual to find the apparent CBR using the historical traffic volumes knowing the thickness of the existing structure. The third method involved the use of ELSYM-5 to predict deflections. The subgrade CBR or resilient modulus were varied until the calculated deflection matched field deflections. The advantage of this method is that subgrade strengths can be evaluated at various times in the year.

In order to interpret Benkleman beam deflections taken at different temperatures most agencies use correction factors to obtain results

at a standard temperature. A study in Minnesota Lukanen (1979)[7] indicated that Benkleman beam deflections on full depth asphalt pavements were more sensitive to temperature changes than deflections on subbase-base asphalt pavements. He proposed a series of correction factors based on a standard temperature of 26.6°C (80°F). These have been plotted in Figure 5.

Also included in Figure 5 are temperature correction factors that have been proposed by other agencies. The RTAC (1977) factors are based on a standard temperature of 21.1°C (70°F). The Asphalt Institute (1969) factors are also shown and they too are based on a standard temperature of 21.1°C (70°F). It is worth noting that the deflections corrected to 21.1°C (70°F) are meant to be used with the Asphalt Institute Overlay Design Procedure (MS-17) currently used by Saskatchewan.

The current Saskatchewan correction factors are also shown in Figure 5 and these are based on 10°C (50°F). With the exception of the Minnesota evaluation, these methods do not distinguish between full depth and conventional pavement structures. Since ELSYM-5 is needed to back-calculate subgrade strength from deflection, the same calculations can be used to check temperature corrections, as the stiffness of the asphalt is dependent on the pavement temperature. For the most part, ELSYM-5 calculations agree with the Saskatchewan corrections, and these factors have been used in this study.

Figure 6 shows the variation of the average maximum Benkleman beam deflections on a year to year basis. C.S. 9-2, 9-3 and 9-4 indicate that maximum deflections have generally increased from year to year. Data on C.S. 9-5 is compatible with this data and it shows an improvement after the rehabilitation work in 1982. Data for C.S. 9-6, however, shows considerable scatter on a year to year basis based on the correction factors used. As a matter of interest, when the data for 9-6 was replotted using the Minnesota correction factors for temperature, the data was consistent with the other control sections showing increasing deflection with age.

Tables 12 to 16 contain deflection data for C.S. 9-2 to C.S. 9-6 on a monthly basis in 1982. The tables contain mean data uncorrected for temperature, mean data corrected for temperature using the Saskatchewan correction and the mean corrected deflections $\pm$ 2 standard deviations.

From the deflections it is obvious that the subgrade stiffness varies throughout the year. From the analysis of all five control sections, the subgrades can be classified into four basis conditions; winter when the material is frozen; May and June when the subgrade is weakest, and the rest of the year when it has stabilized. In this analysis May and June are treated separately as the temperature of the asphalt is considerably different in each month.

While this report interprets the spring conditon as May and June, it should be remembered that what is meant is a two-month period between the first of April and the end of June; the exact timing being a function of the weather in a given year.

Figures 7 to 10 contain the relationship between deflection and subgrade modulus as calculated by ELSYM-5. The temperatures on the curves cover the range of Benkleman readings. The raw deflection readings are entered in the graph until that line intersects the appropriate temperature plot and the corresponding subgrade modulus is read from the graph. Table 17 shows the inferred subgrade from the ELSYM-5 analysis using the 1982 Benkleman beam results.

Table 18 contains a comparison of the subgrade strength based on CBR tests prior to paving in the early 1970s, the apparent CBR's calculated from the Saskatchewan design graphs, and the aforementioned ELYSM-5 analysis. There tends to be a reasonable agreement between the design CBR and ELSYM-5, while the apparent CBR tends to be higher. The subgrade modulus for winter frozen conditons has been arbitrarily chosen at 690 MPa (100,000 psi).

IV PAVEMENT CROSS SECTIONS

The pavement cross sections are not identical on different control sections and indeed are not uniform within the control sections. To limit the analyses of the control sections, average thicknesses have been assigned to each control section. This procedure was

required mainly on C.S. 9-5 and 9-6 where the thicknesses varied considerably within the control section. Table 19 is a summary of the thicknesses used in this study.

V TRAFFIC ANALYSIS

The traffic on Highway No. 9 has consisted of two main types; potash hauler and the random mixture of other commercial truck traffic. Due to the vastly different characteristics of this latter type of traffic they are analyzed separately in this study.

The random commercial traffic was inferred from Annual Daily Traffic counts which did not include the heavy potash haulers. The ADT counts used were 1974, 1977, 1981 and 1982. A compound growth formula was used with these data to obtain initial ADT's for the year that the sections were paved, and a growth factor to predict future growth.

To convert this commercial traffic into EAL's, previous traffic pavement designs on Highway No. 9, as well as other primary highways in the province were studied. Assuming a 50%/50% haul split, and 14% commercial trucks, the total number of trucks could be calculated. Using an equivalency factory of 0.75 equivalent axles per truck, the total number of EAL's to date and forecasts for future traffic are tabulated in Table 20 for the commercial truck traffic only.

The past potash traffic which constitutes the main design parameters consisted of three types of haul units. Prior to 1973 the potash was hauled with standard 5 axle semi-trailer units. The payload was approximately 20 tons and the vehicle and an equivalent EAL factor of 2.52 to 3.32 per truck.

The second type of hauler was essentially the same type of 5 axle hauler with a pup tanker, but now hauling the legal maximums which have been in effect in Saskatchewan since 1974.

These units have been used to haul potash since 1979. In 1980, 17% of trips were made with these units. In 1981, 7% of the trips were hauled by these units, and up to October 1982, approximately 5% of the trips were made with these units.

The other unit that has been used on this haul has a gross weight of 614 kN (138,000 lbs) and carried the bulk of the potash.

Table 21 contains the number of trips per year that has been used in this study on the information available from these sources.

In the computer analysis which follows, the traffic was divided by month and by the hour of the day such that an estimate could be made of how much traffic would be travelled on a pavement while it has a certain strength. The commercial traffic was assumed to be

evenly distributed throughout the year, and the hourly distribution was similar to that published in the Saskatchewan truck traffic study[15] (Figure 11). The potash traffic was assumed to be evenly distributed throughout the day and night. The potash haulers stop for holidays traffic in July therefore traffic in July was assumed to be 3.2% of the total potash hauled. The remaining 96.8% was assumed to be evenly distributed throughout the rest of the year.

VI A DETAILED ANALYSIS OF C.S. 9-2, 9-3 AND 9-6

With the stiffness of the asphalt concrete, seasonal subgrade strength and historical traffic not established, a model can now be formulated to simulate fatigue cracking and rutting on Highway No. 9.

The procedure used to estimate the extent of fatigue damage caused by historic traffic is shown in Figure 12. The procedure was based on evaluating maximum tensile strain in asphalt concrete stiffness in all four annual subgrade conditions, for each axle weight and configuration. From these strains, failure criteria were used with the two governing strains and allowable repetitions to failure at the specified strain level were calculated. Actual load repetitions from historical traffic data were then divided by allowable repetitions and the quotients were added to obtain cumulative fatigue damage for all strain levels.

The random mixture of commercial vehicles can be calculated from AADT counts. Equivalent single axle loads (80 kN (18 kips), and tire pressure of 552 kPa (80 psi) are used to characterize this portion of the traffic. This axle weight and tire pressure were used as the standard axle.

The potash traffic consisted of two hauler types. One unit used the legal load limits and consisted of a conventional semi-trailer unit with a pup trailer; the second unit, which accounted for the largest percentage of potash moved, was the 9 axle unit. A 44.5 kN (10 kip) steering axle and a tandem 142 kN (32 kips) axle both operation at a tire pressure equal to 670 kPa (100 psi) were analyzed using ELSYM-5. These three axle types were analyzed for each control section, for each of the three asphalt age groups and each of the four various subgrade conditions throughout the year.

Representative values of asphalt concrete stiffness for each subgrade condition were used in the computer simulation. For example, in May, analyses corresponding to AC stiffnesses at 21.2°C (70°F), 15.6°C (60°F), 26.7°C (80°F) were calculated. Solutions for intermediate stiffnesses were interpolated from a plot of the above data. These graphs were plotted for each axle loading, control section, stiffness age group and subgrade condition. Figures 13, 14 and 15 are representative plots of maximum tensile strain at the underside of the asphalt concrete for the 145 kN (32 kip) tandem axle load for May, June and

the "rest of the year (winter excluded)". These particular curves were based on the air voids as calculated from the Marshall specimens at the time of construction.

Similar curves were plotted for vertical compressive strain. These curves facilitated interpolation of the governing strains for each temperature value within each of the four yearly subgrade conditions. The governing strains for both cracking and rutting are now known for every temperature interval for any given: subgrade condition, asphalt concrete age group, axle configuration and control section. These strains were then used in conjunction with various fatigue failure criteria to obtain allowable road repetitions to failure at given strain levels.

The Chevron criteria, developed by Santucci (1977)[12] was used as the fatigue cracking criteria. This criteria is also used in the new Asphalt Institute Design Procedure (M.S.1. 1982)[2]. To use this method, the curves reproduced in Figure 16, the maximum tensile strain as calculated by ELSYM-5 and stiffness of asphalt concrete at which this strain level occurs are required. This cracking criteria is influenced directly by the percentage of

asphalt in the mix and the air voids in the asphalt concrete. The following equation is used to account for these factors:

$$N_c = N_f \cdot 10^m$$

Where:

$$m = 4.84 \left(\frac{V_b}{V_v + V_b} - 0.69 \right)$$

N_c = corrected number of repetitions to failure

N_f = number of repetitions to failure at a given strain level. (E_t and modulus, E. See Figure 16.)

N_f = Number of repetitions to failure from Figure 16.

V_b = Volume of asphalt.

V_v = Volume of air.

A summary of these correction factors are found in Table 22. Three separate failure criteria for rutting, based on limiting subgrade compressive strain were employed in this analysis. The Shell Oil Company devised the original criteria for 19 m (3/4") ruts in highway pavements. These were later revised for airfield pavements and have been adapted for highway pavements. A Chevron criteria which estimates a 9.5 mm (3/8") rut was also analysed. Values for these failure criteria for rutting are shown in Table 23.

To perform a comprehensive fatigue analysis, actual load repetitions (n) are divided by allowable load repetitions (N) for each strain level encountered for each subgrade condition, control section, asphalt age grouping and axle configuration. The allowable repetitions have been structured in this manner and the

actual load repetitions must now be organized in the same manner. The raw traffic data acquired for this report was given as number of trucks per day and year. The commercial ADT traffic was assumed to be distributed equally throughout the twelve months of the year. With the aid of Barber's solution and the hourly traffic distribution, Figure 11, the percentage of daily traffic in each temperature interval for each month was calculated. It was further assumed that hour 21 from Barber's solution, corresponded to 7:00 a.m. and hour 9 to 7 p.m. Table 24 summarized the percentage of ADT traffic for each temperature in each month.

The potash truck traffic distribution was calculated with a similar procedure but with one major difference. The potash traffic was distributed equally in each hour of the day. This yields different percentages in each temperature interval for each month, Table 25 shows this distribution. The potash traffic supplied by IMC indicates that approximately 3.2% of yearly traffic occurs in July and 8.8% in each of eleven remaining months.

Traffic data was organized into temperature intervals for each subgrade condition, (traffic had previously been structured in temperature intervals for each month). This was accomplished by summing the traffic for the months with the same subgrade condition and each temperature interval. This exercise now organizes actual load repetitions in the same format as allowable load repetitions.

This methodology for fatigue analysis is necessary for the acquisition of meaningful results because of the wide variance in strain levels caused by: different loading coditions, asphalt concrete stiffnesses, various subgrade conditions throughout the year and truck traffic distributions.

A fatigue analysis was performed using the historical traffic on control sections: 9-2, 9-3 and 9-6 for the different asphalt concrete stiffnesses used on each section. Table 26 is a summary of the fatigue damage incurred due to historic traffic for each control section. An examination of Table 26 indicates several trends that are worth noting. The fatigue damage calculated using stiffnesses based on the air voids obtained from the laboratory compacted specimens is more realistic than the fatigue damage estimated by the 6% field air void contents. A field inspection has indicated that most control sections show 9.5 mm (3/8") rut, and C.S. 9-6 shows some 19 mm (3/4") rutting. C.S. 9-3 and 9-6 have significant areas that show minor fatigue cracking. Using the 6% air void criteria, there should be no sign of fatigue cracking nor should there be rutting greater than 9.5 mm (3/8") rut. It could well be that the stiffnesses at 6% air voids when used with aging rates other than those asssumed with the asphalt cement would give acceptable results. However, stiffnesses based on the laboratory air voids when used with the asphalt stiffnesses described previously, do give acceptable results and these are used in the remainder of this study.

From the data it is clear that the original Shell criteria for 19 mm (3/4") ruts does not yield satisfactory answers. This criteria does predict very serious rutting which is not apparent in the field. The revised Shell criteria however indicates that the control sections should be approaching a 3/4" (19 mm) rut ($n/N > 1$). Only a few sections in the field show a (19 mm) rut but almost all sections show at least a 10 mm (3/8") rut. The Chevron criteria suggests that this should be the case. The fact that the Chevron and revised Shell rutting criteria do correctly predict rutting behavior whereas the Original Shell Criteria does not, was also found to be the case on the James Bay Road MacLeod and Hurd, (1982)[10].

A couple of factors might account for the fact that the revised Shell criteria slightly over-estimates the amount of 19 mm (3/4") rutting. In the field, it was noted that ruts were not confined into a defined wheel path. The severe ruts were approximately 1.2 m (4') wide. If a distribution factor for traffic wander is included then the n/N or fatigue damage is less than one.

A second factor is evident on examination of Table 26. A change in the air voids from 4% to 6% changed the rutting damage factors much more significantly than it changed the cracking factors. As the air voids change, the increase in stiffness (Table 7) is offset by a correction for air voids (Table 22). No such compensating factor exists for the rutting criteria. A small change in the air voids

would change the cracking model only slightly but would reduce the rutting predictions considerably. This refinement was not considered in this report. Given the assumptions required the predictive model as developed, is adequate.

The cracking analysis indicates that the roadway should not be completely fatigue cracked but that initial signs of distress should be becoming apparent since over 80% of the fatigue life has been used. The weaker sections should be showing some sign of distress. This is essentially what has happened. A field inspection has indicated that sections 9-3 and 9-6 were beginning to show extensive minor cracking in the outer wheel path.

In conclusion, the cracking model correctly predicts the observed distresses. The rutting model correctly predicts 9.5 mm (3/8") ruts and slightly over predicts 19 mm (3/4") ruts based on the revised Shell criteria.

VII COMPARISON WITH EXISTING DESIGN PROCEDURES

The equivalencies have been calculated for the past traffic and it is opportune to look at what thicknesses existing design procedures would require for these subgrades and traffic volumes. The reason for this exercise is twofold:

(1) The model would be confirmed if the thicknesses required using these methods are close to the existing values which are now approaching the end of their design life.

(2) It provides an opportunity to access the applicability of certain design procedures.

Table 27 contains the results of this analysis. The pavement thicknesses that have given satisfactory performance are within the range of the different design procedures. The applicability of each method varies from section to section. The JPA procedure shows a greater variation which could be due to CBR evaluation procedure and converting 80 kN single axles to 5 tonne axles.

VIII CONCLUSIONS

The use of a rational design procedure using elastic layer programs such as ELSYM-5 to verify a design procedure to actual pavement performance requires considerable estimation of critical variables to carry out the study. Even with the required assumptions and estimates one can develop a model to help define design standards that can be applied on a highway system.

The differences (Table 27) in many cases are within construction tolerances that would have an effect on performance evaluations. The larger difference shown by the Japan Road Association Design procedure may be attributed to the method used to measure CBR and changing 80 kN axles to 5 tonne axles.

The use of elastic larger programs such as ELSYM-5 provide the pavement engineers with a tool to produce design criteria that can be field verified given enough information on the in field conditions.

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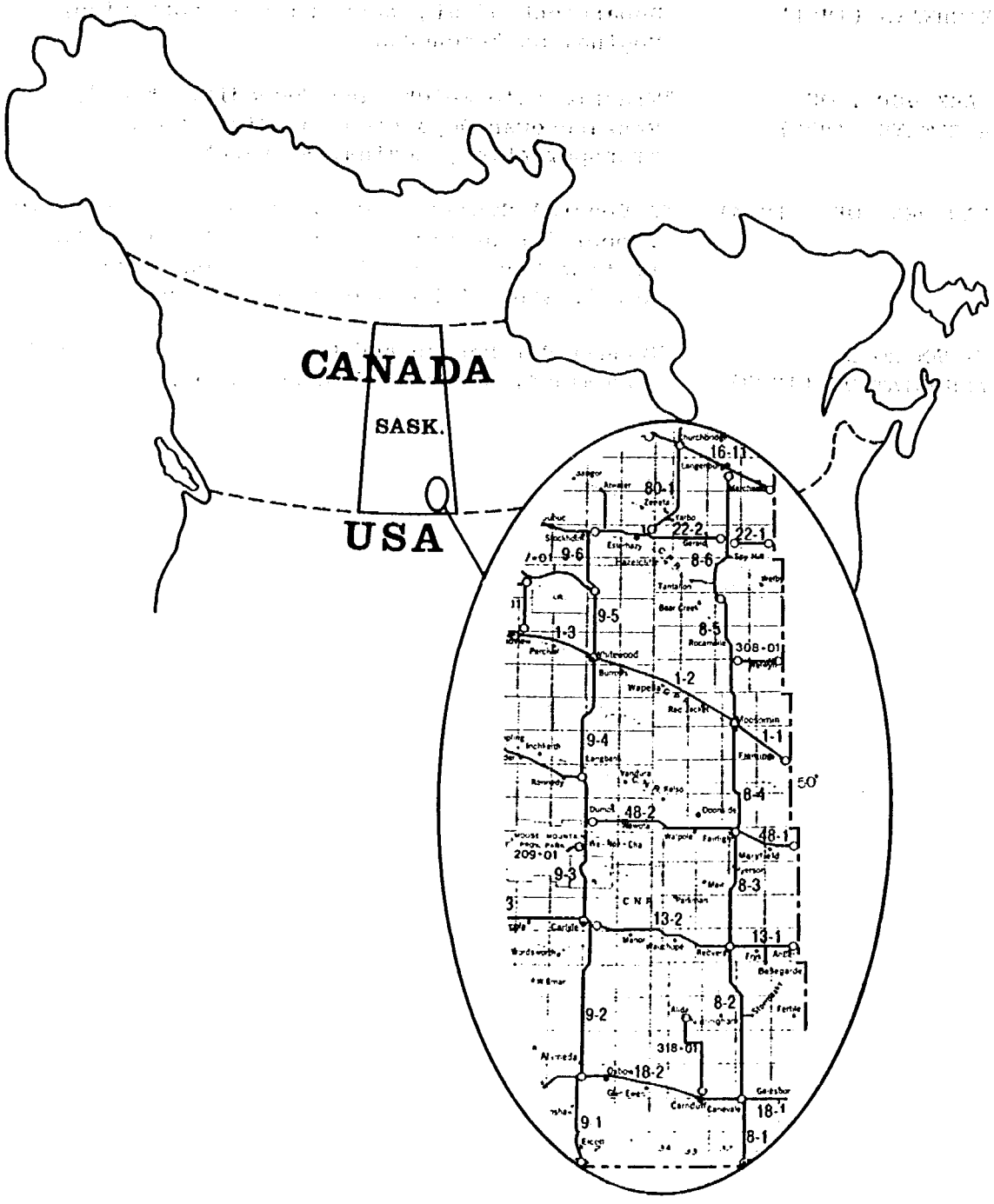


FIGURE 1 - LOCATION OF THE STUDY PROJECT

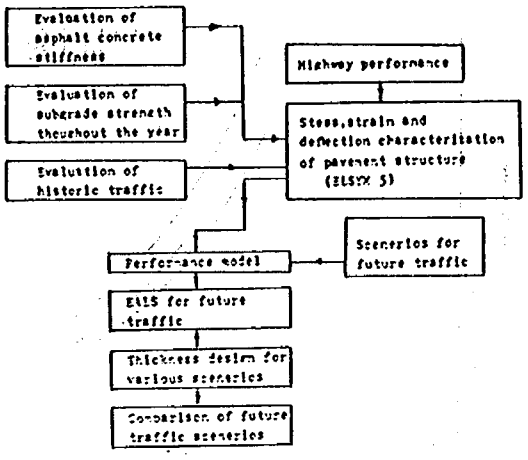


FIGURE 2 - FLOWCHART OF STUDY METHOD

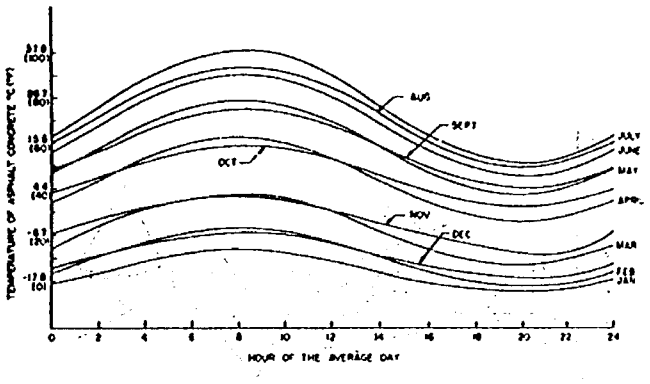


FIGURE 3 - TEMPERATURE OF ASPHALT CONCRETE 38 mm BELOW SURFACE

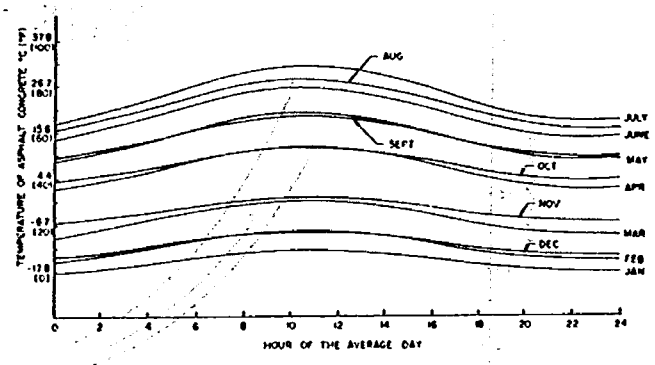


FIGURE 4 - TEMPERATURE OF ASPHALT CONCRETE 140 mm BELOW SURFACE

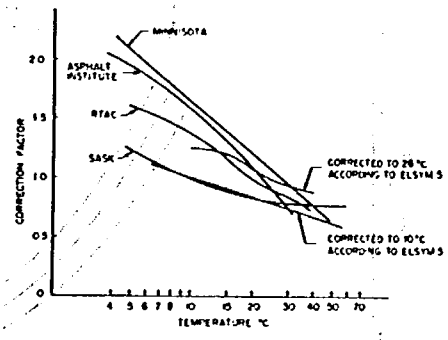


FIGURE 5 - TEMPERATURE CORRECTION FACTOR FOR BENKELMAN BEAM TESTS

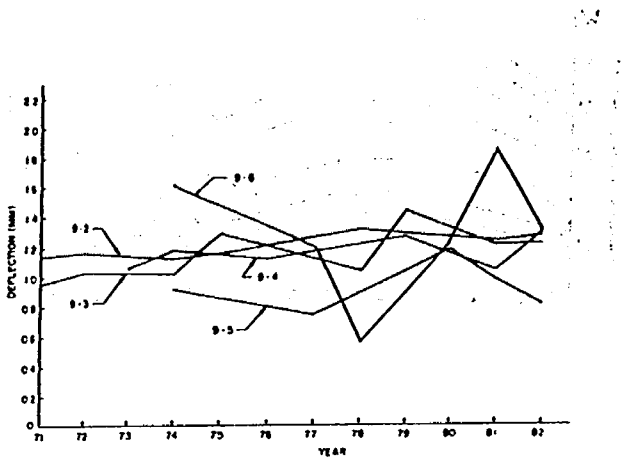


FIGURE 6 - MAXIMUM BENKLEMAN BEAM DEFLECTIONS VERSUS YEAR

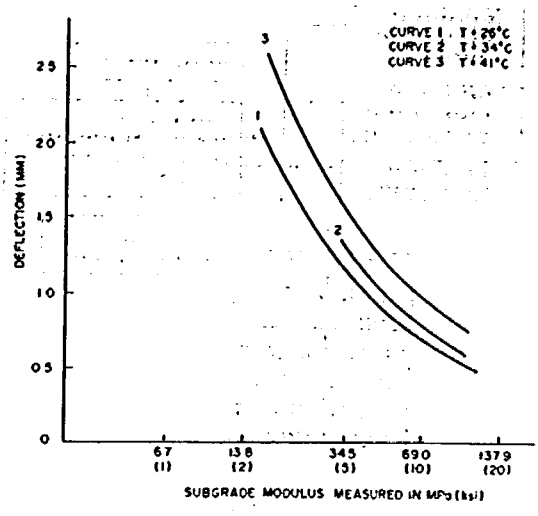


FIGURE 7 - DEFLECTION VERSUS SUBGRADE MODULUS FOR CONTROL SECTION 9-2

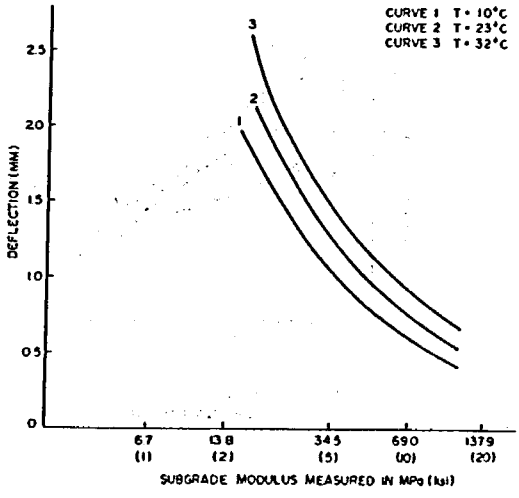


FIGURE 8 - DEFLECTION VERSUS SUBGRADE MODULUS FOR CONTROL SECTION 9-3

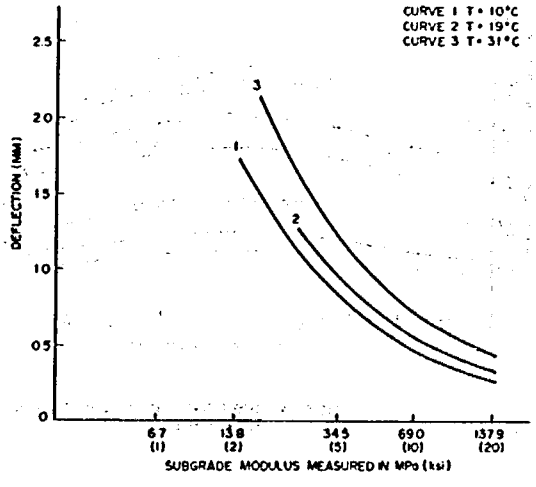


FIGURE 9 - DEFLECTION VERSUS SUBGRADE MODULUS FOR CONTROL SECTION 9-4

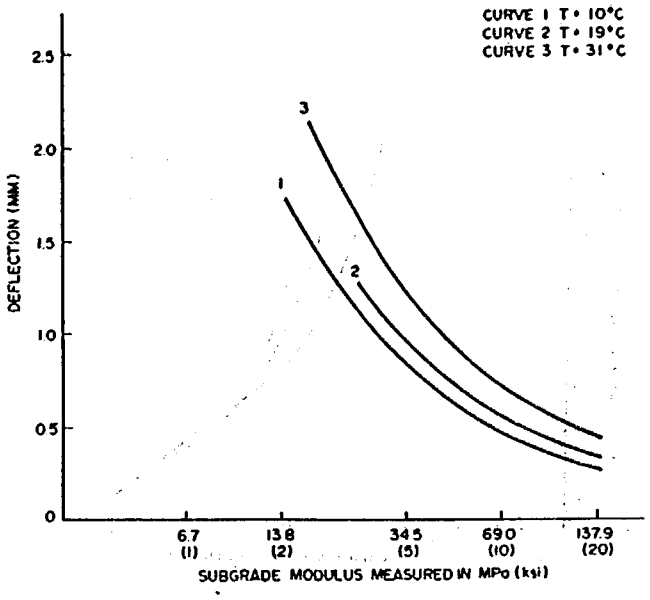


FIGURE 10 - DEFLECTION VERSUS SUBGRADE MODULUS FOR CONTROL SECTION 9-5 AND 9-6

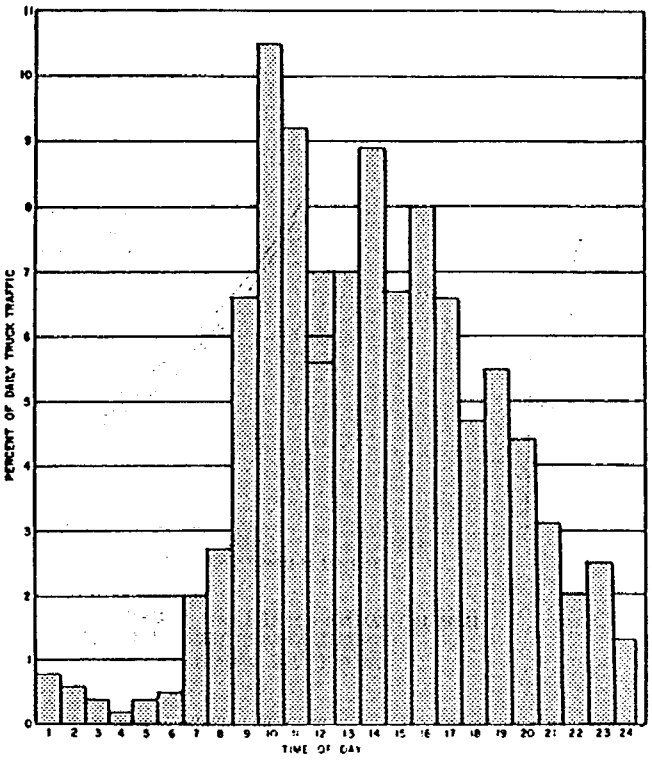


FIGURE 11 - HOURLY TRUCK DISTRIBUTION

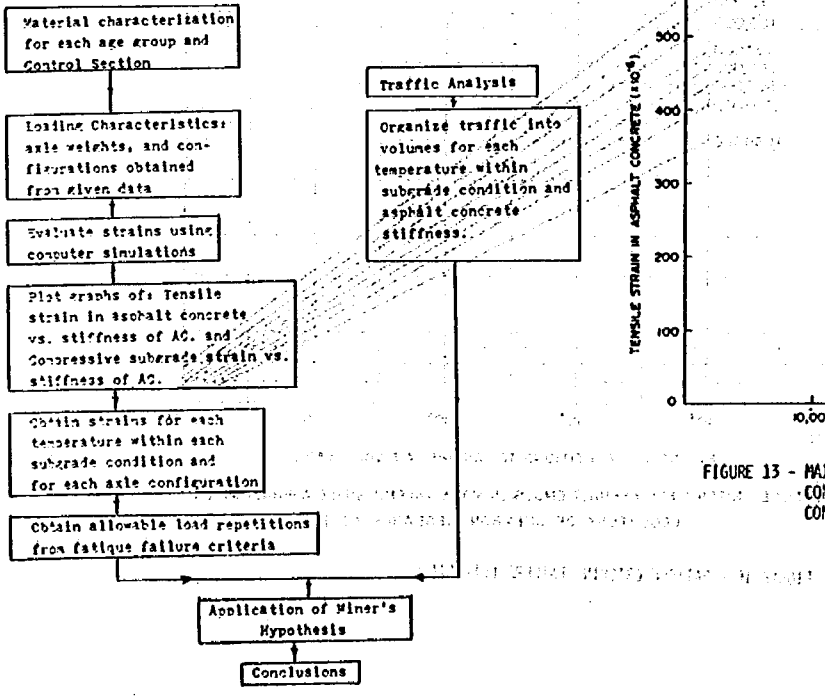


FIGURE 12 - FLOWCHART FOR HISTORICAL FATIGUE DAMAGE

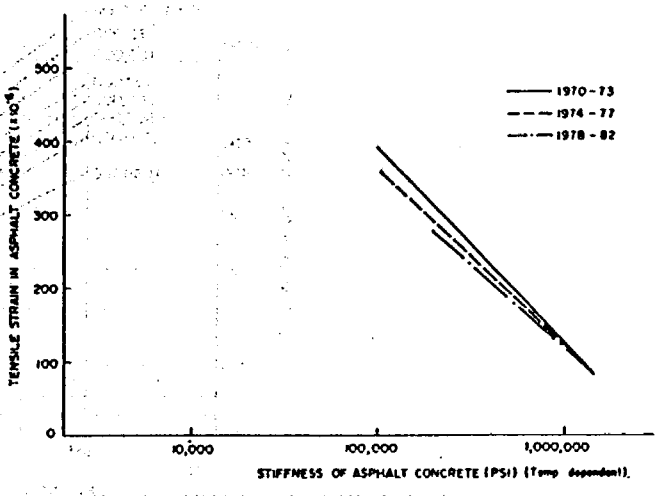


FIGURE 13 - MAXIMUM TENSILE STRAIN IN ASPHALT CONCRETE VERSUS STIFFNESS OF ASPHALT CONCRETE (MAY SUBGRADE)

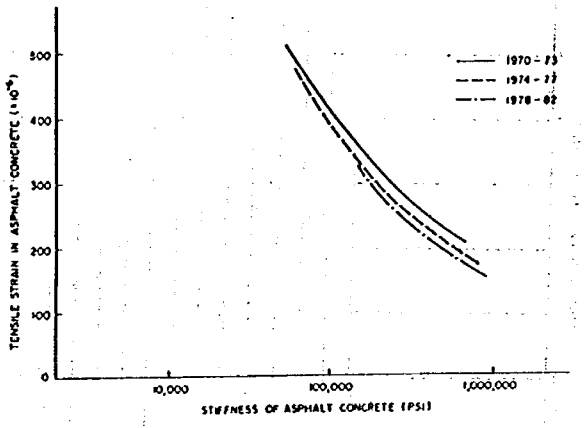


FIGURE 14 - MAXIMUM TENSILE STRAIN IN ASPHALT CONCRETE VERSUS STIFFNESS OF ASPHALT CONCRETE (JUNE SUBGRADE)

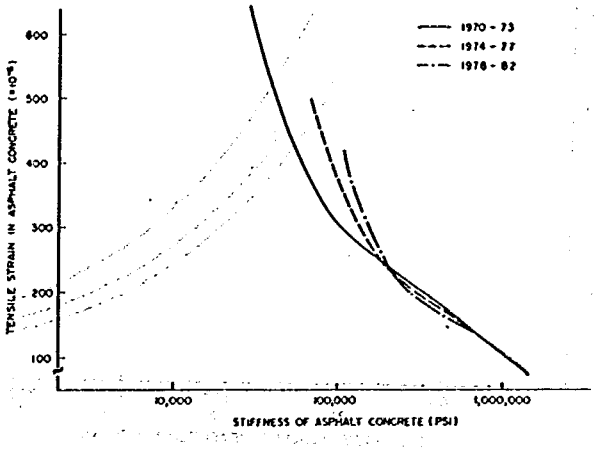
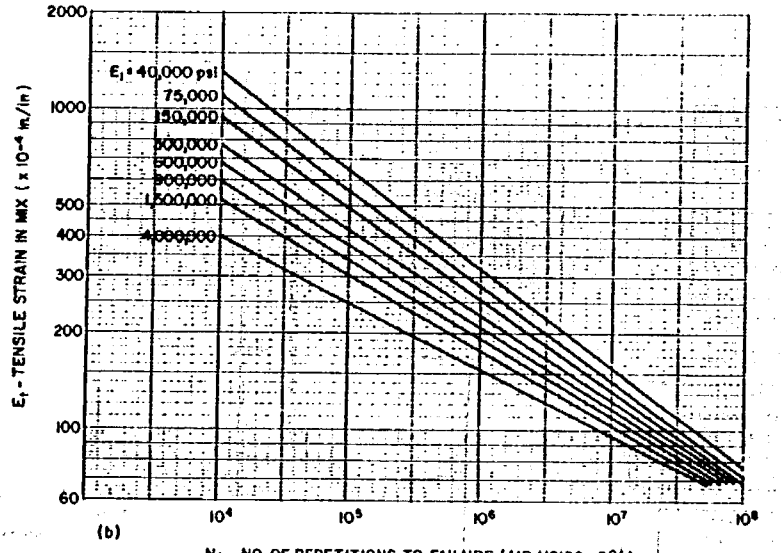


FIGURE 15 - MAXIMUM TENSILE STRAIN IN ASPHALT CONCRETE VERSUS STIFFNESS OF ASPHALT CONCRETE (SUBGRADE "REST" OF THE YEAR)



FATIGUE CRITERIA FOR ASPHALT EMULSION MIXES (b) EMULSIFIED ASPHALT MIXES
(COURTESY OF CHEVRON RESEARCH CO.)

FIGURE 16 - FATIGUE CRACKING FAILURE CRITERIA.

TABLE 1 - WEATHER DATA FROM BROADVIEW, SASKATCHEWAN

	JAN.	FEB.	MARCH	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
WIND SPEED KMPH (MPH)	(10.7)	(10.6)	(10.2)	(11.6)	(11.5)	(10.5)	(9.7)	(9.2)	(10.2)	(10.6)	(10.2)	(10.4)
TEMPERATURE DAILY MEAN °C	17.2	17.1	16.4	18.7	18.5	16.9	15.6	14.8	16.4	17.1	16.4	16.7
	-17.0	-14.6	- 8.9	2.3	9.4	14.2	17.8	16.6	10.4	5.2	- 5.4	-13.2
(°F)	(1.4)	(5.7)	(16.0)	(36.2)	(49.0)	(57.6)	(64.1)	(61.9)	(50.8)	(41.4)	(22.2)	(8.2)
MEAN MAXIMUM °C	-11.5	- 8.8	- 3.4	8.4	16.4	20.7	24.8	23.8	17.3	11.8	- 0.6	- 8.1
(°F)	(11.3)	(16.2)	(25.9)	(47.1)	(61.5)	(69.3)	(76.7)	(74.8)	(63.1)	(53.2)	(31.0)	(17.5)
MEAN MINIMUM °C	-22.6	-20.4	-14.4	- 3.7	2.4	7.7	10.8	9.4	3.6	- 1.3	-10.3	-18.4
(°F)	(-8.6)	(-4.8)	(6.1)	(25.3)	(36.4)	(45.9)	(51.4)	(49.0)	(38.4)	(29.7)	(13.4)	(-1.2)
SOLAR RADIATION LANGLEYS	125	225	320	420	465	510	575	470	340	210	180	150

TABLE 2 - SURFACE ASPHALT CONCRETE TEMPERATURE AND CORRESPONDING BASE TEMPERATURES AS CALCULATED BY BARBER'S SOLUTION

MONTH	SURFACE 38 mm TEMPERATURE °C	(°F)	BASE 140 mm TEMPERATURE °C	(°F)	MONTH	SURFACE 38 mm TEMPERATURE °C	(°F)	BASE 140 mm TEMPERATURE °C	(°F)
JAN.	-17.8	(0)	-17.8	(0)	JULY	10.0	(50)	15.6	(60)
	-12.8	(10)	-17.2	(10)		15.6	(60)	21.1	(70)
FEB.	-17.8	(0)	-12.2	(10)		21.1	(70)	21.1	(70)
	-12.2	(10)	-12.2	(10)		26.7	(80)	21.1	(70)
	- 6.7	(20)	-12.2	(10)		32.2	(90)	26.7	(80)
MARCH	-12.2	(10)	- 6.7	(20)	AUG.	37.8	(100)	29.4	(85)
	- 6.7	(20)	- 6.7	(20)		10.0	(50)	18.3	(65)
	- 1.1	(30)	- 6.7	(20)		15.6	(60)	21.1	(70)
	4.4	(40)	- 1.1	(30)		21.1	(70)	21.1	(70)
APRIL	- 1.1	(30)	4.4	(40)		26.7	(80)	21.1	(70)
	4.4	(40)	10.0	(50)		32.2	(90)	23.9	(75)
	10.0	(50)	10.0	(50)	SEPT.	4.4	(40)	10.0	(50)
	15.6	(60)	10.0	(50)		10.0	(50)	15.6	(60)
MAY	4.4	(40)	10.0	(50)		15.6	(60)	15.6	(60)
	10.0	(50)	15.6	(60)		21.1	(70)	15.6	(60)
	15.6	(60)	15.6	(60)	OCT.	- 1.1	(30)	4.4	(40)
	21.1	(70)	15.6	(60)		4.4	(40)	4.4	(40)
	26.7	(80)	15.6	(60)		10.0	(50)	4.4	(40)
JUNE	10.0	(50)	21.1	(70)	NOV.	15.6	(60)	10.0	(50)
	15.6	(60)	21.1	(70)		-12.2	(10)	- 6.7	(20)
	21.1	(70)	21.1	(70)		- 6.7	(20)	- 3.9	(25)
	26.7	(80)	21.1	(70)		- 1.1	(30)	- 1.1	(30)
	32.2	(90)	21.1	(70)	DEC.	-17.8	(0)	-12.2	(10)
						-12.2	(10)	-12.2	(10)
						- 6.7	(20)	-12.2	(10)

TABLE 3 - SURFACE TEMPERATURE AND BASE TEMPERATURE FOR ALL MONTHS

SURFACE TEMPERATURE @ 38 mm (1 1/2") °C	(°F)
-17.8	(0)
-12.2	(10)
- 6.7	(20)
- 1.1	(30)
4.4	(40)
10.0	(50)
15.6	(60)
21.1	(70)
26.7	(80)
32.2	(90)
37.8	(100)

BASE TEMPERATURE @ 140 mm (5 1/2") °C	(°F)
-17.8	(0)
-12.2	(10)
-12.2	(10)
- 6.7	(20)
-12.2	(10)
- 6.7	(20)
- 3.9	(25)
- 6.7	(20)
- 1.1	(30)
4.4	(40)
- 1.1	(30)
4.4	(40)
10.0	(50)
4.4	(40)
10.0	(50)
15.6	(60)
18.3	(65)
10.0	(50)
15.6	(60)
21.1	(70)
15.6	(60)
21.1	(70)
15.6	(60)
21.1	(70)
21.1	(70)
26.7	(80)
26.7	(80)
29.4	(85)

TABLE 4 - % CHANGE OF ASPHALT PROPERTY DUE TO AGING

YEARS AFTER PAVING	CULLEY (1978)	ANDERSON (1975)	SHELL DESIGN PROC (1978)	JAMES BAY	DATA FOR 22-03 AC 5	DATA FOR 22-03 AC 6	MACLEOD (1975)	USE IN THIS STUDY
Penetration at 77°								
Laydown	67%		70%					70
1	48%	63%						55%
2	49%				37%	46%		50
3	46%			80%				
4		55%						
8		48%					40%	45
13					29%	21%		30
15							35%	
Penetration at 40°								
Laydown	70%							
1	53%							
3	58%							
Ring and Ball Temperature								
1		111%						110
2					117%			
3		112%		122%				112
5		112%						
8		113%					110%	113
13							117%	
15							122%	120
Penetration Index								
1	-0.6%		Similar		-1.4%	-0.6%		
2								
3	-0.7%							
15					+1.8%	+2.6%		

TABLE 5 - ASPHALT PROPERTIES USED FOR STIFFNESS CALCULATIONS

YEAR	AC 6		AC 5	
	TEMPERATURE AT PEN 800	P.I.	TEMPERATURE AT PEN 800	P.I.
SURFACE	NOT APPLICABLE			
0-4			38°C	-1.0
8-12			55°C	-1.0
BASE				
0-4	42°C	-0.5	38°C	-1.0
8-12	50°C	1.5	45°C	-0.5

TABLE 6 - MIX CHARACTERISTICS

CONTROL SECTION	YEAR PAVED	BASE OR SURFACE	DATA FROM FIELD LOGS	@ 6% AIR VOIDS
C.S. 9-2	1970	Base AC6	% AC : 5.79%	5.79%
			Airvoids : 4.3%	6.0%
			CV : 0.852	0.889
		Surface AC5	% AC : 5.88%	5.88%
			Airvoids : 3.8%	6%
			CV : 0.853	0.887
C.S. 9-3	1970	Base AC6	% AC : 6.14%	6.14%
			Airvoids : 4.1%	6%
			CV : 0.845	0.889
		Surface AC5	% AC : 6.83%	6.83%
			Airvoids : 4.9%	6%
			CV : 0.824	0.875
C.S. 9-5 - 9-6	1972	Surface AC5	% AC : 5.7%	5.7%
			Airvoids : 3.1%	6%
			CV : 0.863	0.692

TABLE 7 - ASPHALT STIFFNESS (P_a) CONTROL SECTION 9-2

(Stiffnesses Based on Air Voids Obtained by Laboratory Compacted Specimens)

TEMPERATURE		YOUNG (1970-73)		MIDLIFE (1974-77)		OLD (1978-82)	
		SURFACE	BASE	SURFACE	BASE	SURFACE	BASE
°C	(°F)						
-17.8	(0)	2.7×10^{10}	2.1×10^{10}	2.3×10^{10}	1.6×10^{10}	2.0×10^{10}	9.3×10^{9}
-12.2	(10)	2.1×10^{10}	1.7×10^{10}	2.0×10^{10}	1.3×10^{10}	1.8×10^{10}	9.0×10^{9}
- 6.7	(20)	1.8×10^{10}	1.5×10^{10}	1.5×10^{10}	1.2×10^{10}	1.2×10^{10}	8.3×10^{9}
- 1.1	(30)	1.2×10^{10}	1.1×10^{10}	1.2×10^{10}	8.3×10^{9}	1.1×10^{10}	5.9×10^{9}
4.4	(40)	8.3×10^{9}	7.6×10^{9}	8.3×10^{9}	5.8×10^{9}	8.3×10^{9}	4.0×10^{9}
10.0	(50)	6.1×10^{9}	4.2×10^{9}	4.9×10^{9}	3.4×10^{9}	5.6×10^{9}	2.6×10^{9}
15.6	(60)	2.0×10^{9}	2.8×10^{9}	3.0×10^{9}	2.3×10^{9}	4.1×10^{9}	1.9×10^{9}
21.1	(70)	1.1×10^{9}	2.0×10^{9}	1.8×10^{9}	1.8×10^{9}	2.4×10^{9}	1.5×10^{9}
26.7	(80)	5.7×10^{8}	2.0×10^{9}	1.0×10^{9}	1.8×10^{9}	1.4×10^{9}	1.5×10^{9}
32.2	(90)	3.2×10^{8}	9.0×10^{8}	6.2×10^{8}	9.7×10^{8}	9.0×10^{8}	1.0×10^{9}
37.8	(100)	1.7×10^{8}	5.0×10^{8}	4.1×10^{8}	5.2×10^{8}	6.3×10^{8}	5.4×10^{8}

TABLE 8 - ASPHALT STIFFNESS (P_a) CONTROL SECTION 9-3

(Stiffnesses Based on Air Voids Obtained by Laboratory Compacted Specimens)

TEMPERATURE		YOUNG (1970-73)		MIDLIFE (1974-77)		OLD (1978-82)	
		SURFACE	BASE	SURFACE	BASE	SURFACE	BASE
°C	(°F)						
-17.8	(0)	2.1×10^{10}	2.0×10^{10}	2.3×10^{10}	9.3×10^{9}	1.4×10^{10}	9.7×10^{9}
-12.2	(10)	1.6×10^{10}	1.5×10^{10}	1.6×10^{10}	8.3×10^{9}	1.2×10^{10}	8.3×10^{9}
- 6.7	(20)	1.2×10^{10}	1.3×10^{10}	1.4×10^{10}	6.9×10^{9}	8.3×10^{9}	7.6×10^{9}
- 1.1	(30)	8.3×10^{9}	9.7×10^{9}	8.3×10^{9}	5.0×10^{9}	7.6×10^{9}	5.3×10^{9}
- 4.4	(40)	5.2×10^{9}	6.6×10^{9}	5.5×10^{9}	3.5×10^{9}	5.5×10^{9}	3.7×10^{9}
10.0	(50)	2.5×10^{9}	4.6×10^{9}	3.9×10^{9}	2.4×10^{9}	3.5×10^{9}	2.7×10^{9}
15.6	(60)	1.2×10^{9}	3.4×10^{9}	2.0×10^{9}	1.4×10^{9}	2.5×10^{9}	1.9×10^{9}
21.1	(70)	6.0×10^{8}	1.8×10^{9}	1.2×10^{9}	9.7×10^{8}	1.4×10^{9}	1.2×10^{9}
26.7	(80)	3.0×10^{8}	1.0×10^{9}	6.0×10^{8}	6.9×10^{8}	8.3×10^{8}	8.3×10^{8}
32.2	(90)	1.6×10^{8}	9.0×10^{8}	3.3×10^{8}	4.8×10^{8}	4.8×10^{8}	4.8×10^{8}
37.8	(100)	9.3×10^{7}	4.5×10^{8}	1.4×10^{8}	3.8×10^{8}	3.3×10^{8}	4.8×10^{8}

TABLE 9 - CONTROL SECTION 9-3 (6% AIR VOIDS)

(Stiffnesses Based on Air Voids Assumed in Asphalt Mat or Highway)

TEMPERATURE		YOUNG (1970-73)		MIDLIFE (1974-77)		OLD (1978-82)	
		SURFACE	BASE	SURFACE	BASE	SURFACE	BASE
°C	(°F)						
-12.2	(10)	2.8×10^{10}	2.8×10^{10}	2.6×10^{10}	2.2×10^{10}	2.3×10^{10}	1.7×10^{10}
- 6.7	(20)	2.3×10^{10}	2.6×10^{10}	2.0×10^{10}	2.1×10^{10}	1.7×10^{10}	1.5×10^{10}
- 1.1	(30)	1.7×10^{10}	1.9×10^{10}	1.6×10^{10}	1.5×10^{10}	1.5×10^{10}	1.1×10^{10}
4.4	(40)	1.1×10^{10}	1.4×10^{10}	1.2×10^{10}	1.1×10^{10}	1.2×10^{10}	8.3×10^{9}
10.0	(50)	6.0×10^{9}	1.0×10^{10}	6.9×10^{9}	8.3×10^{9}	8.3×10^{9}	5.7×10^{9}
15.6	(60)	5.2×10^{9}	6.2×10^{9}	4.6×10^{9}	5.3×10^{9}	6.0×10^{9}	3.3×10^{9}
21.1	(70)	1.8×10^{9}	4.5×10^{9}	2.8×10^{9}	4.0×10^{9}	3.7×10^{9}	3.5×10^{9}
26.7	(80)	9.6×10^{8}	4.5×10^{9}	1.7×10^{9}	4.0×10^{9}	2.3×10^{9}	3.5×10^{9}
32.2	(90)	5.5×10^{8}	2.3×10^{9}	1.0×10^{9}	2.5×10^{9}	1.4×10^{9}	2.6×10^{9}

TABLE 10 - ASPHALT STIFFNESS (P_a) CONTROL SECTION 9-6

(Stiffnesses Based on Air Voids Obtained by Laboratory Compacted Specimens)

TEMPERATURE		YOUNG (1970-73)		MIDLIFE (1974-77)		OLD (1978-82)	
		SURFACE	BASE	SURFACE	BASE	SURFACE	BASE
°C	(°F)						
-12.2	(10)	2.3×10^{10}	2.3×10^{10}	2.1×10^{10}	2.2×10^{10}	1.9×10^{10}	2.3×10^{10}
- 6.7	(20)	1.9×10^{10}	2.0×10^{10}	1.7×10^{10}	2.1×10^{10}	1.4×10^{10}	2.1×10^{10}
- 1.1	(30)	1.4×10^{10}	1.4×10^{10}	1.3×10^{10}	1.4×10^{10}	1.2×10^{10}	1.5×10^{10}
4.4	(40)	9.0×10^{9}	9.0×10^{9}	9.7×10^{9}	9.7×10^{9}	9.7×10^{9}	1.0×10^{10}
10.0	(50)	4.8×10^{9}	5.6×10^{9}	5.6×10^{9}	6.4×10^{9}	6.4×10^{9}	6.9×10^{9}
15.6	(60)	2.4×10^{9}	2.8×10^{9}	3.6×10^{9}	3.7×10^{9}	4.8×10^{9}	4.4×10^{9}
21.1	(70)	1.3×10^{9}	1.9×10^{9}	2.1×10^{9}	2.6×10^{9}	2.9×10^{9}	3.2×10^{9}
26.7	(80)	6.9×10^{8}	1.4×10^{9}	1.2×10^{9}	2.6×10^{9}	1.7×10^{9}	3.2×10^{9}
32.2	(90)	3.9×10^{8}	1.2×10^{9}	7.6×10^{8}	1.7×10^{9}	1.1×10^{9}	2.1×10^{9}
37.8	(100)	2.1×10^{8}	5.4×10^{8}	4.9×10^{8}	8.3×10^{8}	7.6×10^{8}	1.0×10^{9}

TABLE 11 - CONTROL SECTION 9.6 (6% AIR VOIDS)

TEMPERATURE		YOUNG (1970-73)		MIDLIFE (1974-77)		OLD (1978-82)	
		SURFACE	BASE	SURFACE	BASE	SURFACE	BASE
°C	(°F)						
-12.2	(10)	3.7×10^{10}	3.3×10^{10}	3.4×10^{10}	3.9×10^{10}	3.2×10^{10}	3.7×10^{10}
- 6.7	(20)	2.9×10^{10}	3.1×10^{10}	2.6×10^{10}	3.2×10^{10}	2.4×10^{10}	3.4×10^{10}
- 1.1	(30)	2.4×10^{10}	2.3×10^{10}	2.3×10^{10}	2.5×10^{10}	2.3×10^{10}	2.6×10^{10}
4.4	(40)	1.7×10^{10}	1.7×10^{10}	1.7×10^{10}	1.9×10^{10}	1.8×10^{10}	1.9×10^{10}
10.0	(50)	1.0×10^{10}	1.0×10^{10}	1.1×10^{10}	1.3×10^{10}	1.2×10^{10}	1.4×10^{10}
15.6	(60)	5.6×10^{9}	6.3×10^{9}	6.9×10^{9}	8.3×10^{9}	8.3×10^{9}	9.7×10^{10}
21.1	(70)	2.8×10^{9}	4.2×10^{9}	4.6×10^{9}	5.6×10^{9}	6.4×10^{9}	6.9×10^{9}
26.7	(80)	1.9×10^{9}	4.2×10^{9}	3.0×10^{9}	5.6×10^{9}	4.2×10^{9}	6.9×10^{9}
32.2	(90)	1.0×10^{9}	2.6×10^{9}	1.9×10^{9}	3.8×10^{9}	2.8×10^{9}	5.0×10^{9}
37.8	(100)	5.2×10^{8}	1.5×10^{9}	1.3×10^{9}	2.1×10^{9}	2.1×10^{9}	2.7×10^{9}

TABLE 12 - DEFLECTION DATA C.S. 9-2

DATE	UNCORRECTED MEAN PAVEMENT DEFLECTION	TEMP °C	TEMPERATURE CORRECTED MEAN mm	TEMPERATURE CORRECTED MEAN + 2
82/04/26	0.8287	14	0.774	1.164
05/04	1.2104	20	1.043	1.879
05/26	1.5522	26	1.288	2.192
06/08	0.8261	19	0.719	1.351
06/15	1.4383	34	1.187	2.111
06/22	1.1183	23	0.942	1.646
06/28	1.1113	22	0.942	1.512
07/19	1.5364	41	1.212	2.086
08/23	1.1270	30	0.916	1.464
10/12	0.6435	16	0.581	.885

TABLE 13 - DEFLECTION DATA C.S. 9-3

DATE	UNCORRECTED MEAN PAVEMENT DEFLECTION	TEMP °C	TEMPERATURE CORRECTED MEAN mm	TEMPERATURE CORRECTED MEAN + 2
82/04/27	0.4933	12	0.478	0.678
05/04	1.4924	25	1.224	2.052
05/18	0.9714	26	0.806	1.506
05/26	1.5571	36	1.259	2.047
06/08	0.9171	24	0.773	1.467
06/16	0.9657	22	0.819	1.483
06/22	1.0781	24	0.908	1.532
06/28	0.9114	24	0.765	1.499
07/19	0.8210	34	0.666	0.302
03/23	0.9730	30	0.798	1.362
10/12	0.5505	17	0.488	0.694

TABLE 14 - DEFLECTION DATA C.S. 9-4

DATE	UNCORRECTED MEAN PAVEMENT DEFLECTION	TEMP °C	TEMPERATURE CORRECTED MEAN mm	TEMPERATURE CORRECTED MEAN + 2
82/04/27	0.5156	13	0.490	0.710
05/05	0.8056	11	0.797	1.155
05/18	0.8311	22	0.704	1.088
05/26	1.6094	36	1.300	2.186
06/09	0.6867	20	0.591	0.811
06/16	1.0978	30	1.004	1.680
06/22	0.8944	21	0.764	1.040
06/29	0.7333	29	0.601	0.991
07/19	0.7033	34	0.571	0.839
10/12	0.5267	15	0.483	0.683

TABLE 15 - DEFLECTION DATA C.S. 9-5

DATE	UNCORRECTED MEAN PAVEMENT DEFLECTION	TEMP °C	TEMPERATURE CORRECTED MEAN mm	TEMPERATURE CORRECTED MEAN + 2
82/04/27	0.3644	14	0.340	0.482
05/05	0.5289	15	0.485	0.737
05/08	0.4444	12	0.374	0.574
05/26	1.0656	37	0.814	1.418
06/09	0.5289	25	0.440	0.660
06/16	0.6822	26	0.566	0.970
06/22	0.6267	27	0.518	0.800
06/29	0.5644	30	0.462	0.714
07/19	0.6978	30	0.572	0.816
10/12	0.3244	15	0.308	0.462

TABLE 16 - DEFLECTION DATA C.S. 9-6

DATE	UNCORRECTED MEAN PAVEMENT DEFLECTION	TEMP °C	TEMPERATURE CORRECTED MEAN mm	TEMPERATURE CORRECTED MEAN + 2
82/04/27	0.6057	14	0.566	0.746
05/05	0.6343	19	0.522	0.780
05/18	0.7600	23	0.641	1.109
05/26	1.6086	36	1.300	2.098
06/09	0.8375	27	0.691	1.061
06/16	1.1743	26	0.975	1.563
06/22	1.1343	28	0.934	1.400
06/29	0.9514	31	0.778	1.284
07/19	1.2057	29	0.991	1.641
08/23	0.8771	27	0.725	1.135
10/12	0.2171	10	0.220	0.300

TABLE 17

SUBGRADE STRENGTHS INFERRED FROM ELSYM-5
AND BENKLEMAN BEAM RESULTS FROM 1982

CONTROL SECTION	DATE	SUBGRADE MODULUS		MONTHLY AVERAGE	
		MPa	psi	MPa	psi
9-2	APR 27	41	6,000	41	6,000
	MAY 4	30	4,400		
	18	29	4,200	28	4,000
	26	23	3,400		
	JUN 8	46	6,600		
	16	30	4,400	37	5,300
	22	35	5,000		
	28	35	5,000		
	JUL 19	35	5,100	35	5,100
	AUG 23	39	5,700	39	5,700
	OCT 12	55	8,000	55	8,000
9-3	APR 27	97	14,000	97	14,000
	MAY 4	30	4,400		
	18	52	7,600	40	5,800
	26	37	5,400		
	JUN 8	55	8,000		
	16	52	7,600	52	7,550
	22	46	6,600		
	28	55	8,000		
	JUL 19	97	14,000	97	14,000
	AUG 23		10,000		10,000
	OCT 12	97	14,000	97	14,000
9-4	APR 27	97	14,000	97	14,000
	MAY 4	52	7,600		
	18	57	8,300	49	7,100
	26	37	5,300		
	JUN 8	76	11,000		
	16	52	7,600	67	9,750
	22	51	7,400		
	28	90	13,000		
	JUL 19	110	16,000	110	16,000
	AUG 23				
	OCT 12	97	14,000	97	14,000
9-5	APR 27	110	16,000	110	16,000
	MAY 4	69	10,000		
	18	110	16,000	76	11,000
	26	48	7,000		
	JUN 8	83	12,000		
	16	63	9,200	80	11,550
	22	76	11,000		
	28	97	14,000		
	JUL 19	76	11,000	76	11,000
	OCT 12	117	17,000	117	17,000
9-6	APR 27	57	8,200	57	8,200
	MAY 4	59	8,500		
	18	54	7,800	47	6,800
	26	28	4,000		
	JUN 8	54	7,800		
	16	34	5,000	43	6,300
	22	35	5,100		
	28	50	7,200		
	JUL 19	36	5,200	36	5,200
	AUG 23	50	7,200	50	7,200
	OCT 12	207	30,000	207	30,000

TABLE 18 - ESTIMATES OF SUBGRADE STRENGTH

C.S	DESIGN			APPARENT CBR			ELYSN-5		
	CBR	MODULUS		CBR	MODULUS			MODULUS	
		psi	MPa		psi	MPa		psi	MPa
9-2	Ave 5.2 1.33	7,800	54	9	13,500	93	Winter May June Rest of Yr	100,000 4,000 5,000 7,000	690 28 34 48
9-3	Ave 7.5 1.24	11,250	78	15	22,550	155	Winter May June Rest of Yr	100,000 6,000 10,000 14,000	690 41 69 97
9-4	Ave 7.7 1.25	11,550	80	17	25,550	155	Winter May June Rest of Yr	100,000 7,000 10,000 14,000	690 48 69 97
9-5	Ave 6	9,000	62	12	18,000	124	Winter May June Rest of Yr	100,000 11,000 12,000 16,000	690 76 83 110
9-6	Ave .5	7,500	52	12	18,000	124	Winter May June Rest of Yr	100,000 6,000 6,000 8,000	690 41 41 55

TABLE 19 - THICKNESS OF ASPHALT CONCRETE

C.S. 9-2	Surface	75 mm (3")
	Base	140 mm (5.5")
C.S. 9-3	Surface	75 mm (3")
	Base	115 mm (4.5")
C.S. 9-4	Surface	75 mm (3")
	Base	100 mm (4")
C.S. 9-5	Surface	75 mm (3")
	Base	115 mm (4.5")
C.S. 9-6	Surface	75 mm (3")
	Base	115 mm (4.5")

TABLE 20 - ESAL'S FOR MIXED COMMERCIAL TRAFFIC

CONTROL SECTION	1982	1992	1997
C.S. 9-2 Growth Factor 4.7%	163,000	388,000	546,000
C.S. 9-3 Growth Factor 4.0%	186,000	424,000	584,000
C.S. 9-4 Growth Factor 3.1%	144,000	310,000	415,000
C.S. 9-5 Growth Factor 3.1%	129,000	325,000	438,000
C.S. 9-6 Growth Factor 2.9%	117,000	294,000	394,000

TABLE 21 - NUMBER OF POTASH TRIPS PER YEAR

	ANNUAL NO. OF TRIPS
October 1, 1968 - September 30, 1969	22,700
October 1, 1969 - September 30, 1970	22,700
October 1, 1970 - September 30, 1971	22,700
October 1, 1971 - September 30, 1972	22,700
October 1, 1972 - September 30, 1973	22,700
October 1, 1973 - September 30, 1974	12,120
October 1, 1974 - September 30, 1975	12,120
October 1, 1975 - September 30, 1976	12,120
October 1, 1976 - September 30, 1977	12,120
October 1, 1977 - September 30, 1978	13,767
October 1, 1978 - September 30, 1979	14,757
October 1, 1979 - September 30, 1980	22,070
October 1, 1980 - September 30, 1981	22,181
October 1, 1981 - September 30, 1982	17,372

TABLE 22 - CORRECTION FACTORS FOR FATIGUE CRACKING BASED ON ASPHALT CONCRETE MIX CHARACTERISTICS

CONTROL SECTION	CORRECTION FACTOR FOR FATIGUE CRACKING ACCORDING TO MIX CHARACTERISTICS
9-2 Laboratory Air Voids	2.046
9-3 Laboratory Air Voids 6% Air Voids	2.250 .982
9-6 Laboratory Air Voids 6% Air Voids	1.293 .781

TABLE 23

RUTTING CRITERIA USED IN FATIGUE ANALYSIS

ALLOWABLE LOAD REPETITIONS	COMPRESSIVE SUBGRADE STRAIN X 10^{-6}		
	Original Shell 19 mm (3/4") rut	Revised Shell 19 mm (3/4") rut	Chevron 9.5 mm (3/8") rut
1×10^5	1050	1700	800
1×10^6	650	1030	485
1×10^7	420	650	285
1×10^8	260	400	170

TABLE 24 - PERCENTAGE OF ADT TRAFFIC FOR EACH TEMPERATURE INTERVAL

	0	10	20	30	40	50	60	70	80	90	100
JAN.	39.4	60.6									
FEB.	23.9	23.8	52.3								
MAR.		24.7	18.5	35.7	21.1						
APR.				23.9	16.6	20.4	38.9				
MAY					12.8	26.6	10.8	28.7	21.1		
JUNE						12.8	21.1	10.3	20.7	29.1	
JULY						5.8	10.1	10.0	16.3	17.6	32.2
AUG.						12.4	11.5	16.8	16.4	40.9	
SEPT.					12.4	21.5	16.3	49.8			
OCT.				5.0	33.6	28.4	33.2				
NOV.		12.2	21.7	66.1							
DEC.	12.8	37.4	49.8								

TABLE 25 - PERCENTAGE OF POTASH TRAFFIC FOR EACH TEMPERATURE INTERVAL

	0	10	20	30	40	50	60	70	80	90	100
JAN.	54.2	45.8									
FEB.	37.5	18.75	43.75								
MAR.		35.4	20.8	25.0	18.8						
APR.				35.4	16.7	18.75	29.15				
MAY					29.1	16.7	16.7	18.75	18.75		
JUNE						29.1	16.7	14.6	18.75	20.85	
JULY						20.8	16.7	12.5	12.5	16.6	22.9
AUG.						25.0	16.7	12.5	14.6	31.2	
SEPT.					25.0	20.8	18.75	35.45			
OCT.				20.8	29.2	27.1	22.9				
NOV.		20.85	27.05	52.1							
DEC.	29.2	35.4	35.4								

TABLE 26 - ESTIMATE OF FATIGUE DAMAGE
FOR HISTORIC TRAFFIC

CONTROL SECTION AND STIFFNESS CHARACTERISTICS	CRACKING	RUTTING		
		REVISED SHELL 3/4"	ORIGINAL SHELL 3/4"	CHEVRON 3/8"
9-2				
Smix (Laboratory Air Voids)	.774	.788	8.0725	31.497
9-3				
6% Air Voids Smix	.399	.069	.712	3.451
Smix (Laboratory Air Content)	.876	1.298	15.586	50.031
9-6				
6% Air Voids Smix	.151	.316	2.237	1.269
Smix (Laboratory Air Content)	.831	.964	5.659	43.582

TABLE 27 - COMPARISON DESIGN METHODS BASED ON
HISTORICAL TRAFFIC

CONTROL SECTION	EAL's TO DATE	SUBGRADE C.B.R.	ACTUAL THICKNESS	ASPHALT INSTITUTE		AASHTO P_t - 2.5		SASK DESIGN PROCEDURE		JRA (T_A)	
				DESIGN	DIFFERENCE FROM ACTUAL	DESIGN	DIFFERENCE FROM ACTUAL	DESIGN	DIFFERENCE FROM ACTUAL	DESIGN	DIFFERENCE FROM ACTUAL
9-2	1.06×10^6	6	215 mm	222 mm	+ 7 mm	195 mm	- 20 mm	224 mm	+ 9 mm	279 mm	+ 64 mm
9-3	1.05×10^6	7	190 mm	210 mm	+ 20 mm	185 mm	- 5 mm	224 mm	+ 34 mm	266 mm	+ 76 mm
9-4	1.01×10^6	8	178 mm	200 mm	+ 22 mm	173 mm	- 5 mm	200 mm	+ 22 mm	254 mm	+ 86 mm
9-5	9.95×10^5	8	190 mm	200 mm	+ 10 mm	173 mm	- 17 mm	208 mm	+ 18 mm	253 mm	+ 63 mm
9-6	9.83×10^5	6	190 mm	215 mm	+ 15 mm	185 mm	- 5 mm	218 mm	+ 28 mm	276 mm	+ 86 mm

The Rationale and Benefits for Revision of B.C. Graranular Base Specifications

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ABSTRACT

The majority of highways in British Columbia are built with base and subbase materials which are dense and well graded. When dry these gradations are stable. However these gradations have a number of serious disadvantages. The foremost concerns are that they can be frost susceptible and can lose strength when wet or saturated. The permeability of highway construction aggregates is a key variable which can be used to evaluate the extent of frost susceptibility or potential strength loss. In order to increase the lifetimes of B.C. pavements the permeability of aggregates must be sufficient to meet the needs of vertical or, where required, lateral drainage. In order to achieve this objective, modifications to the Highway specifications for well graded B.C. aggregates are proposed. The resulting specifications are designed to achieve minimum permeability requirements for vertical drainage. Also optimum stability is ensured. Specifications for intermediate and open graded aggregate are also presented. These latter gradations are suitable where lateral drainage is required.

These improvements, if implemented, will guarantee that rock or gravel aggregate sources can be efficiently utilized and that most aggregate products will be drainable. Significant road life extensions are to be expected from the use of these proposed new and improved specifications.

THE RATIONALE AND BENEFITS FOR REVISION OF B.C. GRANULAR BASE SPECIFICATIONS

by

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and
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INTRODUCTION

For almost half a century, dense, well graded aggregate has been a primary construction material for base and subbase components of flexible pavement structures. When dry it is well suited to placement using modern construction equipment. In this condition it is stable and with sufficient thickness can easily support heavy and frequent traffic loads. Unfortunately, dense, well graded aggregate has a number of serious disadvantages all related to water retention characteristics and frost susceptibility.

In British Columbia, wherever highways have poor drainage, are saturated, or are subject to freeze thaw processes, roads become weak and pavement lifetimes may be only 10 years or less. In response to this chronic situation new specifications are herein proposed for use by the B.C. Ministry of Highways.

MAXIMUM DENSITY WELL GRADED AGGREGATE

During the last four decades the two most significant factors defining the stability of base and subbase materials were considered to be gradation and density. When this concept was developed it was well known

that, at low moisture contents, high mechanical stability could be achieved by utilizing dense aggregate gradations. Furthermore it was judged that static tests such as CBR tests were sufficient to measure stability under saturated conditions. Partly as a result of these concepts and beliefs most highway construction aggregates now are dense, well graded gradations. For a "maximum density" gradation the following equation conveniently defines the proportions passing each sieve size (1).

$$P = 100 \left(\frac{d}{D}\right)^{n} \qquad \text{----------(1}$$

d = sieve size (mm)

P = percent finer by weight than the specified sieve

D = maximum aggregate size

n = gradation index (0.45 for maximum density)

A gradation which satisfies the above equation will possess minimum voids and therefore maximum grain to grain content, thereby obtaining high stability. A plot of the above equation for any value of "D", with the horizontal scale equal to sieve size raised to the power 0.45 and the vertical scale representing percent passing, will produce a straight line passing through zero. A plot of other gradations on such a chart provides an estimate of the proximity of any gradation to a maximum density gradation.

For well graded aggregate, under dry conditions, maximum stability is achieved at maximum density, however when well graded aggregate is wet or saturated, maximum stability under static loads is achieved at lower values of fines than those required for maximum density (2).

Under rapid cyclic loading, stability in a high density, low voids, material is dependent upon pore pressure, and moisture content. The effect of pore pressure upon shear strength in an aggregate is expressed by the equation:

$$\tau = c + (\sigma - \mu) \tan \phi$$

c = cohesion

μ = pore water pressure

σ = total normal stress

$(\sigma - \mu)$ = effective stress

τ = shear strength

ϕ = angle of internal friction

The above equation indicates that as pore pressure in the base and subbase increases, shear strength will decrease. The rapid cyclic traffic loading of wet, greater than 85% saturated, low permeability base can produce the high pore pressures (2) which result in low effective stresses and reduced shear strengths.

Repeated load tests (Figure 1) on gravel and crushed stone base, used in the AASHO road test, clearly illustrate that base course materials with more than 80% saturation undergo much greater deflection under repeated loading than do base courses with less than 80% saturation (4).

Pore pressure development in dense, well graded aggregate is not the sole reason for water-related strength loss in pavement aggregate. Free water retained in the aggregate voids can reduce capillary forces which can strengthen aggregate. Free water also reduces intergranular friction by reducing the effective mass of submerged materials.

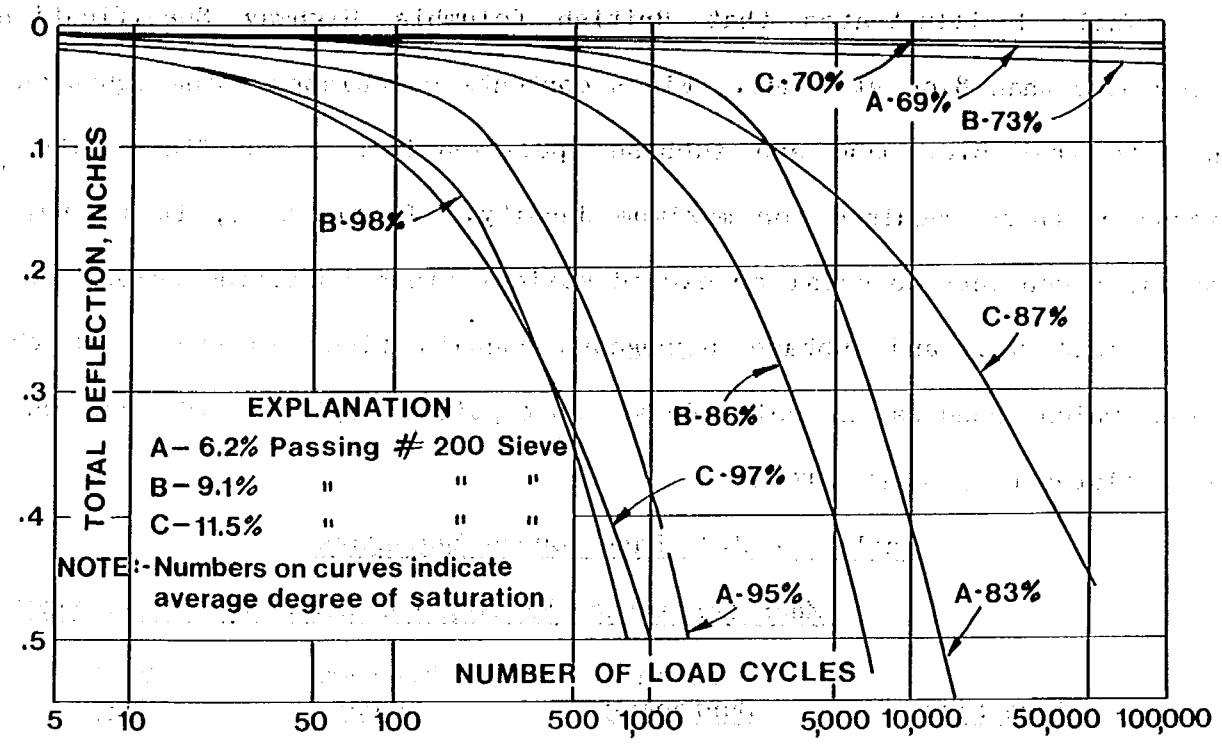


Figure 1 Deflection history of wet aggregate samples(4)

In addition to previously described strength-loss characteristics a dense well graded base or subbase if wet, is prone to the following phenomena:

- degradation of base course.
- freeze thaw disruption of the pavement structure.
- erosion of base.
- pumping of water through the base and subbase.

Dense, well graded base is commonly used throughout North America and Europe. Moreover, many specifications allow fines contents as high or greater than 10% (5). In some areas high fines aggregates have been well suited to the traffic conditions, material cost restrictions and the physiography. For example, in Manitoba and Saskatchewan high fines (to 20%) aggregates have been used successfully. In those Canadian provinces summers are hot, winters are cold and freeze thaw cycles are infrequent. Therefore, thaw weakening and frost heaves are not as serious a problem as in more temperate climates with frequent freeze thaw cycles.

Table 1 illustrates that British Columbia Highway Specifications allow more than 8 or 9% fines. Fines contents calculated using equation 1 indicate that B.C. base and subbase specifications allow fines well in excess of those required for maximum density. In practise, it is common for fines contents to equal or exceed maximum fines specified below. As a result most base and subbase aggregates contain fines in excess of that value which renders a soil frost susceptible (3% at .02 mm which extrapolates to 5.5% at .075 mm).

Table 1 - B.C. Base Course Aggregate

Mean Percentage Passing for Each Type of Aggregate

Sieve Size	19 mm Surfacing	37.5 mm Base	75 mm Base
75 mm			100
37.5 mm		100	60 - 100
19.0 mm	100	60 - 100	40 - 80
9.5 mm	60 - 95	40 - 80	30 - 60
4.75 mm	40 - 70	30 - 60	20 - 45
2.36 mm	30 - 60	20 - 45	15 - 35
1.18 mm	20 - 45	15 - 35	10 - 25
0.300 mm	8 - 20	6 - 18	4 - 16
0.075 mm	2 - 9 (8)[1]	2 - 9 (6)[1]	2 - 9 (4)[1]

Select Granular Subbase

Sieve Size	Percent Passing
75 mm	100
0.150 mm	0 - 16
0.075 mm	0 - 8 (4) [1]

[1] Fines content for maximum density

PERMEABILITY OF B.C. BASE AND SUBBASE AGGREGATE

Eleven crushed aggregate and fifteen pit run samples from B.C. were tested in a preliminary study to evaluate permeabilities of dense graded aggregate. All samples were compacted to maximum Proctor density. Of the eleven crushed samples tested, six met Ministry mean base course

specifications with respect to fines content (2 - 9%) whereas one sample was too low (1.6%) and three samples were too high (10.3%; 10.9%; 11.9%). There was a broad range of permeabilities from 1.9×10^{-1} cm/sec to 1.7×10^{-6} cm/sec. Because permeability is a function of factors such as density, gradation, angularity and mineralogy there was no clear trend observed between grain size and permeability (Figure 2). However, generally the highest fines aggregates had the lowest permeabilities, and the lowest fines aggregates had the highest permeability.

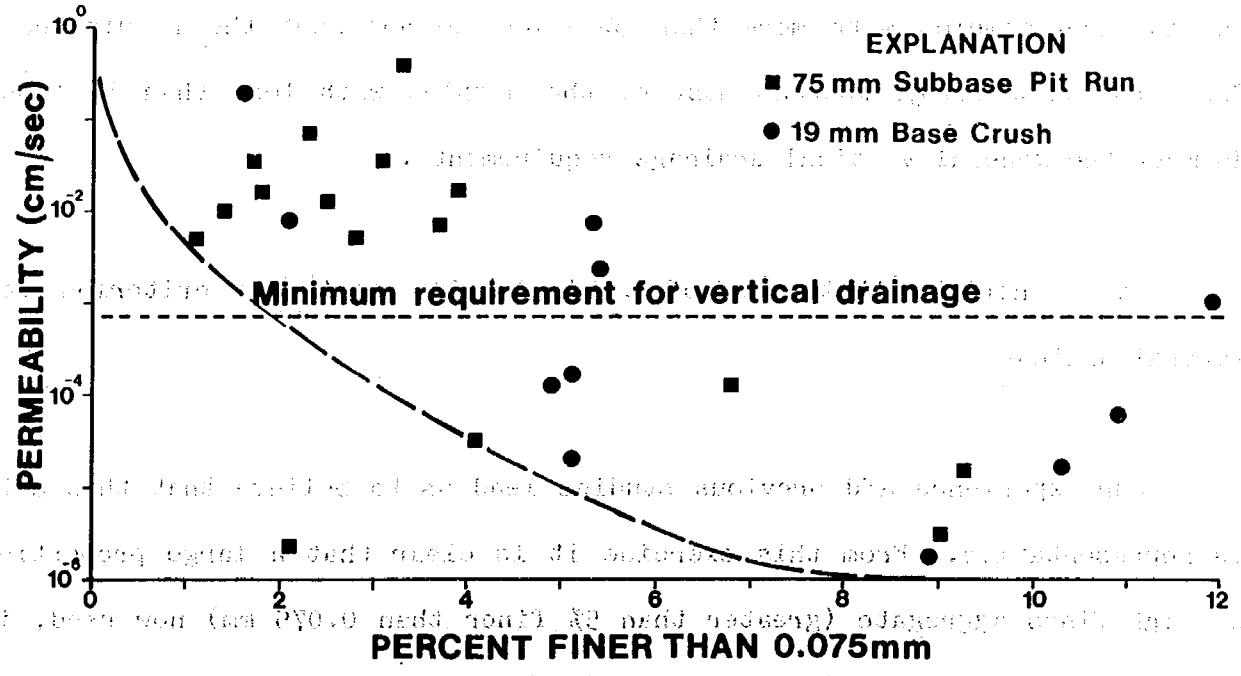


Figure 2: Permeability values corresponding to percent finer than 0.075 mm for 19 mm crush base and 75 mm subbase.

Of the fifteen pit run samples tested for permeability, fourteen met Ministry subbase specifications and one contained one percent too much material finer than the 0.075 mm size (9%). Ministry specifications for subbase allow 0 to 8% fines. As for the crushed samples, only a general correlation was observed between fines content and permeability (Figure 2). Recent studies (6) of pavement subsurface drainage systems indicate that aggregates having permeabilities greater than 7×10^{-4} cm/sec ($10^{-3.15}$ cm/sec) may be suitable for vertical drainage. Aggregates which have permeabilities greater than 0.35 cm/sec may be suitable for lateral drainage.

Using these criteria it is clear that many of the crushed aggregate and pit run samples with more than 5% fines do not meet the requirements for vertical drainage whereas most of the samples with less than 5% fines do meet the general vertical drainage requirements.

It is notable that none of these samples meet the criterion for lateral drainage.

Our experience and previous studies lead us to believe that this data is representative. From this exercise it is clear that a large proportion of high fines aggregate (greater than 5% finer than 0.075 mm) now used, is unsuitable for vertical and lateral drainage and is clearly frost susceptible. We conclude that our present specifications are generally unsuitable for much of the environment within British Columbia and are therefore in need of revision.

B.C. EXPERIENCE WITH DENSE GRADED AGGREGATE

The Province of British Columbia encompasses an area of 948 569 km^2 which contain diverse topography varying from mountain flanked valleys to river dissected plains. Semi-arid environments are found in zones such as the South Columbia River Valley, the South Thompson River Valley, the Okanagan Valley and the Central Interior (rainfall = 24 to 40 cm per year). Areas of high rainfall or precipitation occur along the Coast Range and along the Rocky Mountains (150 to 500 cm per year). Physiographic and precipitation zones in British Columbia are illustrated in Figures 3 and 4.

Seasonal temperature ranges vary considerably throughout the province. For example, southern Vancouver Island has winter temperatures which rarely go below freezing levels. In northern British Columbia, winter temperatures may drop to -50°C. In the southwest interior, summer temperatures may exceed 40°C.

Because of the great variation in topography, geology and climate throughout British Columbia, a great number of soil types have developed. For example, widespread continuous layers of glaciolacustrine clays, produced as a result of continental glaciation, occur in northeast British Columbia. Non-plastic glaciolacustrine silts which are prone to piping are found in semi-arid valleys such as the Okanagan Valley, the South Thompson River Valley and the South Columbia River Valley. Variable deposits of glacial till are found throughout the complex British Columbia alpine terrain, mantled or intermixed with deltaic, colluvial, aeolian or wetland deposits.

Figure 3: Physiographic regions in British Columbia.

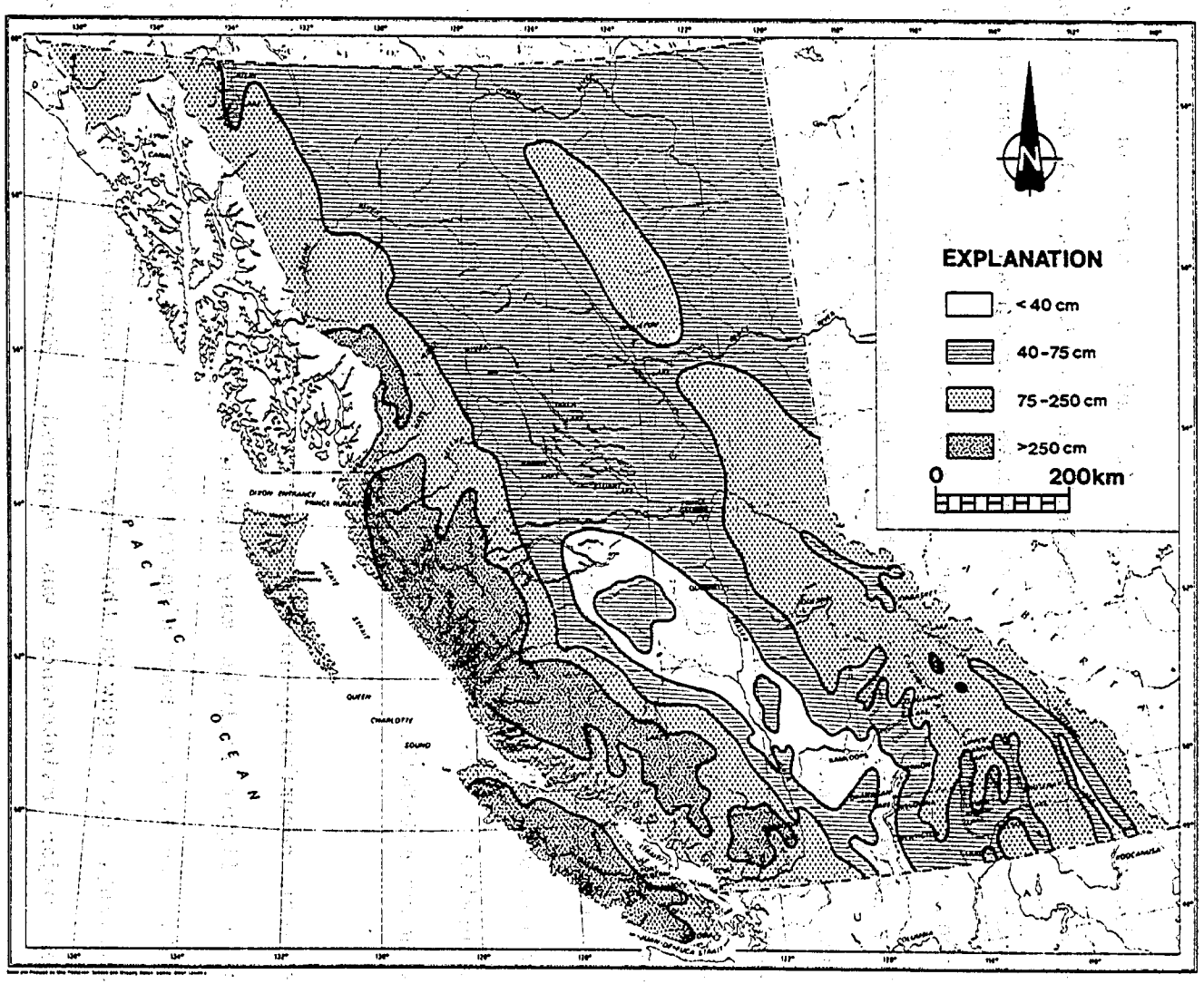


Figure 4: Mean annual precipitation, British Columbia.

The extreme subgrade and physiographic diversity described implies a great range in surface and groundwater drainage requirements and the need for flexible practice with respect to drainage and base course design.

The design process in British Columbia is based on strength characteristics of prototypes or "control sections". Seasonal deflection-rebound measurements are taken at points throughout the prototypes. Because the control sections are constructed of well graded, low permeability aggregate, moisture contents in the granular components may show variations dependent upon seasonal precipitation, temperature and the availability of water. As discussed previously, base course strengths and therefore resilient moduli are highly dependent upon moisture contents. In order to compensate for the variation in strength parameters, deflection-rebound measurements are always corrected to the weakest time recorded. Invariably the weakest strengths occur when the base, subbase or subgrade become saturated. This usually coincides with spring thaw or periods following heavy rainfall.

The mean resurfacing interval for highways in B.C. is 15.9 years (7). But this figure includes many highways in semi-arid environments with long pavement lives, also some highways that have had seal coats before hot mix resurfacing, also highways that have been reconstructed or resurfaced well after major maintenance was required. Examination of the data indicates a wide spread of pavement lifetimes. The statistics reveal that many pavements have lifetimes of 10 to 15 years.

For example, Highway 16 has portions which are seriously affected by drainage problems. One such portion is 10 km to 70 km northwest of McBride (west central B.C.) in the Rocky Mountain Trench and is about 10 years in age.

Figure 5 shows a section of this road where subsurface water has risen to the surface of the road and where weeping and pumping have occurred along wheelpaths, longitudinal cracks and transverse cracks. In this situation severe rutting and alligator cracking have occurred in the wheelpaths. In this pavement structure the base course contains 11 to 13% fines.

Another extreme example of the drastic effect that high fines in well graded aggregate can have upon pavement performance is indicated by Figure 6 which illustrates weeping occurring along a section of Highway 3A, one year after construction. This section of highway was built over porous rocky debris from the Hope Slide in southwest B.C.. However the dense graded aggregate forming the road structure was able to accumulate water as ice during the fall and winter and thereby to saturate the base during spring thaw.

Throughout B.C. there are many road sections which intersect aquifers where the pavement structure acts as a dam and itself becomes saturated with water. Figure 7 illustrates weak, saturated base course materials in such a situation and the resulting pavement deterioration. This site is on the Lougheed Highway (Highway 7) near Vancouver.

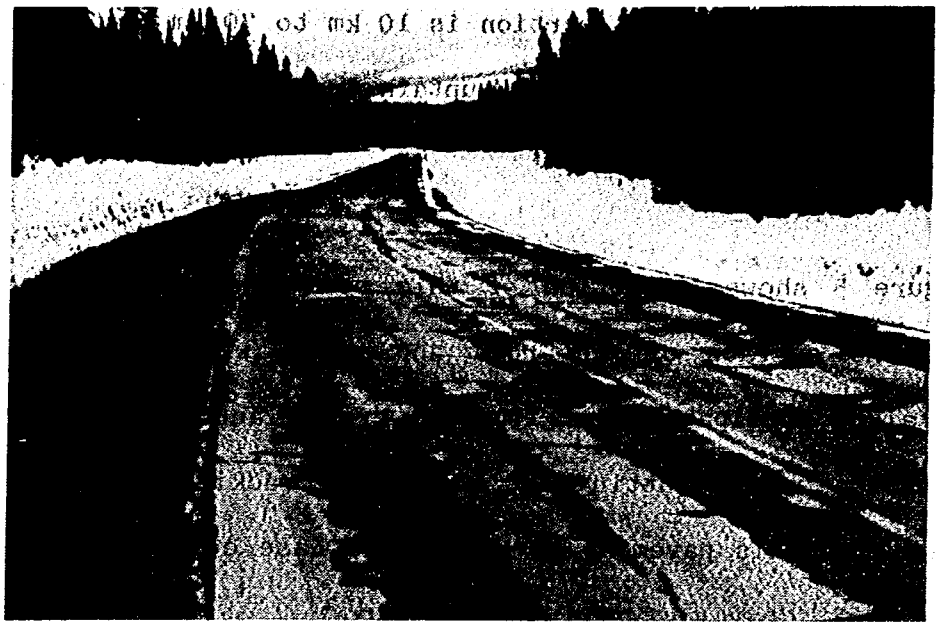


Figure 5: Meltwater brought to the road surface of Highway 16 (Yellowhead Route) by the process of pumping.

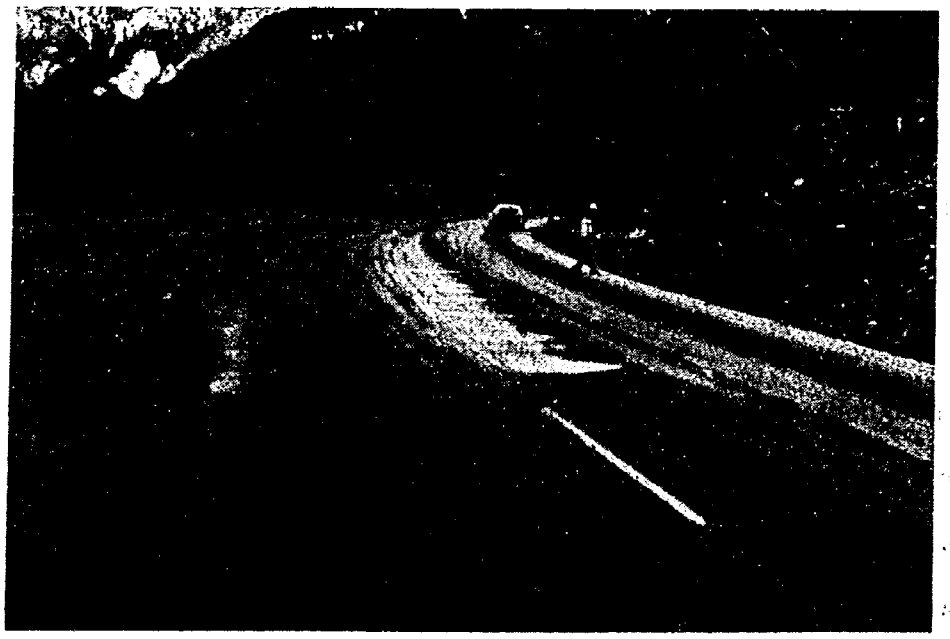


Figure 6: Water flowing onto the surface of Highway 3A from a joint in the asphalt cement concrete.

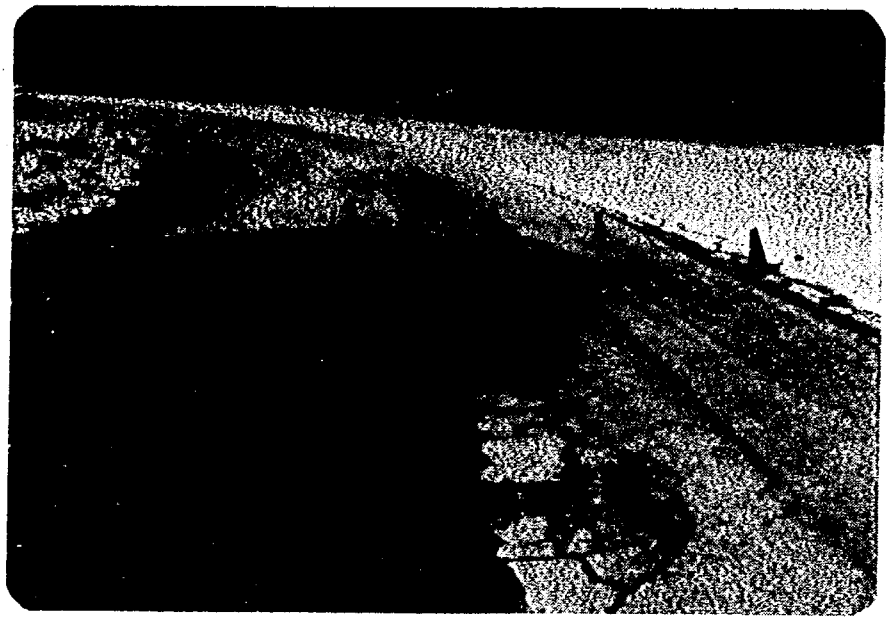


Figure 7: Water seeping from the cracked pavement surface, Highway 7 (Lougheed Highway).

Frost susceptible subbase can produce frost heaves which can easily destroy rigid or flexible pavements. One such pavement is a portion of the John Hart Highway (Highway 97) near Azouzetta Lake in northeast B.C. where extreme transverse and longitudinal cracking have been induced by frost heave of pavement slabs (age less than 5 years) (Figure 8). This latter example of frost heaving in a cement treated base is a prime example of the costly non-traffic related problems which can be generated by high fines aggregate in a subbase.

The previous examples all have one common feature, they are from sites that are poorly drained. Base and subbase aggregates in these structures are all dense, well graded granular materials with excess fines.

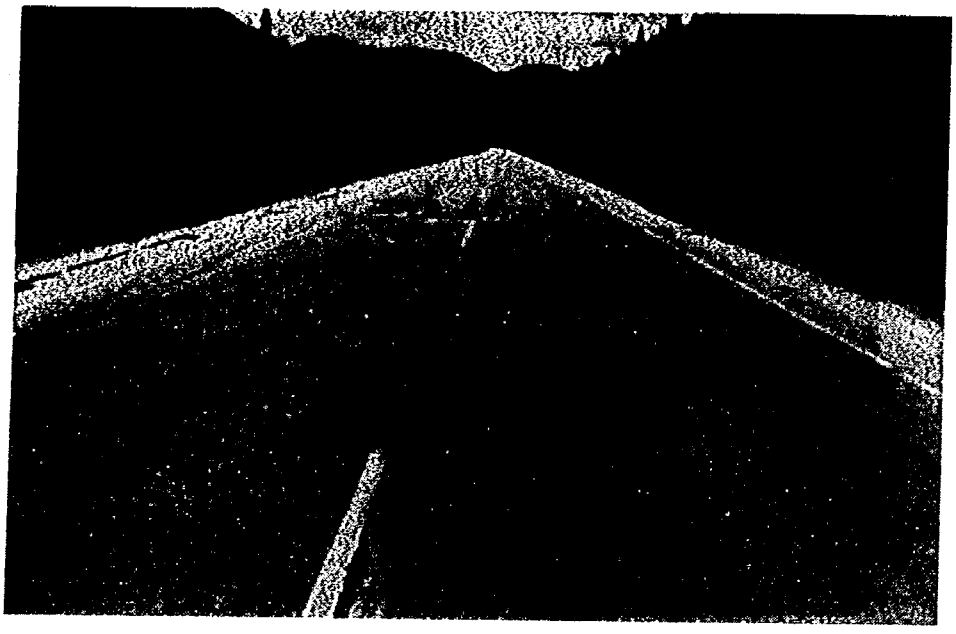


Figure 8: Meltwater which has flowed through transverse and longitudinal cracks to the highway surface, Highway 97.

DRAINAGE DESIGN, STABILITY AND AGGREGATE SPECIFICATIONS

To define the permeability required for suitable drainage, the following information must be known:

1. Highway geometry
2. Details regarding surface drainage
3. Climate (rainfall and seasonal temperature)
4. Physiography
5. Subsurface water distribution
6. Soil properties of subgrade and surrounding soils

From this knowledge the engineer may estimate drainage times and the amount of water entering and exiting from the pavement structure and the location of the free water surface.

Free water may be removed from the pavement structure by two types of drainage or a combination of these drainage modes.

One drainage mode is that of vertical drainage, here the free water drains vertically into the subgrade. Because the distance travelled from the pavement surface to the subgrade is relatively short, the minimum permeability required for such aggregate is low, of the order of 7×10^{-4} cm/sec (6).

The other drainage mode is that of lateral drainage. In this mode free water is drained to the edge of the pavement structure. Experience indicates that minimum permeabilities of the order of 0.35 cm/sec (6) are necessary for this type of drainage.

In addition to being permeable, base and subbase aggregates should be non-frost susceptible and stable. Specifications are herein proposed for three types of gradation which meet those requirements. The gradations are classified as well-graded, intermediate and open graded.

Well graded aggregates are those which are close to maximum density but which have been designed to have much greater permeability than well graded maximum density gradations. Open graded aggregates are uniformly graded aggregates that contain less than 10% finer than 2.36 mm. Intermediate gradations have grain size distributions which are "intermediate" between open and dense graded and which contain more than 10% finer than 2.36 mm. To provide flexibility in design procedures there is some overlap between gradations.

Permeability and reduction in frost susceptibility has been achieved by limiting fines in dense graded aggregates to less than 5% and by designing gradations which deviate from maximum density in a manner which increases voids.

The gradations for the above aggregates are listed in the appendix. Figures 9 (a, b, and c) illustrate the distribution of well graded aggregate and how they depart from well graded, maximum density gradations. Figures 10 (a, b, and c) to 11 (a, b, and c) illustrate the distribution of intermediate and open graded aggregate gradations.

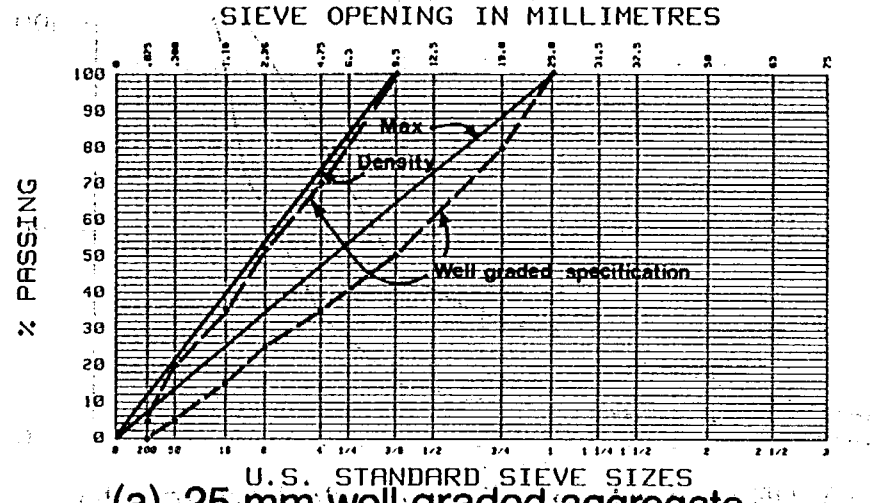
Gradations selected from within all of these specification curves must have slopes intermediate between the two bounding specification curves.

Laboratory permeability tests have been conducted on a number of representative samples of well graded, intermediate and open graded aggregate samples. Table 2 illustrates the permeability ranges to be expected from typical samples from each gradation.

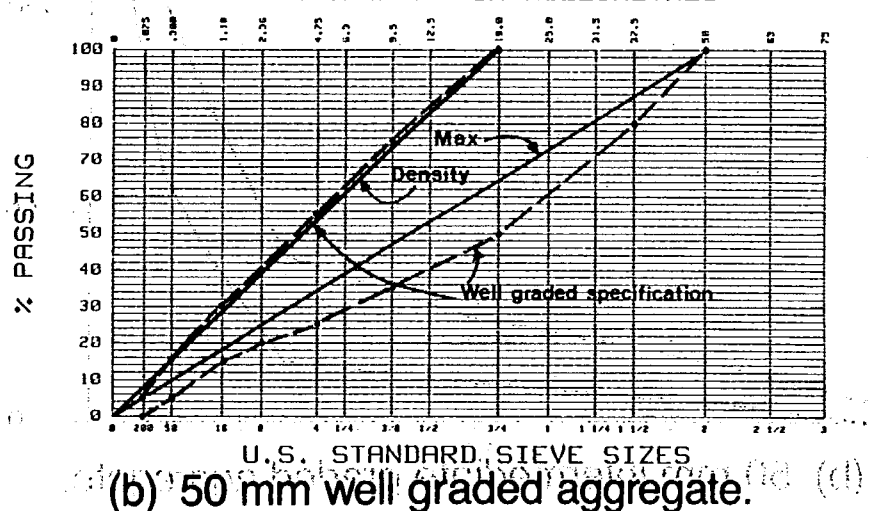
Table 2 - Typical Permeability Ranges

Type	cm/sec
Well graded	10^{-5} to 10^{-1}
Intermediate	0.3 to 10
Open graded	10 to 70

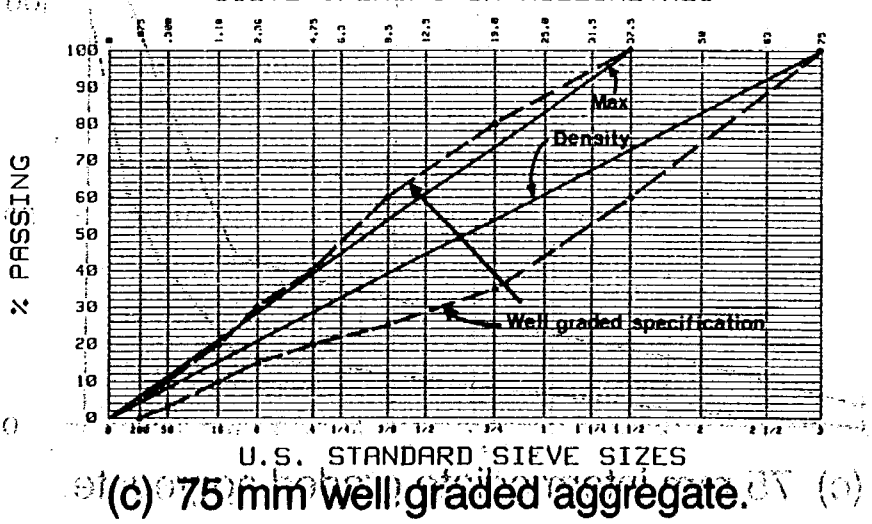
As can be seen from the above information, some well graded samples may not have permeabilities suitable for drainage requirements. Therefore it is proposed that representative samples of all aggregate to be used for base and subbase construction be subjected to permeability tests. This will also enable the engineer to approve samples which do not have the specification gradation yet have suitable permeabilities. Although this option may not optimize stability considerations it enables flexibility in design.



(a) 25 mm well graded aggregate.

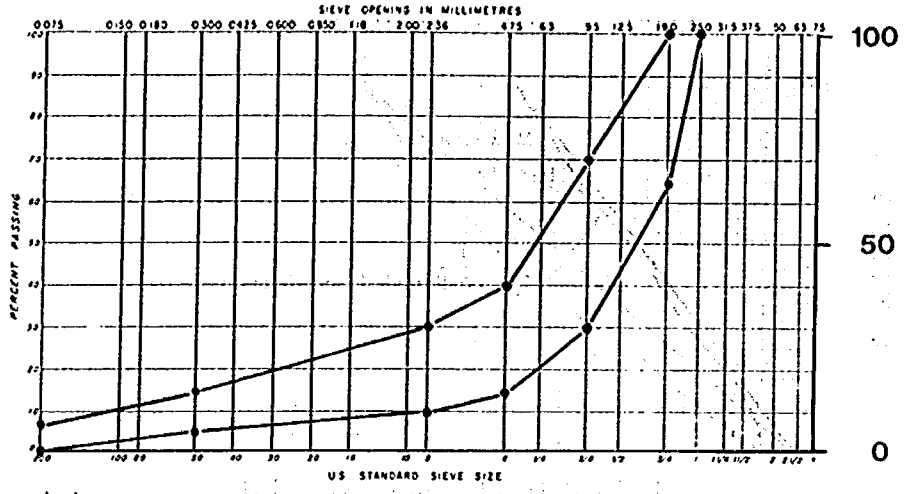


(b) 50 mm well graded aggregate.

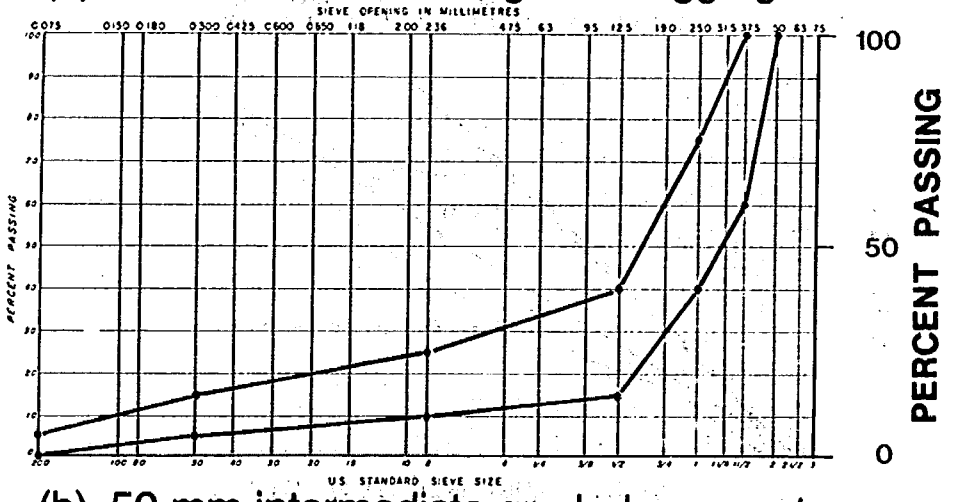


(c) 75 mm well graded aggregate.

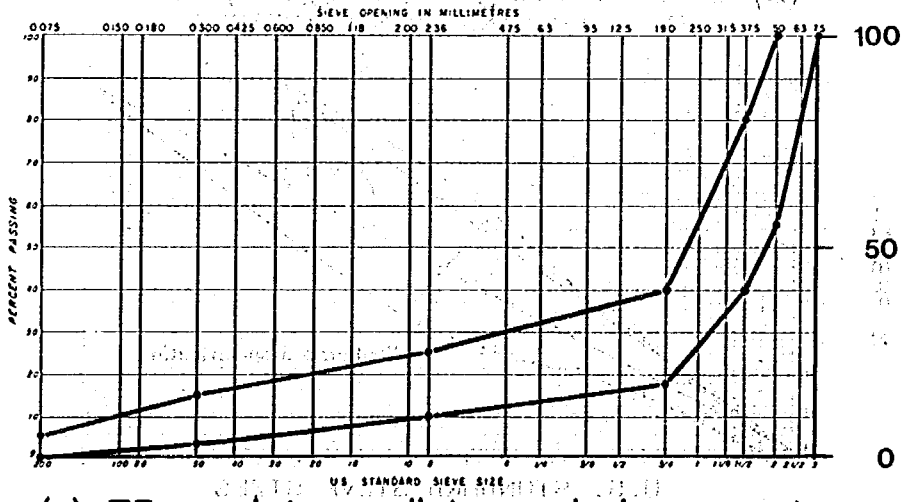
Figure 9: Aggregate gradation charts for sieve sizes raised to the 0.45 power.



(a) 25 mm intermediate graded aggregate.

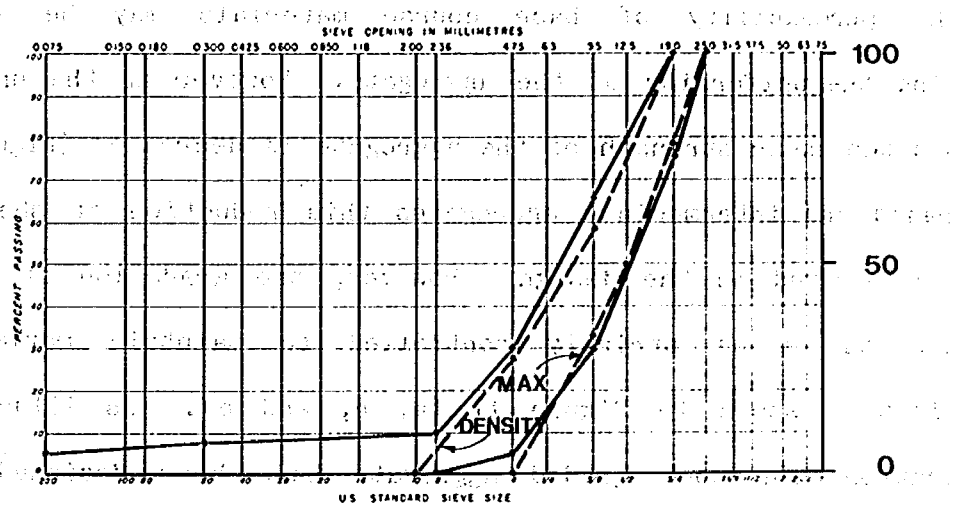


(b) 50 mm intermediate graded aggregate.

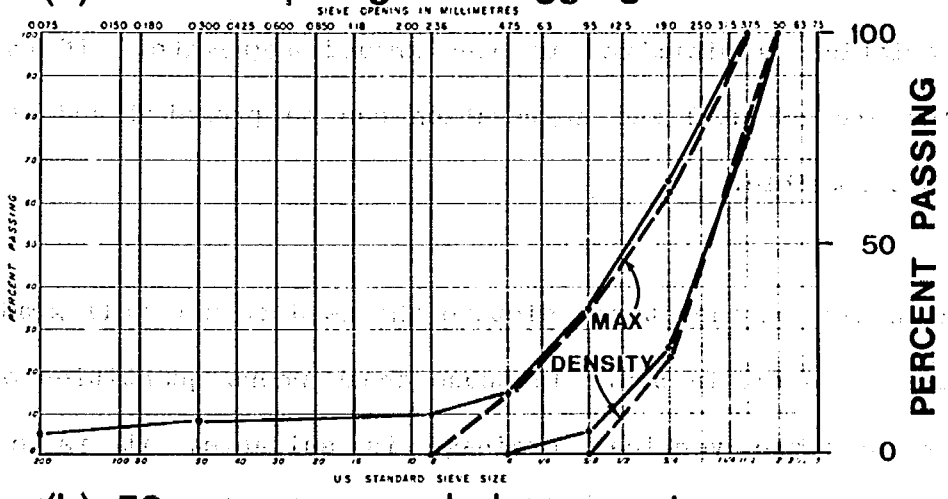


(c) 75 mm intermediate graded aggregate.

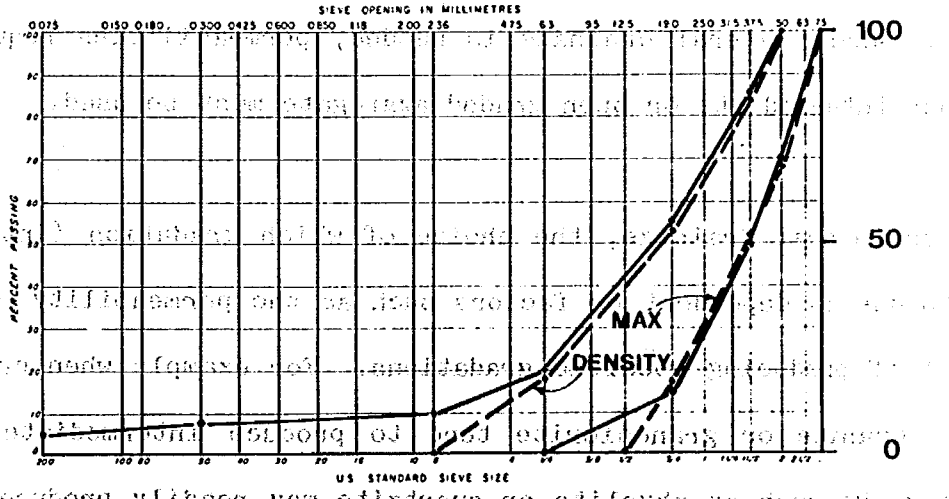
Figure 10: Aggregate gradation charts (logarithmic normal plot).



(a) 25 mm open graded aggregate.



(b) 50 mm open graded aggregate.



(c) 75 mm open graded aggregate.

Figure 11: Aggregate gradation charts (logarithmic normal plot).

The permeability of base course materials may be achieved by increasing the uniformity of the aggregate. However as the uniformity is increased the shear strength of the aggregate is decreased (Figure 12). In well graded and intermediate aggregates this reduction of shear strength can be considered in the design. However, the gradation of open graded aggregate should be strictly controlled to maintain maximum density proportions as shown in Figure 11 (a, b, and c). As illustrated, the specifications for open graded aggregate have been designed with this concept as a guideline. As a result, loss of shear strength from gradation characteristics is minimized in open graded aggregate. If the gradations specified are not utilized segregation can be promoted and shear strength losses are possible.

Economic considerations dictate the need to use well graded materials which are so common in B.C. In those areas where quantities of free water are low or where vertical drainage is suitable, it is commonly most economic to use well graded material of sufficient permeability for base or subbase. Where lateral drainage is needed, permeabilities required are so high that intermediate or open graded aggregate must be used.

For lateral drainage, the choice of which gradation (intermediate or open) to use is dependent on factors such as the permeability required, and the cost of producing specific gradations. For example when crushed, rocks such as granite or granodiorite tend to produce intermediate gradations, whereas rocks such as rhyolite or quartzite may readily produce open graded aggregate without costly processing. Gravels of glaciofluvial origin will have gradation curves, after crushing, dependent upon a number of factors including rock type and pre-crush gradation.

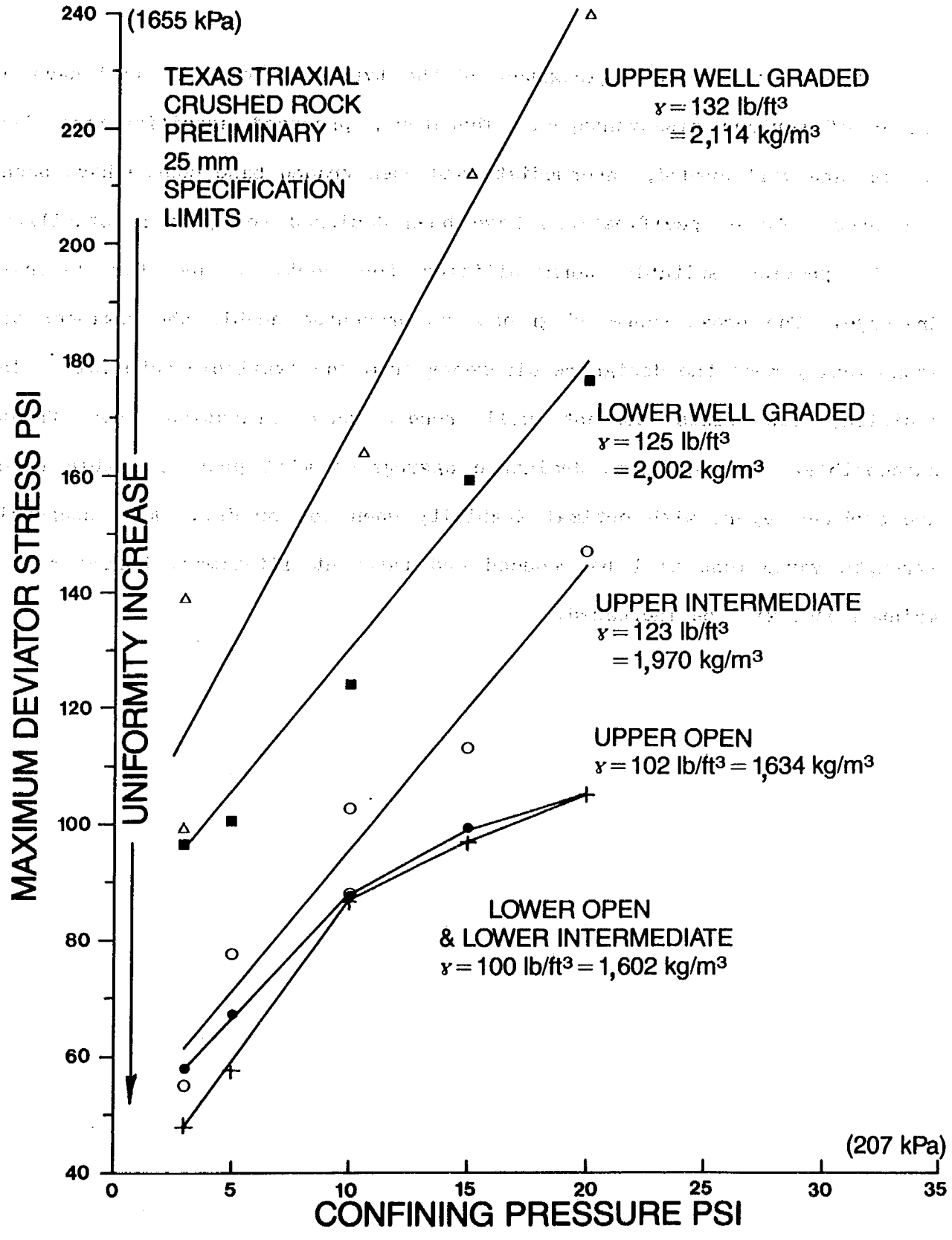


Figure 12: Shear strength reduction with increased uniformity.

CONCLUSIONS

Dense, well graded aggregates of the type now commonly used have a number of serious disadvantages. Therefore, proposed specifications for subbase and well graded, intermediate and open graded base course have been presented. These specifications have been designed to optimize stability and to provide suitable permeabilities for vertical and for lateral drainage. The broad range of gradations presented enable the engineer to economically meet the design requirements from the available materials. In addition, low fines content will render such gradations non frost susceptible. Use of these drainable aggregates will provide stable base and subbase layers with optimal stability when wet or dry. Also seasonal strength variations will be reduced and pavement lifetimes, before major maintenance, will be increased.

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APPENDIX

Aggregate Specifications

Well Graded Base

Sieve Size (mm)	Percentage Passing for Nominal Maximum Size		
	25 mm	50 mm	75 mm
75			100
50		100	-
37.5		80-100	60-100
25	100	-	-
19	80-100	50-100	35-80
9.5	50-100	35-75	25-60
4.75	35-70	25-55	20-40
2.36	25-50	20-40	15-30
1.18	15-35	15-30	10-20
0.300	5-20	5-15	3-10
0.075	0-5	0-5	0-5

Intermediate Graded Base

75	-	-	100
50	-	100	55-100
37.5	-	60-100	40-80
25	100	40-75	-
19	65-100	-	17-40
12.5	-	15-40	-
9.5	30-70	-	-
4.75	15-40	-	-
2.36	10-30	10-25	10-25
0.300	5-15	5-15	4-15
0.075	0-5	0-5	0-5

Open Graded Base

75	-	-	100
50	-	100	70-100
37.5	-	75-100	50-85
25	100	-	-
19	75-100	35-65	15-55
9.5	30-65	5-35	-
6.3	-	-	0-20
4.75	5-30	0-15	-
2.36	0-10	0-10	0-10
0.300	0-8	0-8	0-8
0.075	0-5	0-5	0-5

When Paving in Cold Areas Engineers Can Make Paving Asphalt Temperature Susceptibility Work for Them or Against Them

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WHEN PAVING IN COLD AREAS ENGINEERS CAN MAKE PAVING ASPHALT TEMPERATURE SUSCEPTIBILITY WORK FOR THEM OR AGAINST THEM

by

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ABSTRACT

Paving asphalt temperature susceptibility is defined, and a method for its measurement in terms of pen-vis number (PVN) values is described. It is shown how engineers can make paving asphalt temperature susceptibility work for instead of against them when selecting paving asphalts of different temperature susceptibilities for asphalt surface, binder and base course layers for heavy, medium and light traffic in colder climates.

I INTRODUCTION

Paving asphalt is an engineering material, and its two most important engineering properties are:

1. its penetration at 25°C (77°F)
2. its temperature susceptibility

Paving engineers are reasonably familiar with requirements for penetration at 25°C (77°F), but their attitudes toward the temperature susceptibility property of paving asphalts vary over a very wide spectrum:

(a) A large number of engineers appear to be unaware either of the temperature susceptibility property, or that it is a paving asphalt property with which they should be as familiar as with penetration at 25°C (77°F).

(b) Other engineers, while being either aware or unaware of this property, tend to ignore it. They select and employ paving asphalts without regard for their temperature susceptibilities, and are then often surprised at the resulting questionable pavement performance such as high early maintenance, short service lives, and even premature pavement failure that sometimes occurs.

(c) Probably the most common engineering practice of all on a world wide basis is to merely sweep the property of paving asphalt temperature susceptibility with its various challenges under a rug by writing specifications that exclude the use of all paving asphalts except those of low or relatively low temperature susceptibility.

At the present time, paving asphalts in North America are being made from a very large number of crude oils as compared with the pre-1973 OPEC oil crisis, when crude oil from a single source often provided a user agency with the same paving asphalt year after year for many years. Today, petroleum refiners must often operate on any crude oil that is available. The property of a paving asphalt that is most greatly affected by the current wide variety of crude oil sources (estimated to be 600 crude oil sources and blends in North America (1)), is its temperature susceptibility. Moreover, because of declining crude oil reserves, engineers may eventually be glad to obtain the paving asphalt provided by any crude oil source regardless of its temperature susceptibility. Consequently, it would appear that engineers should begin now to gain some experience with paving asphalts of different temperature susceptibilities. They should approach paving asphalt temperature susceptibility as they ordinarily approach the properties of any other engineering material, and should learn when, how and where paving asphalts of different temperature susceptibilities can be

most effectively employed.

With this in mind, the subject matter of this paper will start at the beginning by first of all indicating what paving asphalt temperature susceptibility is, second, by showing how it can be measured, and finally by demonstrating how paving engineers can use paving asphalt temperature susceptibility to work either for them or against them, when selecting paving asphalts of different temperature susceptibilities for asphalt surface, binder, or base course layers for heavy, medium or light traffic in colder climates.

II WHAT IS PAVING ASPHALT TEMPERATURE SUSCEPTIBILITY?

As illustrated by Figure 1, temperature susceptibility is the change in consistency (penetration or viscosity) that an asphalt undergoes for a given change in temperature. Because Asphalt 3 in Figure 1 has the steepest slope, its consistency changes the most over a given temperature range of the three asphalts illustrated, and it is therefore said to have a high temperature susceptibility. The change in consistency of Asphalt 1 is represented by the line of flattest slope in Figure 1, and therefore, its consistency changes the least over the same range of temperature and it is said to have low temperature susceptibility. The consistency change of Asphalt 2 over the same temperature range in Figure 1, is indicated by an intermediate slope, and it is therefore said to have a medium temperature susceptibility.

The engineering significance of paving asphalt temperature susceptibility for the three asphalts in Figure 1 having the same penetration at 25°C (77°F) but with three different temperature susceptibilities, is that Asphalt 3 will be softer or more fluid at all temperatures above 25°C (77°F) and will therefore provide lower pavement stability at these temperatures than the other two asphalts, but will be harder than them at all temperatures below 25°C (77°F), and will therefore provide harder asphalt pavements (higher moduli of stiffness) at all temperatures below 25°C (77°C). Similarly, Asphalt 2 will be softer or more fluid at all temperatures above 25°C (77°F), and harder at all temperatures below 25°C (77°F), than Asphalt 1.

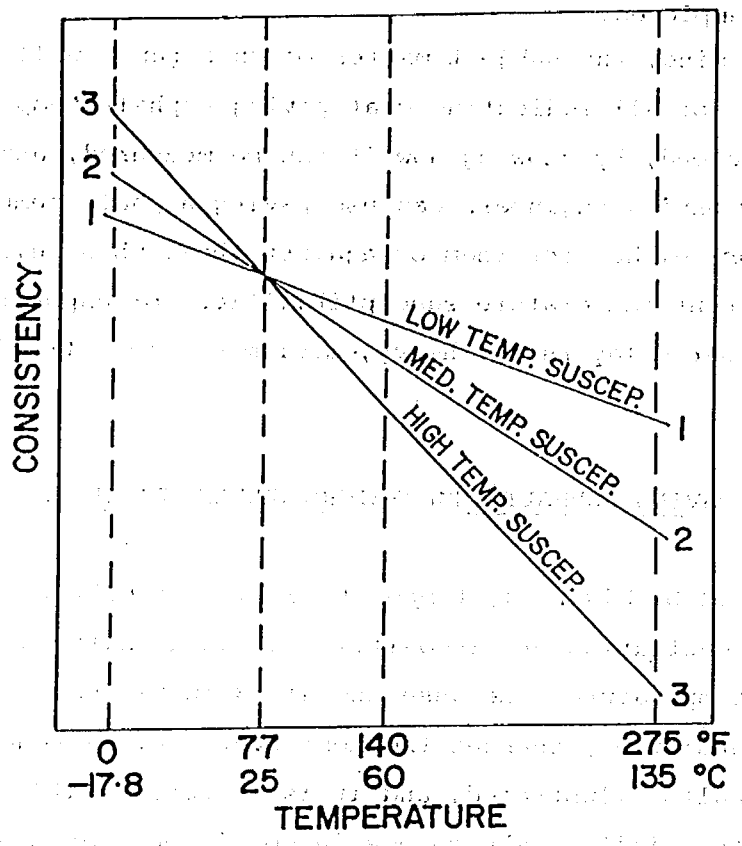

Fig. 1 SKETCH ILLUSTRATING DIFFERENT TEMPERATURE SUSCEPTIBILITIES OF PAVING ASPHALTS

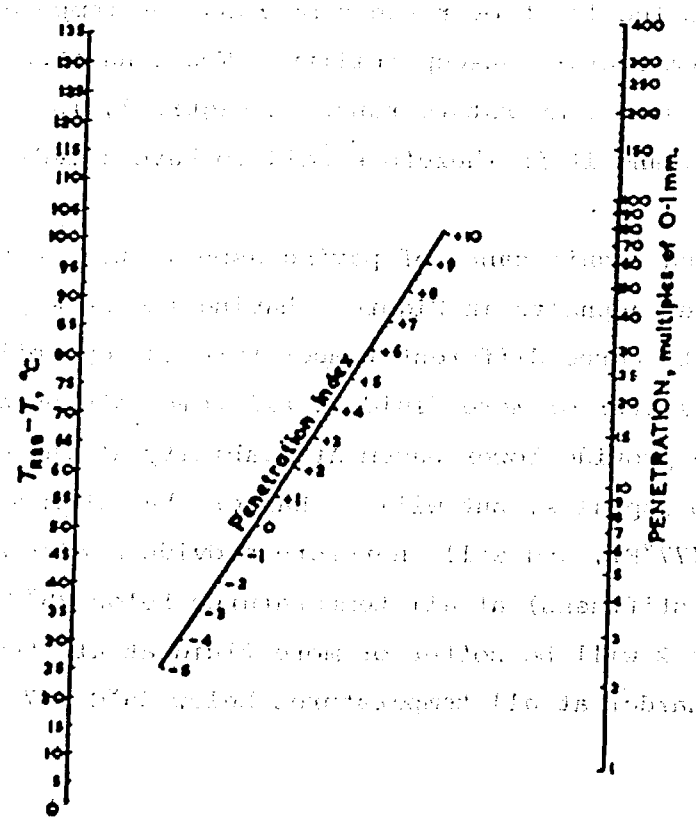

FIG. 2 *Nomograph for determination of penetration index*

III HOW IS PAVING ASPHALT TEMPERATURE SUSCEPTIBILITY MEASURED?

For many years paving asphalt temperature susceptibility has been measured in units of penetration index (PI), Figure 2, a term that was originated by Pfeiffer and Van Doormaal (2) of the Royal Dutch Shell Asphalt Laboratory at Amsterdam, Holland. When using Figure 2, the penetration index (PI) of a paving asphalt is obtained by placing a straight edge at its penetration at 25°C (77°F) on the right hand ordinate, and at a point on the left hand ordinate that is 25°C (77°F) below the ring and ball softening point temperature when 25°C (77°F) is the temperature at which the penetration test is made, and reading off the value that the straight edge intersects on the oblique line in Figure 2 that represents penetration index (PI) ratings. For example, if the penetration at 25°C (77°F) of a paving asphalt is 100, and if its ring and ball softening point temperature is 46°C, a straight edge placed at 46-25=21°C on the left hand ordinate and at 100 penetration on the right hand ordinate, will intersect the oblique line at a penetration index (PI) of -0.5, a quite common value for paving asphalt grades.

This information can then be used to read off from Van der Poel's nomograph (3), Figure 3, the modulus of stiffness value developed by this paving asphalt for a time of loading equal to that of a heavy truck travelling over a point in a pavement surface at 100km (60 miles) per hour, 0.01 second, at a service temperature of 50°C (122°F), for example. When using Figure 3 for this purpose, a straight edge is placed at a time of loading of 0.01 second on the time of loading scale at the bottom of the nomograph, and at a temperature of 50-46=4°C above the ring and ball softening point temperature on the temperature scale, and the modulus of stiffness of the paving asphalt is read from the point where the straight edge intersects the horizontal line across the top of Figure 3 representing a penetration index (PI) of -0.5. This indicates a modulus of stiffness value of $1.4 \times 10^5 N/m^2$ (20.3 psi) for the paving asphalt for these particular conditions.

However, paving asphalts from Western Canadian waxy light crudes have false ring and ball softening point temperatures. Consequently, for these paving asphalts, Figure 2 provides false values for penetration index (PI), and Figure 3 therefore gives incorrect values for modulus of stiffness. Since it appears to overcome or at least to greatly reduce the effect of this problem with paving asphalts from waxy crude oils, and because for any given penetration at 25°C (77°F), a high viscosity at 135°C (275°F) has been known for a long time to indicate low temperature susceptibility for

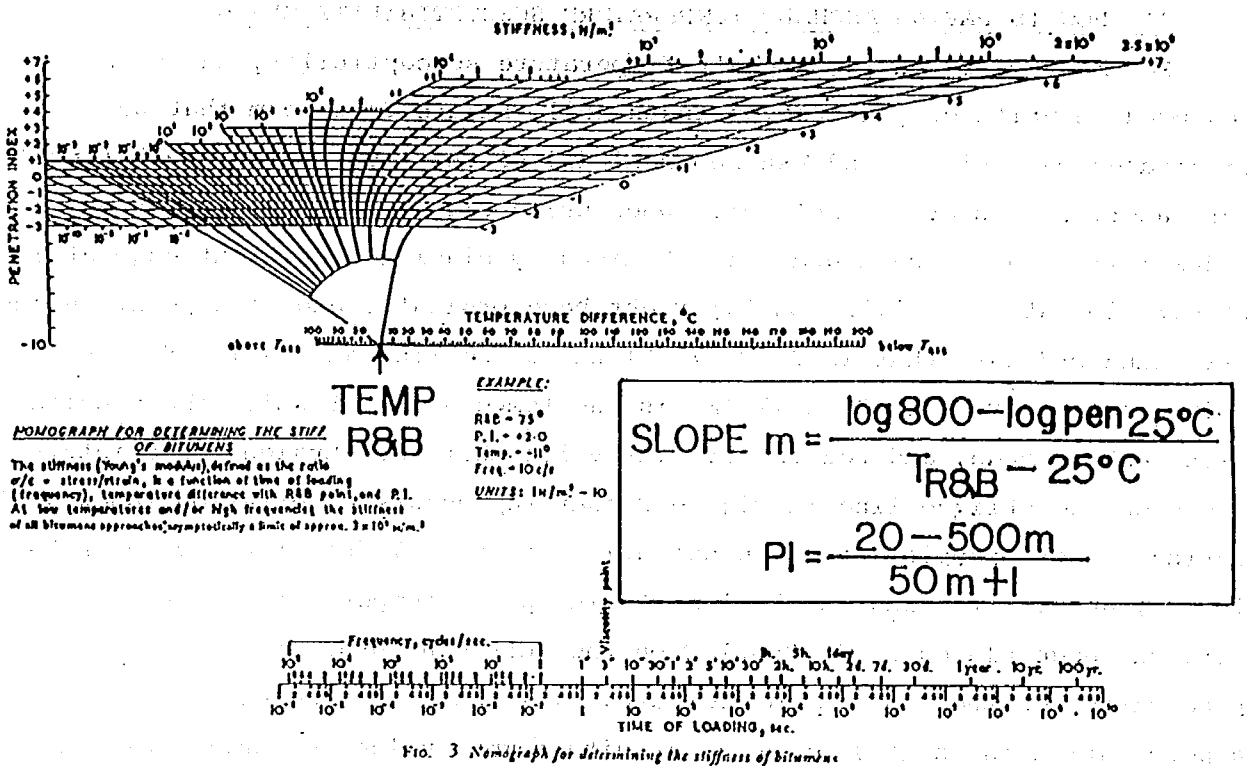

FIG. 3 *Nomograph for determining the stiffness of bitumens*

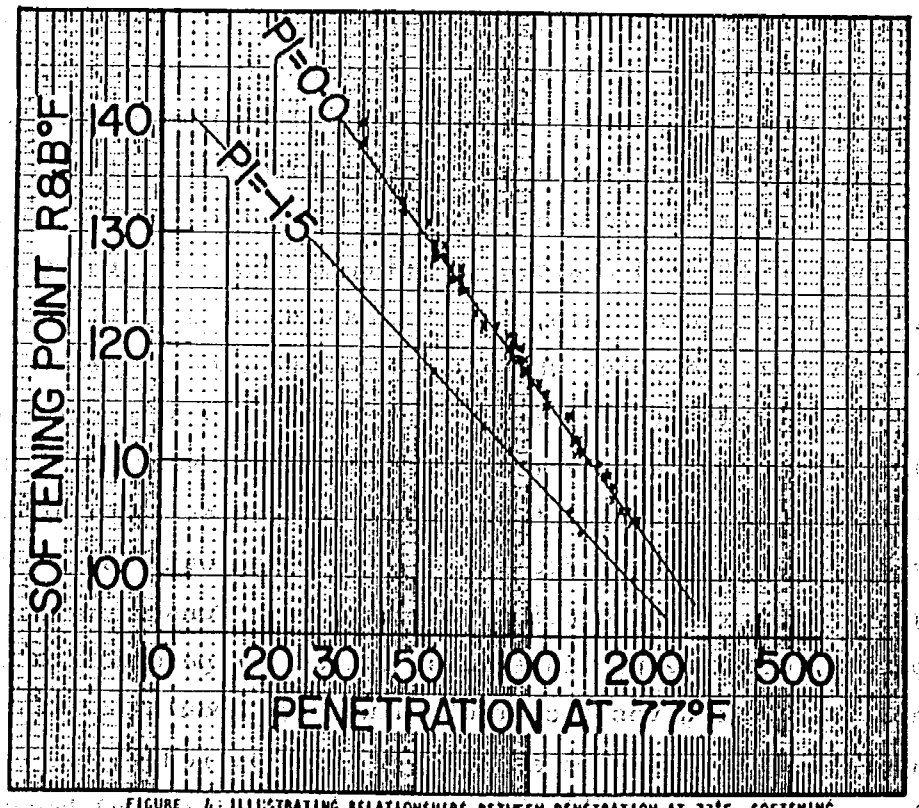

FIGURE 4 ILLUSTRATING RELATIONSHIPS BETWEEN PENETRATION AT 77°F, SOFTENING POINT °F (RING AND BALL), AND PENETRATION INDEX (PI) VALUES OF 0.0 AND +1.5 FOR PAVING ASPHALTS MANUFACTURED BY STEAM OR VACUUM REDUCTION

paving asphalts, and vice versa, I substituted viscosity at 135°C (275°F) in centistokes for ring and ball softening point temperature for the determination of paving asphalt temperature susceptibility (4). To avoid confusion with the term, "penetration index (PI)" that is used in Figure 2 to evaluate temperature susceptibility, this new measure of temperature susceptibility that is obtained from penetration at 25°C (77°F) and viscosity at 135°C (275°F) in centistokes, was given a different designation, "pen-vis number (PVN)".

However, I wished to use Van der Poel's very valuable nomograph (3), Figure 3, to determine paving asphalt moduli of stiffness values, but in terms of PVN rather than PI. Therefore, it was necessary to correlate PVN, which is based on penetration at 25°C (77°F) and viscosity at 135°C (275°F), with PI, which is based on penetration at 25°C (77°F) and ring and ball softening point temperature, but for essentially wax-free paving asphalts.

The U.S. Federal Highway Administration (at that time the U.S. Bureau of Public Roads) had published in 1959 (5) and in 1960 (6) very complete inspection data on more than 300 samples of paving asphalts received from various parts of the United States, most of which had been manufactured by steam or vacuum distillation. These inspection data included penetration at 25°C (77°F), ring and ball softening point temperature, and viscosity at 135°C (275°F). To include approximately the full range of temperature susceptibility represented by PI values for these 300 paving asphalt samples, a PI = 0.0 and a PI of -1.5 were selected. However, to be able to employ data for a large number of samples, the PI values of 0.0 and -1.5 were slightly enlarged to include all asphalts within the ranges of PI = 0.0 $\pm$ 0.2 and PI = -1.5 $\pm$ 0.2.

Figure 4 shows a plot of data for softening point temperature versus corresponding penetration at 25°C (77°F) for each of the more than 300 samples that were manufactured by steam or vacuum distillation, which provide PI values of 0.0 $\pm$ 0.2 and -1.5 $\pm$ 0.2. Because of an abundance of data providing PI values in the vicinity of 0.0, there are approximately 80 points on the line in Figure 4 representing a PI = 0.0 $\pm$ 0.2. However, due to the scarcity of data associated with PI values in the neighbourhood of -1.5, there are only 8 points on the line representing a PI of -1.5 $\pm$ 0.2. Nevertheless, all of these latter data points are practically right on this line. Least squares lines were drawn through these two sets of plotted data in Figure 4 to represent a PI of 0.0 and a PI of -1.5.

It should be recognized that because of the lack of precision due to the repeatability and reproductibility of the penetration at 25°C (77°F) and softening point tests, some of the data included in Figure 4 should not be there, and other data that should have been there, were excluded. This might have some small influence on the locations of the lines for PI = 0.0 and PI = -1.5 that have been drawn in Figure 4.

To convert PI values of 0.0 and -1.5 to corresponding PVN values of 0.0 and -1.5, the next step consisted of going to FHWA's list of inspection data for approximately 300 paving asphalt samples, and for the penetration at 25°C (77°F) for each asphalt sample with a PI of 0.0 $\pm$ 0.2 included in Figure 4, picking out the corresponding value recorded for viscosity at 135°C (275°F) for the same sample and plotting it on Figure 5. A similar procedure was followed to select for the penetration at 25°C (77°F) for each asphalt sample included in Figure 4 for a PI of -1.5 $\pm$ 0.2, the corresponding viscosity at 135°C (275°F) for the same sample and to plot it on Figure 5. Least squares lines were then drawn through the data of Figure 5 to represent values for PVN = 0.0 and PVN = -1.5.

It should be particularly noted that values for penetration at 25°C (77°F) for the same paving asphalt samples have been plotted as abscissa in both Figures 4 and 5, with the corresponding softening point temperatures being plotted as ordinate in Figure 4, and the corresponding viscosities at 135°C (275°F) being plotted as ordinate in Figure 5. Therefore, the PVN values for 0.0 and -1.5 in Figure 5 correspond to PI values of 0.0 and -1.5, respectively, in Figure 4.

Figure 6 indicates the equations for the lines representing a PVN of 0.0 and a PVN of -1.5. It also includes lines for PVN values between +0.5 and -2.0 that cover practically all paving asphalt grades manufactured in North America by steam or vacuum distillation, which is by far the most common method of paving asphalt manufacture.

To use Figure 6 to obtain a PVN value representing the temperature susceptibility of a paving asphalt sample, plot the point representing its penetration at 25°C (77°F) as the abscissa versus its viscosity in centistokes at 135°C (275°F) as the ordinate on Figure 6, and read off its PVN value by interpolating between the lines representing different PVN values. Since pepentration at 25°C (77°F) and viscosity in centistokes at 135°C (275°F) are usually measured during the routine inspection of a sample of paving asphalt, no additonal testing is required to obtain its PVN value.

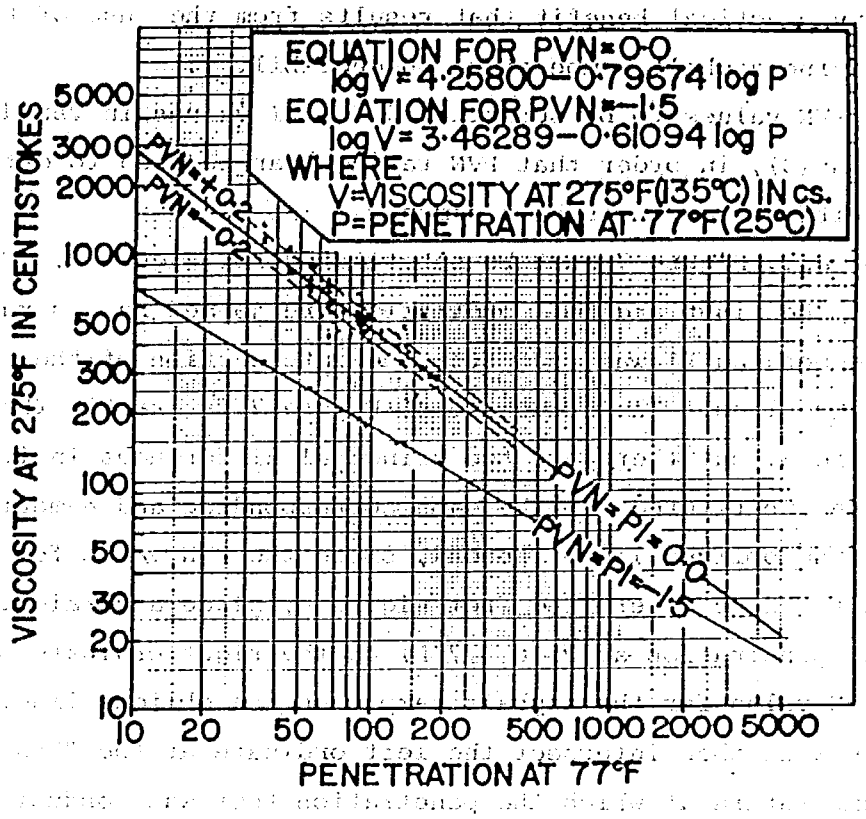

FIGURE 5 ILLUSTRATING RELATIONSHIPS BETWEEN PENETRATION AT 77°F, VISCOSITY AT 275°F IN CENTISTOKES, AND PEN-VIS NUMBERS OF 0.0 AND −1.5 FOR PAVING ASPHALTS MANUFACTURED BY STEAM OR VACUUM REDUCTION

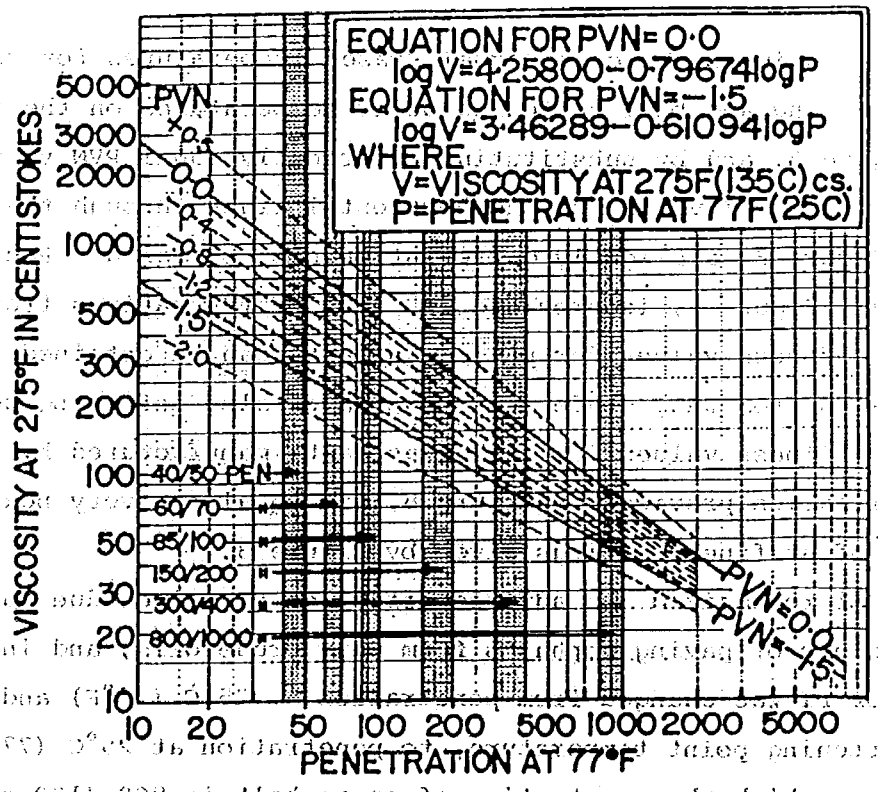

FIGURE 6 A CHART FOR THE DETERMINATION OF APPROXIMATE VALUES FOR PEN-VIS NUMBERS FOR ASPHALT CEMENTS

This is a very practical benefit that results from the use of PVN as a measure of paving asphalt temperature susceptibility.

How are PVN values to be substituted for PI values in Van der Poel's 1954 nomograph (3), in order that PVN ratings can be used to determine the modulus of stiffness of an asphalt cement?

First of all, Figure 7 must be substituted for Figure 2. For Figure 2, the values on the right and left ordinates were joined by a straight edge, and the corresponding PI was read from the intersection of the straight edge with the oblique line. The structure of Figure 7 is exactly the same as that for Figure 2, but there are two principal differences in its use. First, PVN values are substituted for the same corresponding and numerically equal PI values on the oblique line. Second, since the PVN value for a sample of paving asphalt can be determined from Figure 6, place a straight edge on the value for penetration at 25°C (77°F) on the right ordinate of Figure 7, and on the PVN value for the asphalt sample on the oblique line. The straight edge will then intersect the left ordinate at the "base" temperature minus the temperature at which the penetration test was conducted, which is usually 25°C (77°F). Therefore, the "base" temperature in Figure 7 for a sample of paving asphalt is similar to the softening point temperature in Figure 2.

By substituting numerically equal "base" temperatures for the same corresponding ring and ball softening point temperatures on the temperature scale of Figure 3, and by substituting numerically equal PVN values for the same corresponding PI values on the horizontal lines through the upper portion of Figure 3, a slightly different version of Van der Poel's 1954 nomograph (3), Figure 8, is provided in terms of PVN ratings from which modulus of stiffness values for paving asphalts can be obtained. Except for paving asphalts from waxy crude oils that have false softening points, the modulus of stiffness values for paving asphalts manufactured by steam or vacuum distillation provided by Figure 8, are equal, or very nearly so, to the modulus of stiffness ratings given by Figure 3.

Shell Oil recognized the adverse effect on PI values due to the false softening points of paving asphalts from waxy crude oils, and in 1973 (7) the basis for PI was changed from penetration at 25°C (77°F) and the ring and ball softening point temperature, to penetration at 25°C (77°F) and the temperature at which the penetration of an asphalt is 800 (100 g 5 sec). This resulted in the revised Van der Poel's nomograph illustrated in Figure 9 (7). Substituting the temperature at which the penetration is 800, Figure

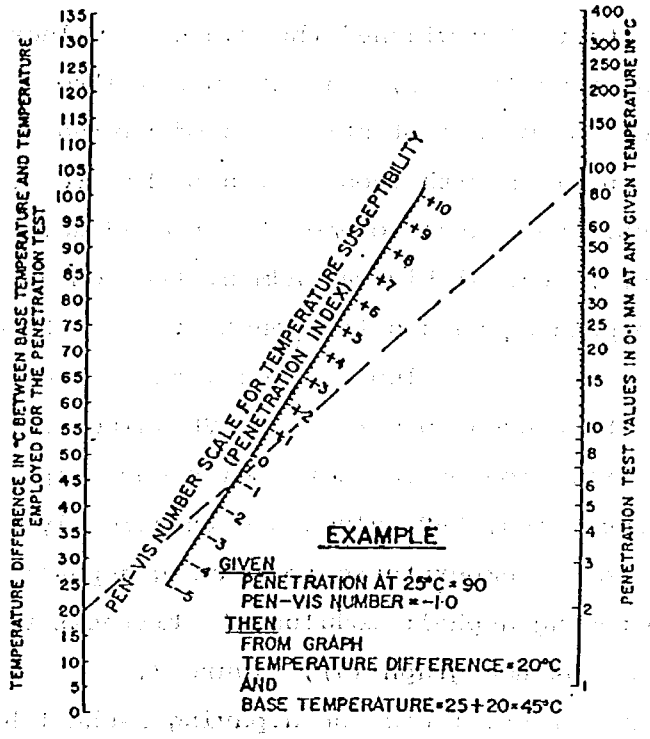

Fig. 7 MODIFICATION OF PFEIFFER'S AND VAN DOORMAAL'S NOMOGRAPH TO DEMONSTRATE RELATIONSHIP BETWEEN VALUES FOR PENETRATION, PEN-VIS NUMBER (PVN) AND BASE TEMPERATURE FOR PAVING ASPHALTS.

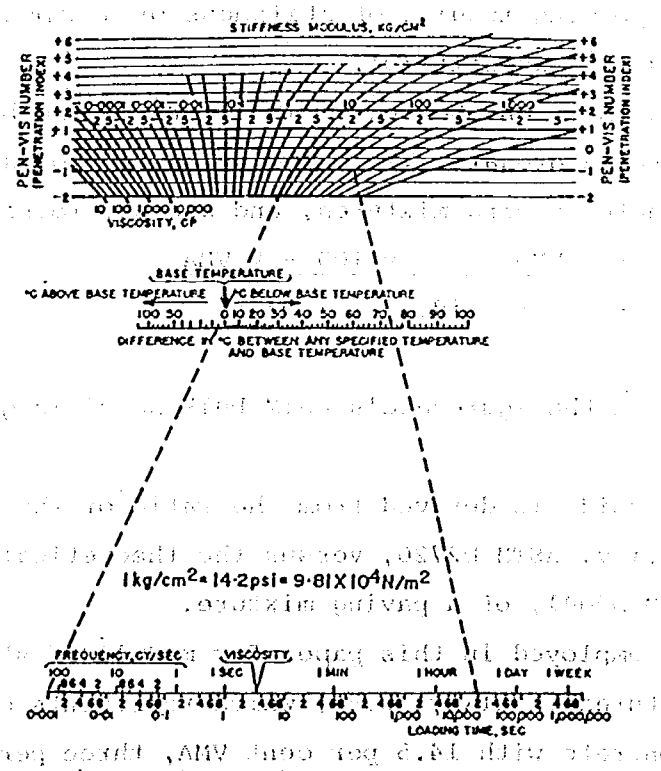

Fig. 8 MODIFICATION OF HEUKELOM'S AND KLOMP'S VERSION OF VAN DER POEL'S NOMOGRAPH FOR DETERMINING MODULI OF STIFFNESS OF PAVING ASPHALTS.

9, for the ring and ball softening point temperature employed for Figure 3, appears to have substantially widened the range of values for the penetration index of paving asphalts (8), and this would also widen the range of corresponding values for moduli of stiffness of paving asphalts determined from Figure 9 in comparison with those given by Figure 3.

There is no published large volume of unbiased data on paving asphalts like those of the U.S. Federal Highway Administration already referred to, that provides corresponding values for penetration at 25°C (77°F), temperature for a penetration of 800 (100 g 5 sec), and viscosity at 135°C (275°F), that would be needed to correlate PVN values with PI values based on Shell Oil's 1973 revision of Van der Poel's nomograph (7). Consequently, at least for the time being, PVN values are unavoidably tied to PI values based on softening point temperatures for paving asphalts from non-waxy crude oils, and to paving asphalt modulus of stiffness values associated with Van der Poel's 1954 nomograph (3), Figure 3.

When the modulus of stiffness for a paving asphalt has been determined, the corresponding modulus of stiffness of a compacted asphalt paving mixture containing this paving asphalt can be read from Figure 10, for which the abscissa is the modulus of stiffness of a paving asphalt, and the ordinate axis is the corresponding modulus of stiffness of a compacted paving mixture containing this asphalt (9).

The C_v values shown on the curves in Figure 10 are based on per cent voids in the mineral aggregate (VMA), and on an air voids value of three per cent for compacted paving mixtures, and are determined as follows:

$$C_v = \frac{100 - \%\ VMA}{100 - \%\ air\ voids} = \frac{100 - \%\ VMA}{97}$$

where:

VMA is based on the aggregate's ASTM bulk specific gravity, and

per cent air voids is derived from the ratio of the bulk compacted specific gravity, ASTM D2726, versus the theoretical maximum specific gravity, ASTM D2041, of a paving mixture.

The C_v value employed in this paper for moduli of stiffness of compacted paving mixtures is always 0.88, which represents compacted dense graded asphalt concrete with 14.5 per cent VMA, three per cent air voids, and a nominal maximum aggregate particle size of 5/8 inch.

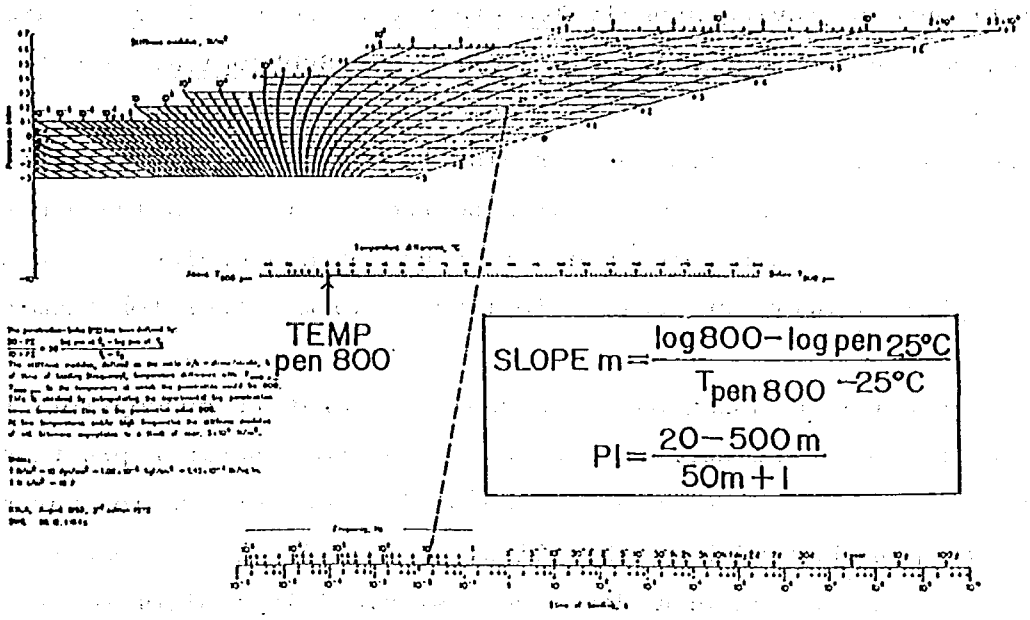

Figure 9 Revised van der Poel Nomograph for bitumen stiffness

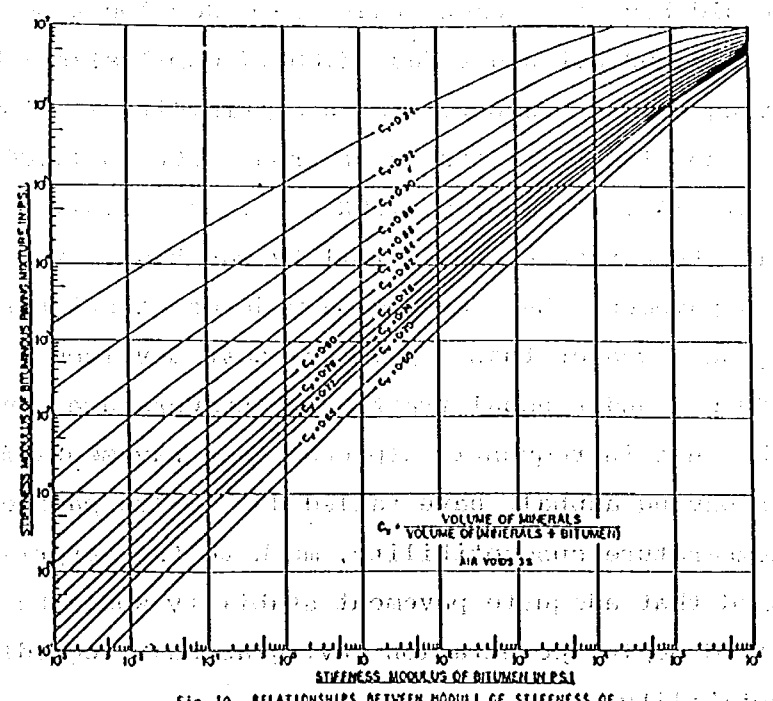

Fig. 10 RELATIONSHIPS BETWEEN MODULI OF STIFFNESS OF PAVING ASPHALTS AND OF PAVING MIXTURES CONTAINING THE SAME PAVING ASPHALTS (BASED ON VAN DER POEL).

IV · · USING PAVING ASPHALT TEMPERATURE SUSCEPTIBILITY TO WORK FOR OR AGAINST AN ENGINEER

1. Influence of Paving Asphalt Temperature Susceptibility on Warm Weather Pavement Stability in Tropical or Subtropical Climates with no Frost.

The following illustration pertains to paving asphalt selection in tropical or subtropical climates with no frost. It is included here because it provides a simple, clear instance of how engineers can use paving asphalt temperature susceptibility to work either for them or against them.

This example of using paving asphalt temperature susceptibility to work against us as engineers, is based on our not uncommon practice of specifying 80/100 (85/100) penetration or 150/200 penetration paving asphalt for a paving project without the slightest regard for its temperature susceptibility. Figure 11 provides a very simple illustration of this when 80/100 penetration asphalt has been specified for a pavement to carry heavy traffic in a warm region with a maximum pavement temperature of 60°C (140°F) at a pavement depth of about 2 inches, but where there is never any frost. The shaded area in Figure 11 demonstrates that by merely specifying 80/100 penetration paving asphalt without regard for its temperature susceptibility, an engineer could have a pavement that would develop a modulus of stiffness from 11,000 to 18,000 psi under fast (100km(60 miles)/hr) heavy truck traffic if the paving asphalt temperature susceptibility was low, (PVN = 0.0 to -0.5), or a modulus of stiffness of from 9,000 to 15,000 psi if its temperature susceptibility was medium, (PVN = -0.5 to -1.0), or from 7,000 to 12,000 psi if its temperature susceptibility was high, (PVN = -1.0 to -1.5). Obviously a pavement that develops a modulus of stiffness of only 7,000 psi, is very much weaker than a pavement that develops a modulus of stiffness of 18,000 psi under equal traffic and temperature conditions. Nevertheless, it has not infrequently happened that pavements made with 80/100 penetration paving asphalt have failed due to low pavement stability because of high temperature susceptibility, much to the surprise of engineers who had assumed that adequate pavement stability would be provided if they merely specified 80/100 penetration paving asphalt regardless of its temperature susceptibility.

The hatched area of Figure 11 demonstrates that to develop a pavement modulus of stiffness from 13,500 to 18,000 psi at 60°C (140°F) under fast heavy truck traffic, an engineer should specify:

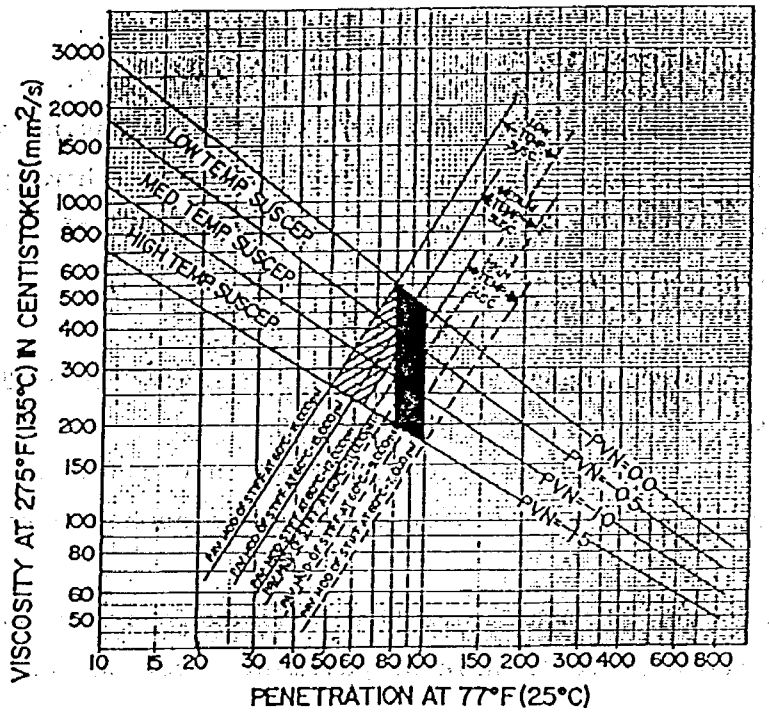

Fig. 11 ILLUSTRATING HOW PAVING ASPHALT TEMPERATURE SUSCEPTIBILITY CAN BE MADE TO WORK FOR OR AGAINST ENGINEERS IN VARM CLIMATES.

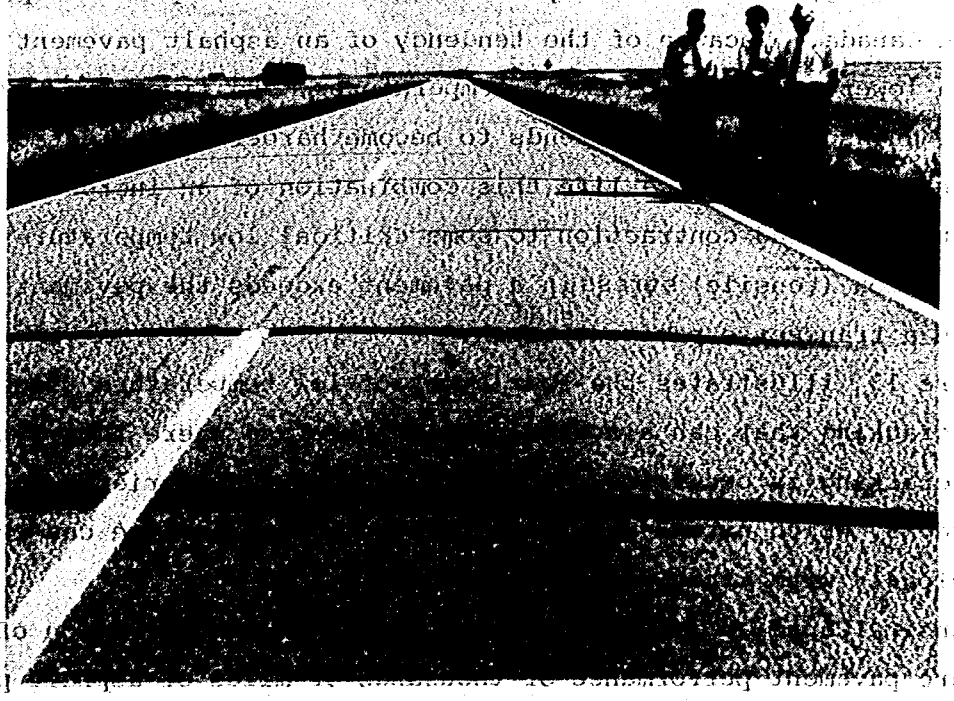

FIGURE 12 ILLUSTRATING LOW TEMPERATURE TRANSVERSE PAVEMENT CRACKING

(a) 80/100 penetration asphalt, if its PVN were 0.0

(b) 70/86 penetration asphalt, if its PVN were -0.5

(c) 60/76 penetration asphalt, if its PVN were -1.0

(d) 50/65 penetration asphalt, if its PVN were -1.5.

This is an example of how an engineer could make paving asphalt temperature susceptibility work for him rather than against him, particularly when he is faced (as he often is) with a limited choice of paving asphalt temperature susceptibility.

2. Influence of Paving Asphalt Temperature Susceptibility On Low Temperature Transverse Pavement Cracking

Because of the extremities of our climate in Canada, paving engineers must almost always compromise between three basic asphalt pavement design requirements:

1. How to design asphalt pavements that will avoid or at least greatly reduce low temperature transverse pavement cracking during our cold winters,
2. How to design asphalt pavements with adequate stability for fast truck traffic in warm summer weather, and
3. How to design asphalt pavements with sufficient resistance to rutting under summer traffic.

Figure 12 provides a typical illustration of low temperature transverse pavement cracking, which is a very serious cold weather pavement performance problem in Canada. Because of the tendency of an asphalt pavement to contract with lower and lower pavement temperatures, and since the paving asphalt in an asphalt pavement tends to become harder and harder each year, at some stage in its service life this combination of an increasingly harder asphalt binder and contraction to some critical low temperature at which the contraction (tensile) stress in a pavement exceeds the pavement strength, results in a transverse pavement crack.

Figure 13 illustrates the four types of low temperature transverse pavement cracking that can almost always be observed where this type of pavement cracking is occurring. Within the writer's experience, only the Type 1 cracks, which extend completely across a traffic lane can be correlated closely with other pavement properties.

Extensive Canadian investigation, together with observation of the low temperature pavement performance of thousands of miles of asphalt pavements

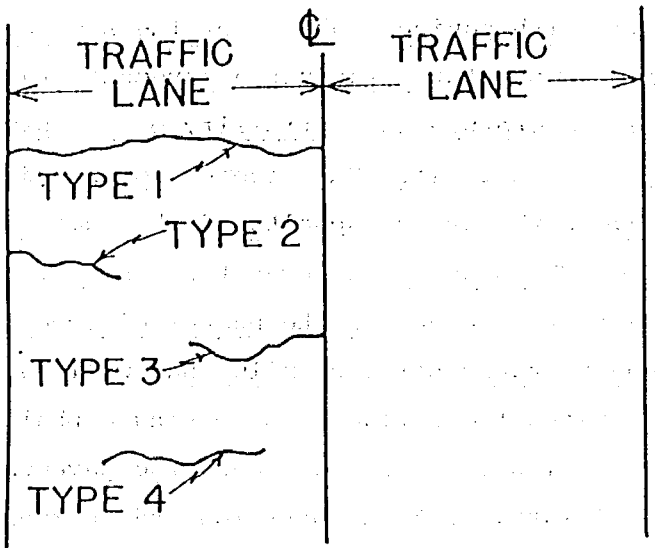

FIG. 13 TYPES OF TRANSVERSE PAVEMENT CRACKS.

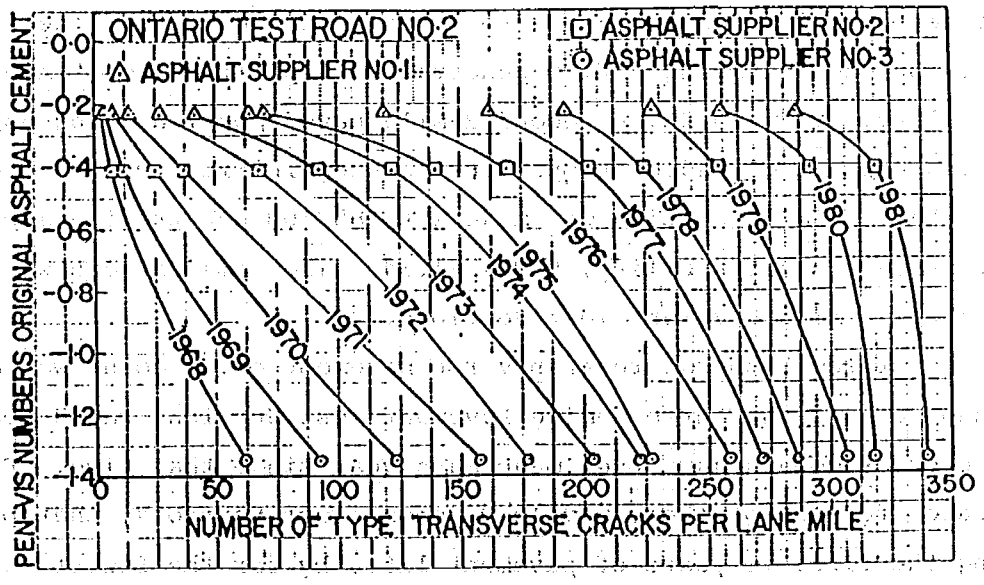

FIGURE 14 INFLUENCE OF PAVING ASPHALT TEMPERATURE SUSCEPTIBILITIES ON ANNUAL COUNT OF TYPE 1 LOW TEMPERATURE TRANSVERSE PAVEMENT CRACKS PER LANE MILE

in service have provided the following two basic conclusions with respect to low temperature transverse pavement cracking:

(a) For pavements made with paving asphalts of the same penetration at 25°C, the number of low temperature transverse pavement cracks per lane mile increases as the temperature susceptibility of the paving asphalt becomes higher and higher (its PVN becomes lower and lower).

(b) For pavements made with paving asphalts of the same temperature susceptibility (same PVN value), the lower the penetration at 25°C of the paving asphalt, the larger is the number of Type 1 low temperature transverse pavement cracks per lane mile that will develop.

Figure 14 demonstrates the influence of paving asphalt temperature susceptibility on Type 1 low temperature transverse pavement cracking. In 1960, the Ontario Ministry of Transportation and Communications constructed three 6-mile Pavement Test Sections as part of an in-service highway paving program at three locations about 40 miles apart in southwestern Ontario. That is, each of these three Pavement Test Sections was constructed as a part of the pavement on an in-service highway. The paving asphalt used for each of the three Pavement Test Sections was 85/100 penetration, but was of three different temperature susceptibilities, low temperature susceptibility, PVN = -0.23, medium temperature susceptibility, PVN = -0.41, and high temperature susceptibility PVN = -1.34. Consequently, each 6-mile Pavement Test Section was divided into three 2-mile portions, one 2-mile paved portion incorporating 85/100 penetration asphalt of low temperature susceptibility, PVN = -0.23, a second 2-mile paved portion using 85/100 penetration asphalt of medium temperature susceptibility, PVN = -0.41, while the third 2-mile portion was made with 85/100 penetration asphalt of high temperature susceptibility, PVN = -1.34. The same pattern was followed for all three 6-mile Pavement Test Sections.

The first low temperature transverse pavement crack count was made in 1968, when each of the three 6-mile Pavement Test Sections was 8 years old, and these crack counts have been made each year since that time. Figure 14 demonstrates for one of these 6-mile Pavement Test Sections, that particularly during the first 16 years of pavement life, by far the fewest total number of Type 1 low temperature transverse pavement cracks per lane mile had occurred each year in the pavement made with 85/100 penetration asphalt of lowest temperature susceptibility, PVN = -0.23, the second lowest total number of these cracks each year had developed in the pavement made with 85/100 penetration paving asphalt of medium temperature susceptibility,

PVN = -0.41, while the greatest number of these cracks developed in the 2-mile pavement portion made with 85/100 penetration paving asphalt of high temperature susceptibility, PVN = -1.34.

From weather records at the nearby London, Ontario, International Airport, the average annual minimum ambient temperature at each of the 6-mile Ontario Pavement Test Sections during the 15 year period between 1960 and 1975 was -24.3°C (-11.8°F).

Precisely similar results were obtained at the other two 6-mile Ontario Pavement Test Sections. These findings are also supported by results from the Ste. Anne Test Road in Manitoba, that was conducted as a joint venture by the Manitoba Department of Highways and Transportation and Shell Canada Limited.

Figure 14 provides supporting data for the conclusion in Item (a) above, which indicated that for pavements made with paving asphalts of the same penetration at 25°C, the number of Type 1 low temperature transverse cracks per lane mile increases as the paving asphalt temperature susceptibility increases.

When paving asphalts have the same temperature susceptibility, Figure 15 demonstrates that the number of low temperature transverse pavement cracks per given length of pavement increases as the penetration at 25°C of the paving asphalt becomes lower and lower (harder and harder). Figure 15 is a picture of low temperature transverse pavement cracking in an asphalt paved runway that was taken at a Canadian airport. The picture shows the occurrence of numerous low temperature transverse pavement cracks along each side of a pavement construction lane. These cracks do not extend to the centre of the lane, which in general shows little or no transverse pavement cracking. The transverse cracking along each side of the construction lane is due primarily to poor compaction along the edges of the lane when it was constructed 12 years before this picture was taken. The centre of the lane was much more thoroughly compacted. Because of poor edge compaction, the longitudinal joints between adjacent construction lanes have opened up, and this added exposure to air has hastened the hardening of the paving asphalt along the sides of each lane, which has been accelerated by the high in-place air voids due to poor compaction of the pavement in the vicinity of the lane edges. Tests on samples from the centre of this construction lane showed compaction to 99 per cent of laboratory compacted density (75-blow Marshall), and in-place air voids of 2 per cent, while the asphalt cement recovered by ASTM D 1856 (Abson method) had penetrations at

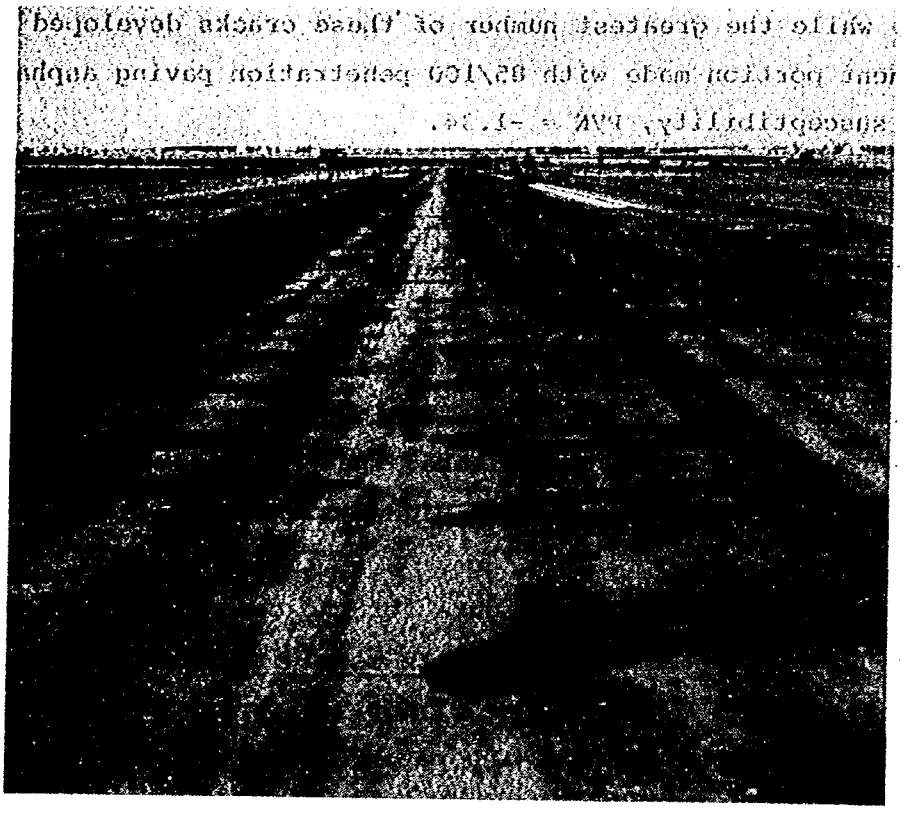

Figure 15 Illustrating Difference Between Low Temperature Transverse Pavement Cracking Along the Edges and in the Centre of a 12-foot Construction Lane on an Airport Runway.

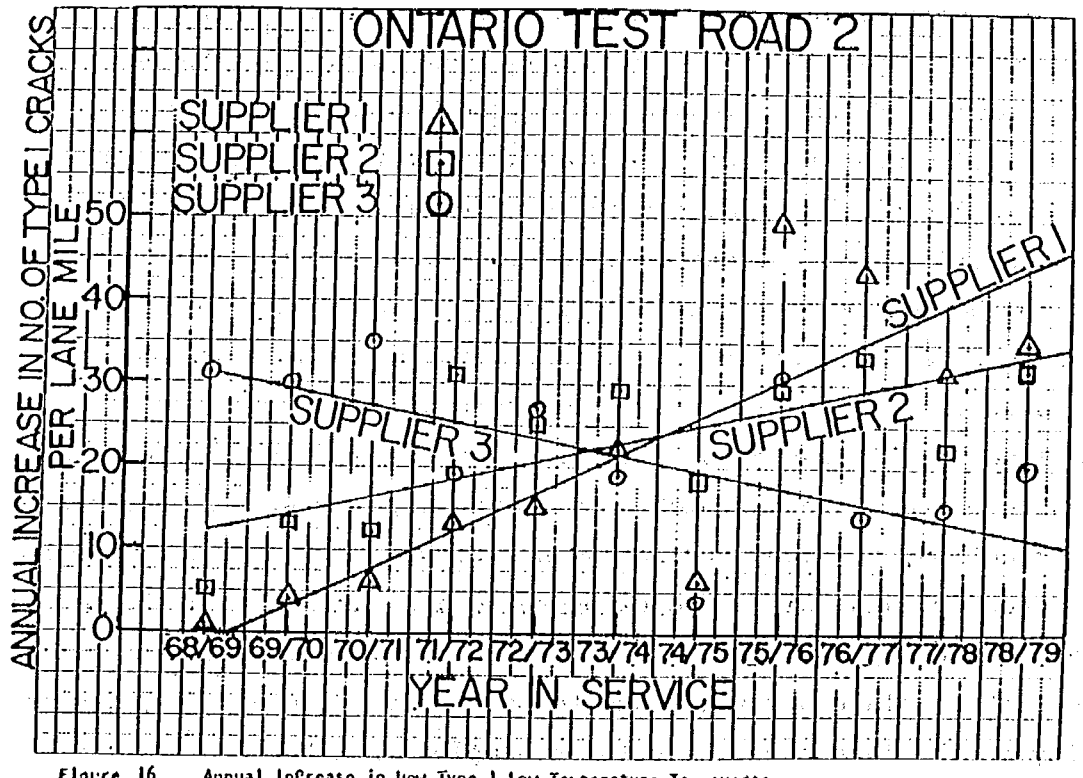

Figure 16 Annual Increase in New Type I Low Temperature Transverse Pavement Cracks Per Lane Mile - Test Road 2

25°C from 77 to 81. By way of contrast, samples from the edges of the construction lane showed an in-place air voids of 9.5 per cent, and only 92 per cent of laboratory compacted specific gravity, while the asphalt cement recovered by ASTM D 1856 had penetrations at 25°C from 13 to 24.

Obviously, the same original paving asphalt (probably 120/150 penetration) was used for the pavement in the centre and at the edtes of the construction lane. Both the centre and the edges of the construction lane were exposed to the same low winter temperatures during a period of 12 years. Consequently, the great differences in low temperature transverse cracking between the centre and the sides of the construction lane has resulted from the much greater hardness that has gradually developed over the 12-year period in the paving asphalt in the pavement along the edges of the construction lane than at its centre.

Therefore, Figure 15 supports the conclusion in Item (b) above, to the effect that for paving asphalts of the same temperature susceptibility, the number of Type 1 low temperature transverse pavement cracks that develop per given pavement length becomes higher and higher as the penetration at 25°C of the paving asphalt selected for the pavement becomes lower and lower (harder and harder).

The least squares lines in Figure 16 indicate the average annual changes in the number of Type 1 low temperature transverse pavement cracks per lane mile that occurred in all three Ontario Pavement Test Sections, although the data shown in Figure 16 are for Ontario Pavement Test Section 2. It shows that for the 2-mile portions make with the two 85/100 penetration paving asphalts of lower temperature susceptibility, PVN = -0.23 to -0.41, there has been an average annual increase in the number of Type 1 low temperature transverse pavement cracks per lane mile since the first crack count was make in 1968. On the other hand, since 1968 there has been an average annual decrease in the number of Type 1 low temperature transverse pavement cracks per lane mile in the 2-mile portions made with paving asphalt of high temperature susceptibility, PVN = -1.34.

There are those who claim that properties of a paving mixture other than the characteristics of the paving asphalt are responsible for low temperature transverse pavement cracking. However, it is difficult to see how any property of an asphalt pavement other than that of the paving asphalt itself could provide the data that are illustrated in Figure 16.

Based on much investigation similar to that just reviewed, Figure 17 points out how paving asphalts can be selected to avoid or greatly reduce

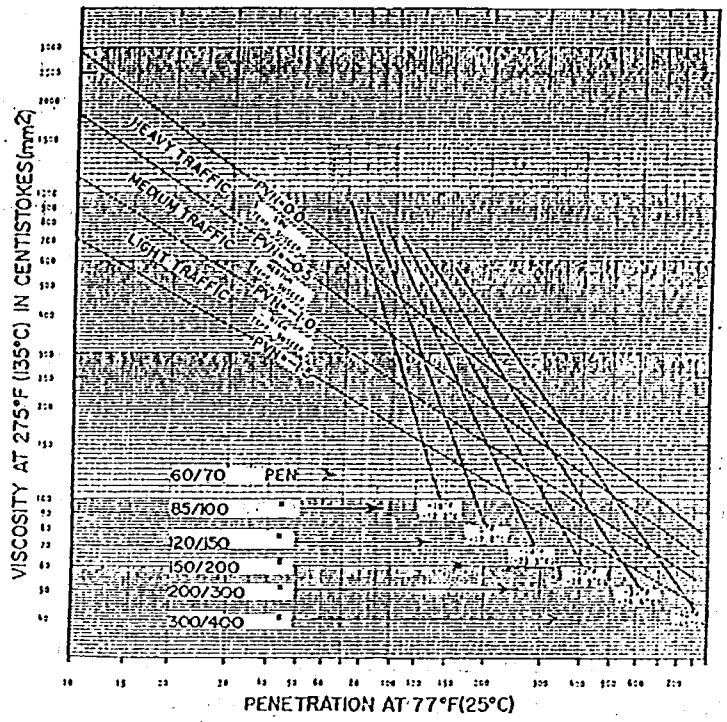

Fig. 17 CHART FOR SELECTING PAVING ASPHALTS WITH VARIOUS COMBINATIONS OF TEMPERATURE SUSCEPTIBILITIES AND PENETRATIONS AT 25°C TO AVOID LOW TEMPERATURE TRANSVERSE PAVEMENT CRACKING AT SELECTED MINIMUM WINTER PAVEMENT TEMPERATURES.

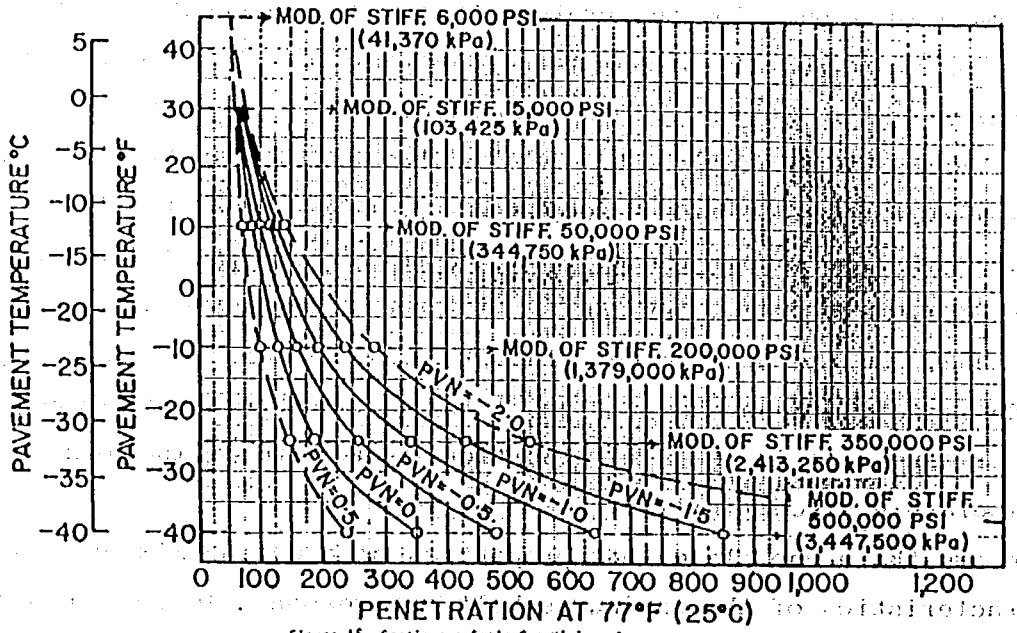

Figure 16 Continuous Scale for Minimum Pavement Temperatures Versus Corresponding Combinations of Temperature Susceptibility (PVN) and Penetration at 25°C To Avoid Low Temperature Transverse Pavement Cracking.

low temperature pavement cracking during our cold Canadian winters. The oblique temperature-labelled lines in Figure 17 indicate the hardest grades of paving asphalt in terms of their temperature susceptibilities and penetrations at 25°C (77°F) that should be selected to avoid low temperature transverse pavement cracking at the lowest pavement temperature anticipated during a pavement's service life. For example, when the expected minimum temperature at a pavement depth of about two inches during a pavement's service life is estimated to be -20°F (-28.9°C), a fairly representative minimum temperature in many parts of Canada, if paving asphalt with a low temperature susceptibility, represented by a PVN = 0.0 is available, Figure 17 shows that the hardest grade to be selected for this asphalt is a penetration at 25°C (77°F) of 165. However, if the only paving asphalt at hand has a high temperature susceptibility indicated by a PVN = -1.5, Figure 17 indicates that the hardest grade of this asphalt to be selected should have a penetration at 25°C (77°F) of 345. Softer asphalt cements selected from the right of this -20°F (-28.9°C) line will provide a greater safety factor against low temperature transverse pavement cracking, while paving asphalts selected from the left of this line will result in increasing low temperature transverse pavement cracking.

The following are the lowest penetrations at 25°C (77°F) for the corresponding PVN values that will provide protection against low temperature transverse pavement cracking at a minimum pavement temperature of -20°F (-28.9°C).

PVN = 0.0 - 165 penetration at 25°C (77°F) or higher (softer)

PVN = -0.5 - 215 penetration at 25°C (77°F) or higher (softer)

PVN = -1.0 - 275 penetration at 25°C (77°F) or higher (softer)

PVN = -1.5 - 345 penetration at 25°C (77°F) or higher (softer)

Similar information is provided by the other oblique temperature-labelled lines in Figure 17.

To facilitate the determination of combinations of temperature susceptibility (PVN) and penetration at 25°F (77°C) values that will just avoid low temperature transverse pavement cracking at minimum pavement temperatures between those shown on the temperature labelled oblique lines in Figure 17, the data in Figure 17 have been rearranged to provide Figure 18, for which the ordinate axis provides a continuous temperature scale.

In recapitulation, therefore, with regard to eliminating or at least greatly reducing low temperature transverse pavement cracking in a region where the minimum pavement temperature at a pavement depth of about two

inches is -20°F (-28.9°C) for example, an engineer would be using paving asphalt temperature susceptibility to work for him, if he selects softer asphalts on or to right of the oblique line labelled -20°F (-28.9°C) in Figure 17. He would be using temperature susceptibility to work against him (more low temperature transverse pavement cracking) if he selects harder asphalts to be left of the oblique line labelled -20°F (-28.9°C).

Some will be wondering about the stability under warm weather traffic of pavements containing the softer paving grades required by Figure 17 to avoid low temperature transverse pavement cracking. This item will be considered next.

3. Relationship Between Paving Asphalt Temperature Susceptibility and Traffic Volume

The Ontario Ministry of Transportation and Communications, The Asphalt Institute, and probably other agencies divide traffic volumes into three categories, heavy, medium and light. These organizations also require the highest minimum pavement stability for heavy traffic, a lower minimum pavement stability for medium traffic, and the lowest minimum stability for light traffic. Table 1 indicates that Ontario specifies minimum Marshall stabilities at 60°C (140°F) of 8900 N (2000 lb) for heavy traffic, 6700 N (1500 lb) for medium traffic and 4400 N (1000 lb) for light traffic. In this paper, pavement stability is usually expressed as modulus of stiffness in psi (kPa or MPa). As shown by Table 1, as a rough rule of thumb, it appears that a minimum modulus of stiffness in psi at the maximum pavement temperature on a project, that is 10 times Ontario's minimum Marshall Stability requirement in pounds at 60°C, provides a reasonable criterion for adequate warm weather pavement stability.

The specifications of many highway, street, and airport agencies exclude all paving asphalts except those of low or relatively low temperature susceptibility for every traffic category. Figure 19 demonstrates that this is an unnecessary engineering requirement that in many cases can be costly and wasteful.

This portion of the paper pertains to the selection of paving asphalts with a wide range of temperature susecptibilities (PVN values) for heavy, medium and light traffic. However, the only asphalts to be considered are those on the right side of the line labelled -10°F (-23.3°C) in Figure 19

TABLE 1

Minimum Modulus of Stiffness and Corresponding Minimum Ontario Marshall Stability Requirements For Heavy, Medium and Light Traffic.

Ontario Minimum Marshall Stability at 60°C (140°F)	Corresponding Minimum Modulus of Stiffness at Maximum Pavement Temperature
Heavy Traffic 8900 N (2000 lb)	20,000 psi (138 MPa)
Medium Traffic 6700 N (1500 lb)	15,000 psi (103 MPa)
Light Traffic 4400 N (1000 lb)	10,000 psi (69 MPa)

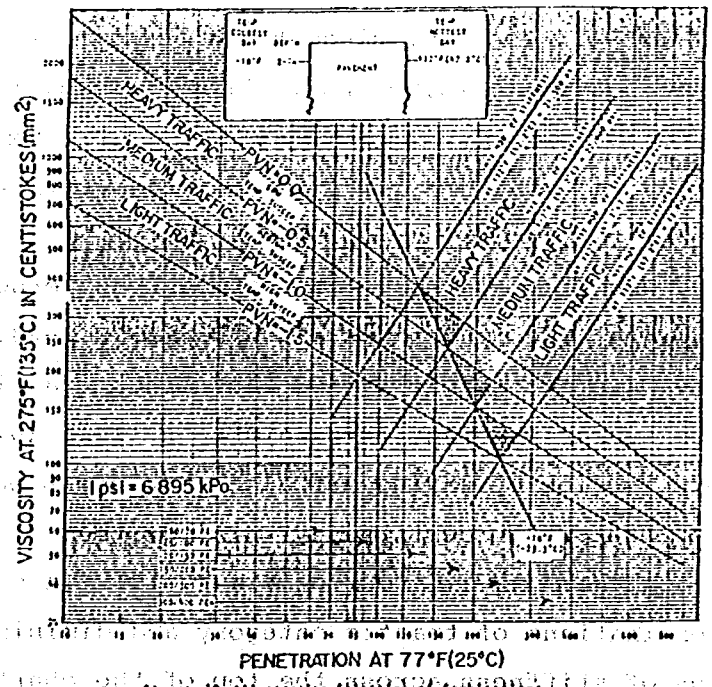

Figure 19 Illustrating selection of combinations of Temperature Susceptibility (PVN) and Penetration at 25°C for Paving Asphalts for Heavy, Medium and Light Traffic in Cold Climates.

that provide pavements that will avoid low temperature transverse pavement cracking at this low pavement temperature.

Across the top of Figure 19, it is shown that heavy traffic should be assigned to pavements containing the hardest paving asphalts that will provide the highest stability, with moduli of stiffness from 22,000 to 31,500 psi, medium traffic to pavements containing softer paving asphalts that furnish lower pavement stability represented by modulus of stiffness values from 17,000 to 22,000 psi, and light traffic to pavements embodying still softer paving asphalts that provide yet lower pavement stability, indicated by moduli of stiffness from 13,000 to 17,000 psi, in every case for traffic travelling at 100km (60 miles) per hour when the pavement temperature at a depth of about 2 inches (50.8 mm) is 47.2°C (117°F). This is in agreement with the principle adopted by many organizations, that for lower and lower traffic volumes, lower and lower pavement stabilities are adequate.

As shown on the left side of Figure 19, heavy traffic should be transported on pavements containing asphalts of lowest temperature susceptibility, PVN = -0.5 to 0.0, because these provide the highest pavement modulus of stiffness values, 22,000 to 31,500 psi at 117°F (47.2°C). Medium traffic should be carried on pavements containing asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, since these furnish somewhat smaller moduli of stiffness, 17,000 to 22,000 psi. Light traffic should be associated with pavements containing paving asphalts of high temperature susceptibility, PVN = -1.5 to -1.0, because these provide still smaller moduli of stiffness, 13,000 to 17,000 psi

Consequently, when avoiding low temperature transverse pavement cracking at a minimum winter pavement temperature of -10°F (-23.3°C), and providing adequate pavement stability for fast truck traffic at the corresponding maximum summer pavement temperature of 117°F (47.2°C), are major criteria for the selection of paving asphalts, engineers would be making paving asphalt temperature susceptibility work for them if they selected paving asphalts from the stippled areas of Figure 19. These stippled areas indicate where combinations of traffic category and diminishing warm weather values for modulus of stiffness across the top of the chart, overlap with combinations of traffic category and increasing paving asphalt temperature susceptibility (decreasing values for PVN) on the left side of Figure 19.

When selected on this basis, it is evident from the stippled areas on Figure 19, that for pavements to carry traffic at a maximum pavement temp-

erature of 117°F (47.2°C) in a region where the minimum pavement temperature is -10°F (-23.3°C), the hardest paving asphalts of lowest temperature susceptibility (PVN=-0.5 to 0.0) and of lowest penetration at 25°C (77°F), 130 to 165 should be selected for pavements to carry heavy traffic, slightly softer paving asphalts of medium temperature susceptibility, PVN=-1.0 to -0.5, and of somewhat higher penetration at 25°C (77°F), 165 to 200, should be selected for pavements for medium traffic, while yet softer paving asphalts of high temperature susceptibility, PVN=-1.5 to -1.0, and of still higher penetration at 25°C (77°F), 200 to 240, should be selected for pavements for light traffic.

As a general principle, in regions where low temperature transverse pavement cracking is to be avoided, while adequate pavement stability is to be provided for warm weather traffic, Figure 19 demonstrates that engineers would be making paving asphalt temperature susceptibility work for them if harder (lower penetration at 25°C) paving asphalts of lowest temperature susceptibility, PVN = 0.0 to -0.5, were selected or reserved for pavements that are to carry heaviest traffic, if somewhat softer paving asphalts of medium temperature susceptibility PVN = -0.5 to -1.0, were chosen or assigned for pavement construction for medium traffic, and if still softer paving asphalts of high temperature susceptibility PVN = -1.0 to -1.5, were employed for pavements for light traffic. This applies particularly to regions where limited volumes of paving asphalts of different temperature susceptibilities are either manufactured or otherwise sparingly available.

While Figure 19 pertains to the selection of paving asphalts that will avoid low temperature transverse pavement cracking at a minimum winter pavement temperature of -10°F (-23.3°C), and that will provide adequate pavement stability for fast truck traffic at the corresponding maximum summer temperature of 117°F (47.2°C), similar information can be easily determined for regions with other combinations of maximum summer and minimum winter pavement temperatures.

With respect to resistance to pavement rutting under warm weather traffic, it should be clear that pavements containing the harder grades of paving asphalt of low temperature susceptibility, PVN = 0.0 to -0.5, indicated by Figure 19 for heavy traffic, will also provide greatest resistance to rutting under heavy traffic. Likewise, pavements incorporating somewhat softer paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, could be expected to have sufficient resistance to rutting under medium traffic, while pavements containing still softer

paving asphalts of high temperature susceptibility, PVN = -1.0 to -1.5, shdulд provide adequate resistance to rutting under light traffic.

4. Making Paving Asphalt Temperature Susceptibility Work For Engineers When Selecting Paving Asphalts For Surface, Binder and Base Courses

Important information to guide the selection of the paving asphalt grade or grades for surface course, binder course and base course for any paving project is illustrated in Figure 20. It provides data indicating the variation of pavement temperature with pavement depth that were actually measured and recorded each hour by The Asphalt Institute at depth intervals of 2 inches throughout the year 1965 for a pavement 12 inches thick at Washington, D.C. (10). It also includes similar information measured during a 2-year joint cooperative research project during 1968 and 1969 by the Manitoba Department of Highways and Transportation and Shell Canada Limited, involving asphalt pavements 4 inches and 10 inches thick at Ste. Anne, Manitoba, which is east of Winnipeg (12), (13). Further background information on Figure 20 is provided elsewhere (14).

The pavement temperature versus pavement depth information illustrated in Figure 20 is for the hottest day and for the coldest day of the year both at Ste. Anne, Manitoba, and at Washington, D.C. It demonstrates that on the coldest day of the year at either location, the lowest pavement temperature occurs at the pavement surface, accompanied by successively warmer temperatures at increasing depths below the surface. On the other hand, on the hottest day of the year, the highest pavement temperature occurs at the pavement surface with accompanying successively lower temperatures at increasing depths below the surface.

Lines A indicate the hottest and coldest pavement temperatures at various asphalt pavement depths recorded at Washington, D.C., and Lines D provide similar information for Ste. Anne, Manitoba. Lines B and C have been arbitrarily drawn at distances of one-third and two-thirds of the interval between Lines A and D. Lines C represent the warmest and coldest pavement temperatures versus pavement depths at locations such as Ottawa and Montreal where winter temperatures are less severe than those in Manitoba. Lines B provide similar information for milder regions in Canada such as southwestern Ontario. Other lines like B and C could be drawn roughly parallel to Lines A and D for any region where an engineer has an accurate measure of a representative minimum or maximum pavement temperature at some convenient pavement depth.

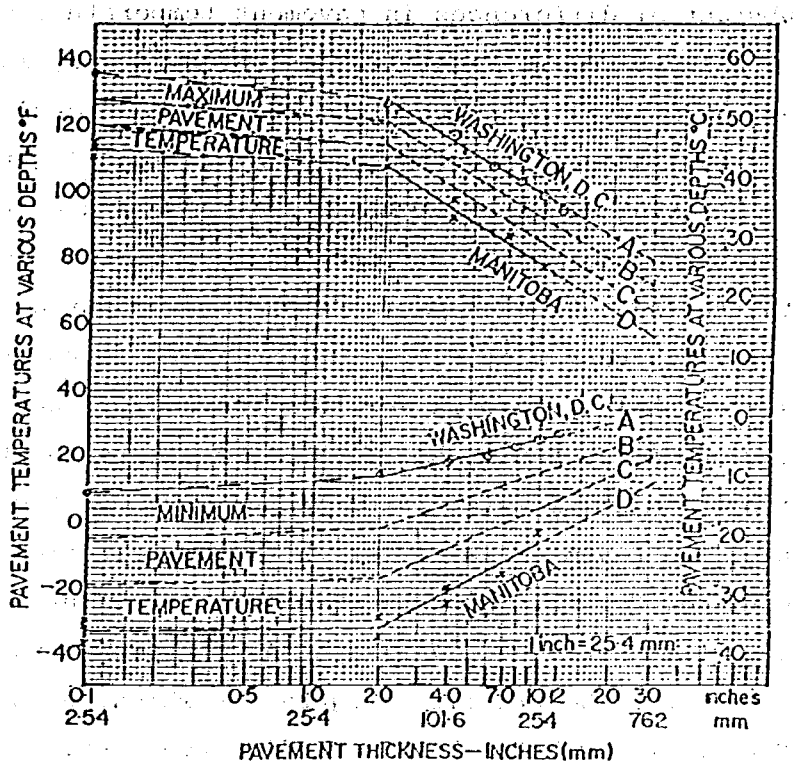

FIGURE 20 CURVE OF ANNUAL MAXIMUM AND MINIMUM TEMPERATURES WITH PAVEMENT DEPTH AT STE. ANNE, MANITOBA AND AT WASHINGTON, D.C.

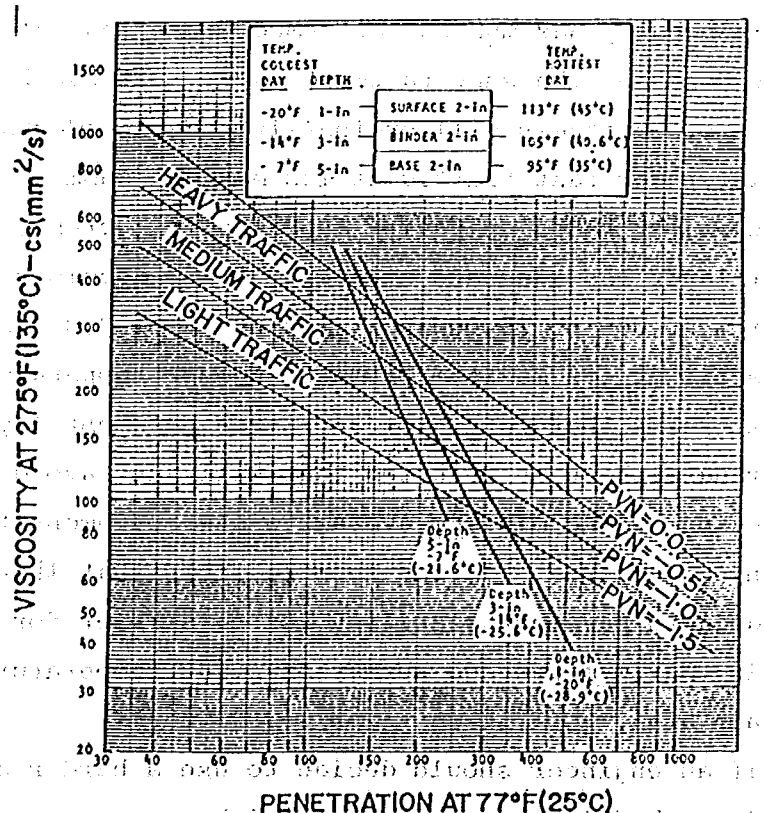

Figure 21 Illustrating the influence of higher minimum winter pavement temperatures with increasing pavement depth on paving asphalt selection in terms of combinations of temperature susceptibility (PVN) and penetration at 25°C for the avoidance of low temperature transverse pavement cracking.

These substantial differences in pavement temperature with pavement depth that are illustrated in Figure 20 have been largely ignored in the past, but they can be utilized with great benefit when designing asphalt surface, binder and base courses for heavy, medium and light traffic, and particularly when using asphalts of different temperature susceptibilities. That is, by utilizing the marked differences of pavement temperature with pavement depth, the selection of paving asphalts for surface, binder and base courses provides very wide scope for engineers to use paving asphalt temperature susceptibility to work for them instead of against them.

To briefly summarize the method to be used, it begins by using Figure 20 to establish the minimum anticipated winter pavement temperature for the average depth of the pavement layer for which the selection of a paving asphalt is required. For this minimum pavement temperature, Figure 18 provides a penetration at 25°C (77°F) and the corresponding PVN value for the paving asphalt for the traffic category for which the pavement is to be designed, PVN = 0.0 to -0.5 for heavy traffic, PVN = -0.5 to -1.0 for medium traffic, and PVN = -1.0 to -1.5 for light traffic. The modulus of stiffness of a pavement containing this asphalt can be obtained from Figures 7, 8 and 10 for the corresponding maximum summer temperature indicated by Figure 20 for the average depth of the same pavement layer. It can be shown that this pavement modulus of stiffness will ordinarily provide adequate stability for this pavement layer for fast traffic at its maximum summer temperature.

From calculations based on paving asphalts in current use, it appears that a pavement layer has adequate stability if it develops a modulus of stiffness of from 20,000 to 25,000 psi (138,000 to 172,000 kPa) under fast heavy truck traffic at its maximum pavement summer temperature.

In briefest terms therefore, for any given pavement layer and traffic category, a paving asphalt is selected that will just avoid low temperature transverse cracking at the minimum average winter pavement temperature for the layer in any given area. It can then be shown that this paving asphalt will provide adequate pavement stability for this layer for fast traffic at the corresponding maximum average summer pavement temperature for the same layer in the same area.

However, if an engineer should decide to use a harder asphalt than is provided by this method in order to obtain higher pavement stability for summer traffic, he must realize that at the same time he will have substantial low temperature transverse pavement cracking.

Three 2-inch layers, surface, binder and base course, are illustrated in Figure 21. To demonstrate this method for paving asphalt selection it will be applied to the choice of a paving asphalt for the 2-inch surface course layer in Figure 21, that is to carry heavy traffic at the maximum average summer pavement temperature of the pavement layer in some given area. The following successive steps are involved:

1. Measure or estimate the average minimum winter temperature for the 2-inch surface layer, which corresponds very nearly to the minimum temperature at a pavement depth of one inch on the abscissa of Figure 20. Assume that this minimum temperature at a pavement depth of one inch is -20°F (-28.9°C).
2. From Figure 18, read off for this minimum temperature of -20°F (28.9°C) the corresponding penetration at 25°C (77°F) for a PVN of 0.0 that will just avoid low temperature transverse pavement cracking. This penetration value is 165.
3. A minimum pavement temperature of -20°C (-28.9°C) at a pavement depth of one inch plots on the abscissa value of one inch in Figure 20 as a point that is 6/7 of the distance from Line D to Line C. For a pavement depth of one inch on the abscissa of Figure 20 read off from the top of the chart the corresponding maximum pavement temperature from a point that is 6/7 of the span from Line D to Line C. This maximum pavement temperature is 113°F (45°C).
4. Consequently, in this particular case, for this 2-inch surface course layer, its maximum summer temperature of 113°F (45°C) at a pavement depth of one inch, corresponds to its minimum winter temperature of -20°F (-28.9°C) at the same pavement depth, one inch.
5. For a temperature of 113°F (45°C), a loading time of 0.12 second, a penetration at 25°C (77°F) of 165 and a PVN of 0.0, the corresponding base temperature for the paving asphalt provided by Figure 7 is 42°C (107°F), its modulus of stiffness from Figure 8 is 18.9 psi, and the modulus of stiffness of a pavement containing this asphalt given by Figure 10 is 27,000 psi.
6. As previously indicated, a pavement modulus of stiffness of 27,000 psi, would provide quite adequate stability for fast heavy truck traffic at a summer pavement temperature of 113°F (45°C) at a pavement depth of one inch.
7. Therefore, a well designed and constructed asphalt surface course two inches thick, containing a paving asphalt of 165 penetration at 25°C

(77°F) and with a PVN of 0.0, would avoid low temperature transverse pavement cracking at a minimum winter temperature of -20°F (-28.9°C), at a pavement depth of one inch, and would at the same time provide adequate stability for fast heavy truck traffic at a corresponding maximum summer pavement temperature of 113°F (45°C) also at a pavement depth of one inch.

With respect to the the 2-inch binder course and 2-inch base course shown in Figure 21, the average minimum temperature for the 2-inch binder course can be approximated by the pavement temperature at a pavement depth of 3 inches, and for the 2-inch base course can be assumed to occur at a 5-inch pavement depth. If as was assumed for the above sample calculation, the minimum winter pavement temperature at a depth of one inch is -20°F (-28.9°C), Figure 20 shows that the corresponding minimum pavement temperature at a pavement depth of 3 inches occurs at 6/7 of the spread from Line D to Line C for an abscissa value of 3 inches. This minimum pavement temperature is -14°F (-25.6°C). Similarly, the corresponding minimum winter temperature at a pavement depth of 5 inches is -7°F (-21.7°C). By similar means, the corresponding maximum summer temperatures at pavement depths of 3 inches and 5 inches are 105°F (40.6°C), and 95°F (35°C), respectively.

The oblique temperature labelled lines in Figure 21 furnish guidance for the selection of paving asphalts that will avoid low temperature transverse pavement cracking at minimum pavement temperatures of -20°F (-28.9°C), -14°F (-25.6°C), and -7°F (-21.7°C). These lines demonstrate that because of the increasingly higher minimum pavement temperatures with increasing pavement depth in winter, correspondingly harder paving asphalts can be used at these pavement depths, one inch, 3 inches and 5 inches that will still avoid low temperature transverse pavement cracking at these higher pavement temperatures. The increasingly harder asphalts that can be used at these increasing pavement depths, also provide increasingly higher pavement stabilities for fast traffic at the corresponding maximum summer temperatures of 113°F (45°C), 105°F (40.6°C), and 95°F (35°C), respectively.

A note of explanation should be added with respect to the loading times applied at different pavement depths due to heavy trucks travelling at high speed. For a heavy truck travelling at 100 km (60 miles) per hour, the length of a tire contract area on a pavement surface is 10.5 inches when the loading time for a point on a pavement is 0.01 seconds. Assuming a 45° angle for spread of load with depth, the corresponding loading times at

pavement depths of one inch, 3 inches and 5 inches are 0.012, 0.016 and 0.02 second, respectively.

Table 2 summarizes data that can be obtained in a manner similar to that illustrated by the earlier sample calculation, for each of the 2-inch pavement layers, surface, binder and base course, for PVN values of 0.0, -0.5, -1.0 and -1.5 with their associated penetrations at 25°C (77°F), and for the corresponding maximum summer pavement temperatures, 113°F (45°C), 105°F (40.6°C), and 95°F (35°C). It should also be remembered, that every paving asphalt listed in Table 2 will provide pavements that will just avoid low temperature transverse pavement cracking at an average minimum pavement temperature of -20°F (-28.9°C) in the 2-inch surface layer.

The stippled areas on Figure 22 are based on data in Table 2, and indicate combinations of PVN and penetration at 25°C (77°F) for paving asphalts that can be selected for a pavement for a 2-inch surface course for heavy, medium, and light fast summer traffic at a pavement temperature of 113°F (45°C) at a pavement depth of one inch, which at the same time will avoid low temperature transverse pavement cracking at the corresponding minimum winter temperature of -20°F (-28.9°C) also at a pavement depth of one inch.

The pavement modulus of stiffness values shown on Figure 22 are adequate for 2-inch surface courses for heavy, medium and light traffic for a pavement temperature of 113°F (45°C) at a pavement depth of one inch. Figures 23 and 24 provide similar information for the 2-inch binder and 2-inch base courses, respectively.

Incidentally, for a pavement containing paving asphalt with <u>any</u> combination of PVN and penetration at 25°C (77°F) on the oblique line in Figure 22 indicating the hardest paving asphalts to be selected to avoid low temperature transverse pavement cracking at -20°F (-28.9°C), the corresponding pavement modulus of stiffness at this low pavement temperature is 300,000 psi (2,700,000 kPa) for a loading time of 20,000 seconds.

Based on the <u>average</u> pavement modulus of stiffness values taken from the right hand column of Table 2, for low, medium and high paving asphalt temperature susceptibilities, Figure 25 indicates the results from selecting paving asphalts of different temperature susceptibilities for surface, binder and base course layers for heavy, medium and light traffic. To conserve space in Figure 25, moduli of stiffness values for pavements containing paving asphalts of low temperature susceptibility, PVN = 0.0 to -0.5, carry an accompanying symbol A. Modulus of stiffness values for

TABLE 2

Influence of Maximum Summer Pavement Temperatures, Paving Asphalt Temperature Susceptibilities and Corresponding Penetrations at 25°C, On Pavement Moduli Stiffness For 2-Inch Surface, Binder and Base Course Layers.

Service Temp. (Max. Summer Pav. Temp.)	Time of Loading Seconds	Temperature Susceptibility	PVN	Pen. At 25°C	Base Temp. °C	Service Temp. Minus Base Temp., °C	Mod. of Stiff. Pav. Asphalt psi	Mod. of Stiff. Corresp. Pav. psi	Average Pav. Mod. of Stiff psi
				2-Inch Surface Course, Pavement Depth = one inch					
113°F (45°C)	0.012	Low	0.0	165	42	+ 3.0	18.5	28,000	24,500
113°F (45°C)	0.012	Medium	-0.5	215	38.2	+ 6.8	11.6	21,000	18,000
113°F (45°C)	0.012	High	-1.0	275	34.8	+10.2	6.9	15,000	12,750
113°F (45°C)	0.012		-1.5	345	32.2	+12.8	5.0	10,500	
				2-Inch Binder Course, Pavement Depth = 3 Inches					
105°F (40.6°C)	0.016	Low	0.0	143	43.6	- 3.0	33.7	40,000	35,000
105°F (40.6°C)	0.016	Medium	-0.5	182	39.9	+ 0.7	21.0	30,000	26,000
105°F (40.6°C	0.016	High	-1.0	225	36.9	+ 3.7	12.8	22,000	19,000
105°F (40.6°C)	0.016		-1.5	275	34.1	+ 6.5	8.6	16,000	
				2-Inch Base Course, Pavement Depth = 5 Inches					
95°F (35°C)	0.02	Low	0.0	123	45.4	-10.2	84.0	80,000	69,500
95°F (35°C)	0.02	Medium	-0.5	150	41.8	- 6.8	55.0	59,000	50,500
95°F (35°C)	0.02	High	-1.0	180	38.9	- 3.9	36.8	42,000	36,000
95°F (35°C)	0.02		-1.5	217	36.1	- 1.1	27.6	30,000	

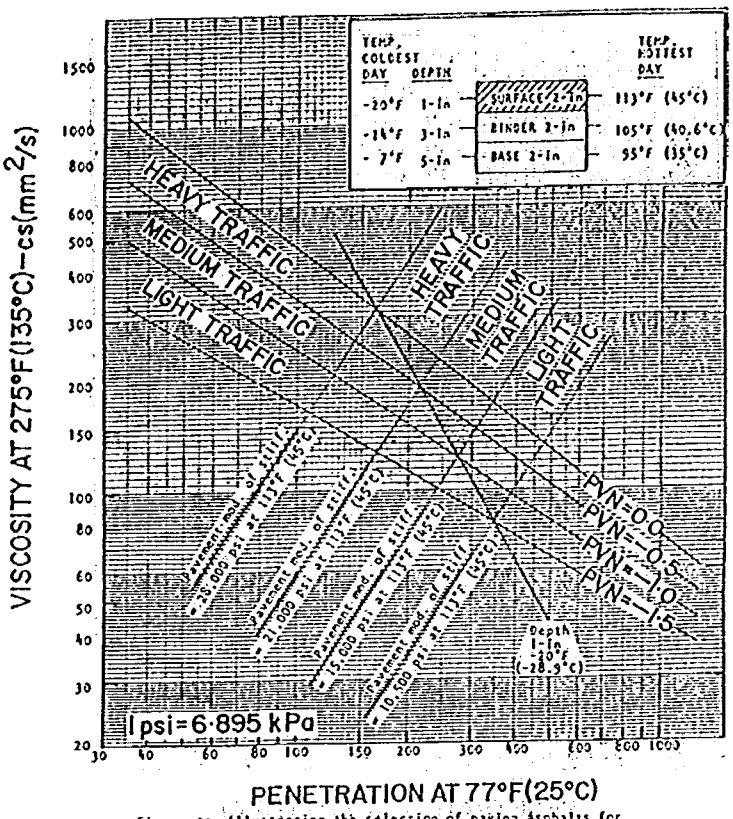

Figure 22 Illustrating the selection of paving asphalts for a 2-inch surface course for heavy, medium, or light traffic, that will avoid low temperature transverse pavement cracking and provide adequate pavement stability for warm weather traffic.

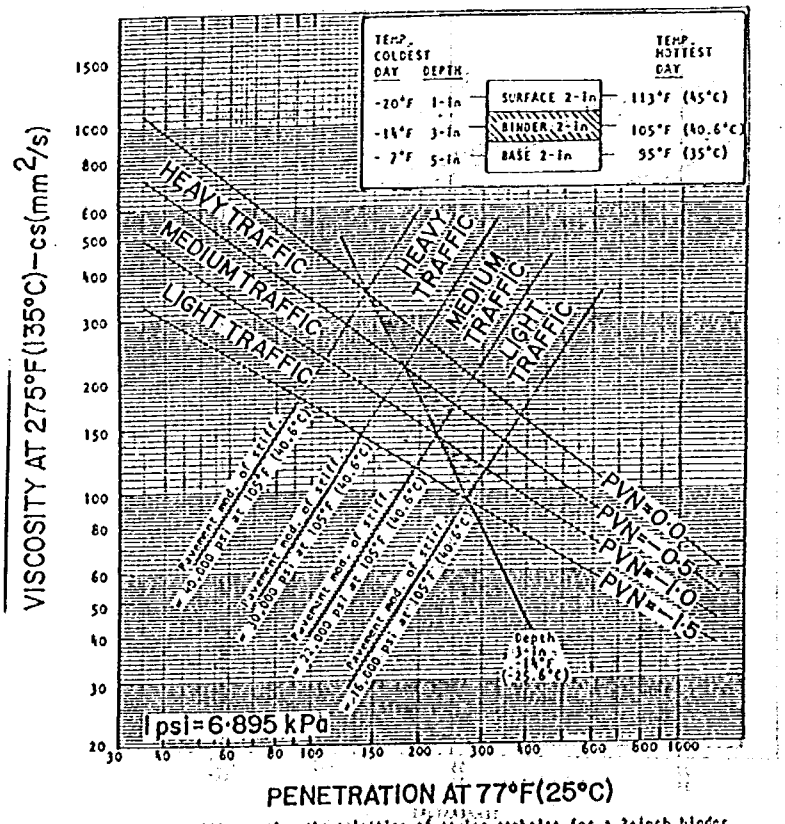

Figure 23 Illustrating the selection of paving asphalts for a 2-inch binder course for heavy, medium, or light traffic that will avoid low temperature transverse pavement cracking and provide adequate pavement stability for warm weather traffic.

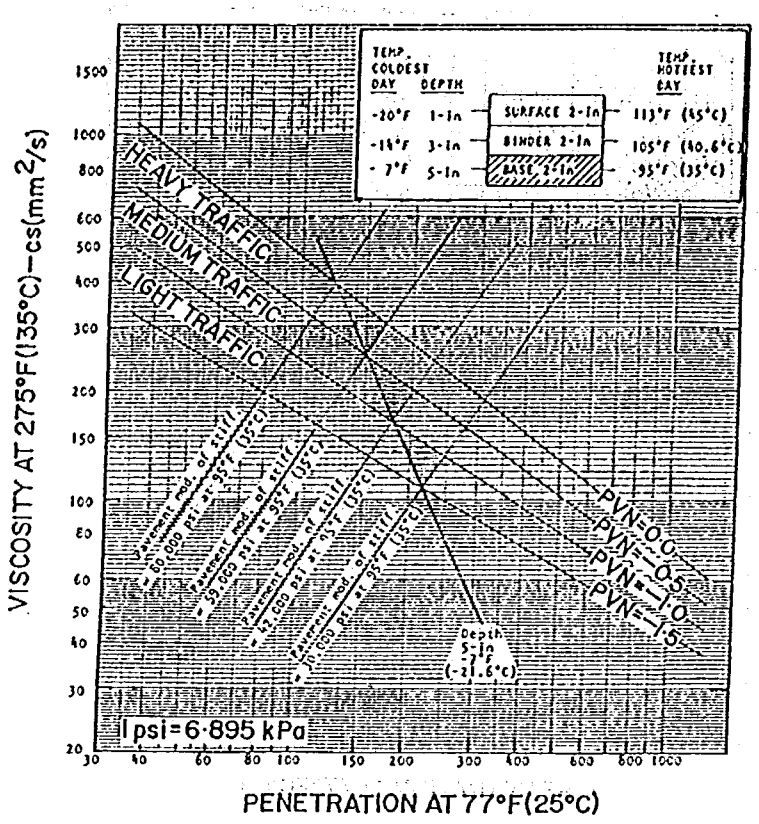

Figure 24 Illustrating the selection of paving asphalts for a 2-inch base base course for heavy, medium, or light traffic that will avoid low temperature transverse pavement cracking and provide adequate pavement stability for warm weather traffic.

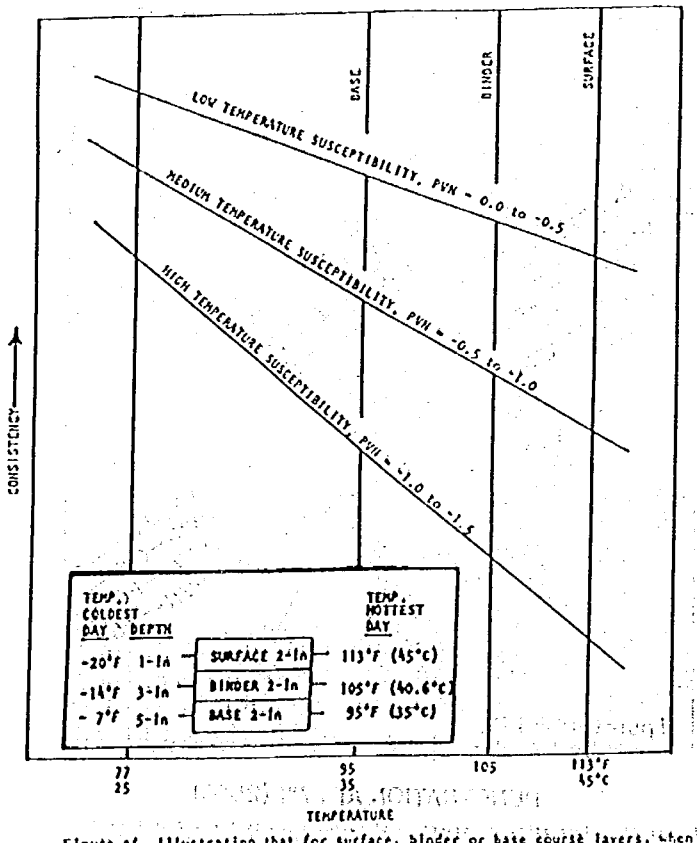

Figure 26 Illustrating that for surface, binder or base course layers, when paving asphalts are selected to avoid low temperature transverse pavement cracking, for warm weather traffic the same paving asphalts become increasingly softer with increasing temperature susceptibility

1	2	3	4	5	6	7
PAVEMENT LAYER	AVERAGE LAYER TEMPERATURE	AVERAGE PAVEMENT MODULUS OF STIFFNESS PSI				
		HEAVY TRAFFIC PAVEMENTS				
2-In Surface	113°F (45°C)	A* 24,500	A 24,500	A 24,500	A 24,500	
2-In Binder	105°F (40.6°C)	A 35,000	B* 26,000	B 26,000	C 19,000	
2-In Base	95°F (23°C)	A 69,500	B 50,500	C* 36,000	C 36,000	
		MEDIUM TRAFFIC PAVEMENTS				
2-In Surface	113°F (45°C)	B 18,000	B 18,000	B 18,000		
2-In Binder	105°F (40.6°C)	B 26,000	B 26,000	C 19,000	----	
2-In Base	95°F (35°C)	B 50,5000	C 36,000	C 36,000		
		LIGHT TRAFFIC PAVEMENTS				
2-In Surface	113°F (45°C)	C 12,750				
2-In Binder	105°F (40.6°C)	C 19,000	----	----	----	
2-In Base	95°F (35°C)	C 36,000				

*A - 2-In pavement layer contains paving asphalt of low temperature susceptibility, PVN = 0.0 to -0.5.

*B - 2-In pavement layer contains paving asphalt of medium temperature susceptibility, PVN = -0.5 to -1.0.

*C - 2-In pavement layer contains paving asphalt of high temperature susceptibility, PVN = -1.0 to -1.5.

FIGURE 25

SELECTING PAVING ASPHALTS OF DIFFERENT TEMPERATURE SUSCEPTIBILITIES FOR SURFACE BINDER AND BASE COURSE LAYERS

pavements containing paving asphalts of medium temperature susceptibility PVN = -0.5 to -1.0, have an associated symbol B, while the symbol C is attached to modulus of stiffness values for pavements containing paving asphalts of high temperature susceptibility, PVN = -1.0 to -1.5. The following comments are made on Figure 25:

(a) Heavy Traffic

For heavy traffic, the Columns 3, 4, 5 and 6 in Figure 25 demonstrate that:

(1) the strongest pavement structure , Column 3, would be provided by paving asphalt of low temperature susceptibility, PVN = 0.0 to -0.5, in all three 2-inch pavement layers, with an average pavement modulus of stiffness of 24,500 psi for the 2-inch surface course, 35,000 psi for the 2-inch binder course, and 69,500 psi for the 2-inch base course. Whether the high pavement modulus of stiffness of 69,500 psi is useful or wasteful would require a pavement structural design evaluation based on elastic theory, which is outside of the scope of this paper. For good engineering, one should, for example, avoid using a 12 X 12 inch beam for a catwalk if a 4 X 12 inch plank is adequate.

(2) A pavement of adequate stability, Column 4, would be provided by a paving asphalt of low temperature susceptibility, PVN = 0.0 to -0.5, for the 2-inch surface course with a modulus of stiffness of 24,500 psi, a paving asphalt of medium temperature susceptibility, PVN = -0.5 to -1.0, for the 2-inch binder course with a pavement modulus of stiffness of 26,000 psi, and for the 2-inch base course with a pavement modulus of stiffness of 50,500 psi.

(3) Adequate pavement stability, Column 5, would also be furnished by paving asphalt of low temperature susceptibility, PVN = 0.0 to -0.5, in a 2-inch surface course having a pavement modulus of stiffness of 24,500 psi, a paving asphalt of medium temperature susceptibility, PVN = -0.5 to -1.0, for the 2-inch binder course that develops a pavement modulus of stiffness of 26,000 psi and a paving asphalt of high temperature susceptibility, PVN = -1.0 to -1.5, for the 2-inch base course with its pavement modulus of stiffness of 36,000 psi.

(4) On the conservative basis, that for design requirements, each underlying layer must have a modulus of stiffness at least equal to that of the surface course when this modulus of stiffness of the surface course is just adequate, the combination of 2-inch surface, binder and base course layers in Column 6 at the top of Figure 25, would be unsatisfac-

tory because if a paving asphalt of high temperature susceptibility, PVN = -1.0 to -1.5, were used for the 2-inch binder course, its pavement modulus of stiffness would be only 19,000 psi, which would be less than the pavement modulus of stiffness of 24,500 psi provided by the 2-inch surface course layer.

(5) Therefore, combinations of 2-inch surface, binder and base course pavements containing paving asphalts with the temperature susceptibilities (PVN values) and with the associated pavement moduli of stiffness shown in Columns 3, 4 and 5 in Figure 25, would avoid low temperature transverse pavement cracking at a minimum winter pavement temperature of -20°F (-28.9°C) in the 2-inch surface layer, and would at the same time provide adequate stability for fast heavy truck traffic at the corresponding maximum pavement summer temperature of 113°F (45°C) at a pavement depth of one inch.

(b) Medium Traffic

(1) For medium traffic, as might be expected, pavements containing paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, in all three 2-inch layers, Column 3, provide adequate pavement stability. This is indicated by pavement moduli of stiffness of 18,000 psi for the 2-inch surface course, 26,000 psi for the 2-inch binder course, and 50,500 psi for the 2-inch base course. Again, whether or not the high pavement modulus of stiffness of 50,500 psi developed by the 2-inch base course can be fully utilized requires a pavement structural analysis.

(2) A combination, Column 4, of 2-inch pavement layers containing paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, for the 2-inch surface course with its pavement modulus of stiffness of 18,000 psi, and for the 2-inch binder course with its pavement modulus of stiffness of 26,000 psi, but with the 2-inch base course containing paving asphalt of high temperature susceptibility, PVN = -1.0 to -1.5, which provides a pavement modulus of stiffness of 36,000 psi, would provide a pavement structure of adequate stability for fast medium traffic at the given summer weather pavement temperatures.

(3) The combination of the three 2-inch pavement layers in Column 5 in Figure 25, would also provide adequate pavement stability for medium traffic. This combination includes a 2-inch surface course layer containing paving asphalt of medium temperature susceptibility, PVN = -0.5 to -1.0, with a modulus of stiffness of 18,000 psi, a 2-inch

binder course and a 2-inch base course both containing paving asphalt of high temperature susceptibility, PVN = -1.0 to -1.5, and developing pavement moduli of stiffness of 19,000 psi for the 2-inch binder course, and 36,000 psi for the 2-inch base course.

(4) Consequently, the combinations of 2-inch pavement layers, surface, binder and base courses, containing paving asphalts of medium and high temperature susceptibilities, with their associated modulus of stiffness values, indicated for medium traffic in Figure 25, would avoid low temperature transverse pavement cracking at an average minimum winter temperature in the 2-inch surface layer of -20°F (-28.9°C), and at the same time would provide adequate pavement stability for medium traffic at the corresponding maximum average summer temperature of 113°F (45°C) in the 2-inch surface course.

(c) Light Traffic

(1) As indicated by Column 3 in Figure 25, the use of paving asphalt of high temperature susceptibility, PVN = -1.0 to -1.5, in all three 2-inch pavement layers would provide adequate warm weather pavement stability for light traffic. This is indicated by average moduli of stiffness of 12,750 psi for the 2-inch surface layer, 19,000 psi for the 2-inch binder course, and 36,000 psi for the 2-inch base course.

(2) As a result, 2-inch surface, binder and base course layers each containing paving asphalt with a high temperature susceptibility, PVN = -1.0 to -1.5, and with their associated modulus of stiffness values given in Figure 25, would avoid low temperature transverse pavement cracking at an average minimum winter temperature in the 2-inch surface course of -20°F (-28.9°C), and would provide at the same time adequate pavement stability for light traffic at the corresponding maximum average summer temperature of 113°F (45°C) in the 2-inch surface layer.

The paving mixture designs illustrated in Figure 25 pertain to 2-inch pavement layers and to a minimum average winter temperature of -20°F (-28.9°C) in a 2-inch surface course and to the corresponding maximum average summer pavement temperature of 113°F (45°C) in the same 2-inch surface layer.

The same method can be applied to other combinations of pavement layer thickness and to other corresponding pairs of minimum and maximum average pavement temperatures in the surface, binder and base course layers.

indicated by Figure 20, that may be considered or required for asphalt pavement design for heavy, medium or light traffic in any area.

5. Why Paving Asphalts of Lowest Temperature Susceptibility Provide Layers With Highest Summer Pavement Moduli of Stiffness in Cold Climates

Column 3 of Figure 25 demonstrates that the highest pavement modulus of stiffness for each pavement layer, surface, binder, and base course, is developed under fast heavy traffic at summer temperatures when paving asphalts of low temperature susceptibility, PVN = 0.0 to -0.5, are employed. Figure 17 indicates why this happens. For the oblique line labelled -20°F (-28.9°C), which guides the selection of the hardest paving asphalts that will avoid low temperature transverse pavement cracking, paving asphalts of low temperature susceptibility, PVN = 0.0 to -0.5, have the associated lowest penetrations at 25°C (77°F), 165 to 215, paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, are accompanied by penetrations at 25°C (77°F), 215 to 275, while for paving asphalts of high temperature susceptibility, PVN = -1.0 to -1.5, the corresponding penetrations at 25°C (77°F) are 275 to 345. Therefore, the penetrations at 25°C (77°F) for these paving asphalts become higher and higher (softer and softer) as their temperature susceptibilities become higher and higher. Since these penetrations at 25°C (77°F) are also employed for the same pavements for summer traffic, when all other factors are equal, pavement stability (pavement modulus of stiffness) decreases (becomes lower and lower) as paving asphalt penetration at 25°C (77°F) increases (becomes softer and softer). Therefore, for any given pavement layer, for fast summer traffic the highest pavement modulus of stiffness is associated with lowest paving asphalt temperature susceptibility (PVN = 0.0 to -0.5) because this PVN range in turn is associated with the hardest permissible paving asphalt (lowest permissible penetration at 25°C (77°F).

This is also illustrated in Figure 26, which is a sketch showing that as indicated by Figures 17 and 18, to avoid low temperature transverse pavement cracking in any given pavement layer at the minimum winter pavement temperature in any area, paving asphalts of low temperature susceptibility (high PVN values, 0.0 to -0.5), are associated with harder (low) penetrations at 25°C (77°F), paving asphalts of medium temperature susceptibility (medium PVN values, -0.5 to -1.0) are paired with softer (higher) penetrations at 25°C (77°F), while paving asphalts of high temperature susceptibility (PVN values -1.0 to -1.5) have correspondingly still softer (still

higher penetrations at 25°C (77°F).

Summarized a little differently, as shown by Figure 26, within each pavement layer, surface, binder or base course, as the temperature susceptibility of the paving asphalt becomes higher and higher, a softer and softer (higher and higher penetration at 25°C) paving asphalt must be selected to avoid low temperature transverse cracking on a cold day in winter, Figure 17. These softer and softer paving asphalts result in lower and lower pavement stabilities for fast traffic at summer temepratures, Figures 22, 23 and 24. On the other hand, for an overall pavement structure consisting of surface, binder, and base course layers, when the paving asphalt temperature susceptibility is held constant either low, medium or high, Figure 26, since during a cold winter day, deeper pavement layers are at a higher temperature than the surface layer, Figure 20, harder and harder (lower and lower penetration at 25°C) paving asphalts can be used for the binder and base course layers respectively, than for the surface layer, and still avoid low temperature transverse pavement cracking in each layer, Figure 21. In turn, these harder and harder paving asphalts result in higher and higher pavement stabilities with pavement depth for fast traffic at summer temperatures, Figure 25.

Consequently, the information contained in Table 2, Figure 25, and Figure 26, provides useful guidance to assist engineers to make asphalt temperature susceptibility work for them when selecting paving asphalts for asphalt surface, binder and base courses.

V GENERAL COMMENTS

1. It is an interesting and very useful feature of this method for paving asphalt selection, that the paving asphalt combination of temperature susceptibility, PVN, and penetration at 25°C (77°F) that just avoids low temperature transverse pavement cracking at the minimum winter pavement temperature in a pavement layer in any cold region, at the same time provides adequate stability for the pertinent category of traffic at the maximum summer temperature in the same pavement layer in the same region.
2. At the present time, many paving asphalt specifications permit the use of only low or relatively low temperature susceptibility regardless of the traffic category, high, medium, or light, that a pavement is expected to carry. This paper indicates that paving asphalts of low temperature susceptibility, PVN = 0.0 to -0.5, are required only

for pavements for heavy traffic, and that when selected on the basis of the method described in the paper, paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, will provide well designed and constructed pavements with adequate stability for medium traffic, and that pavements made with paving asphalts of high temperature susceptibility PVN = -1.0 to -1.5, will furnish sufficient stability for light traffic.

3. The mileage of roads carrying heavy traffic is in a very definite minority. Therefore, the requirement for paving asphalts of low temperature susceptibility for pavements for heavy traffic is also a minor percentage of the total annual paving asphalt demand.

4. By far the greatest requirement for paving asphalt is for roads for medium and light traffic. When selected in accordance with the procedure described in this paper, paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, will provide pavements with ample warm weather stability for medium traffic, and pavements with sufficient stability for light traffic will be supplied by paving asphalts of high temperature stability PVN = -1.0 to -1.5.

5. In regions blessed with plentiful supplies of paving asphalts of low temperature susceptibility, no problem of choice of paving asphalt oridnarily exists for pavements for heavy, medium or light traffic, or for surface, binder, or base course. However, these regions are very often islands that are surrounded by areas where the only paving asphalts that can be manufactured, or are otherwise available, are those of medium and high temperature susceptibility.

6. In regions where paving asphalts of low temperature susceptibility, PVN = 0.0 to -0.5, required for pavements for heavy traffic are scarce, several means are available for modifying the temperature susceptibilities of paving asphalts of medium and high temperature susceptibility that may be plentiful, to make them acceptable for pavements for heavy traffic. Meyer, Hignell, Kennepohl and Haas (15) have shown that the addition of substantial quantities of sulphur to paving asphalt can be used for this purpose. Khosla and Ostermeyer (16) have indicated that residues from high float emulsions have lower temperature susceptibilities than those of the original asphalt bases for these emulsions. Gaw, Burgess, Young and Fromm (17) and Clark and Culley (18), have furnished some indication that the temperature susceptibilities of paving asphalts of medium and high temperature susceptibility can

be reduced by air blowing. The addition of polymers for this purpose has also been suggested.

7. There is considerable evidence to show that there is little or no difference in pavement service life that can exceed 20 years, when properly selected paving asphalts with a wide range of temperature susceptibilities are chosen for well designed and constructed dense graded asphalt paving mixtures.

8. While the method described in this paper begins by measuring or estimating the minimum winter temperature expected during a pavement's service life at some pavement depth, it is recognized that the method could also begin with the maximum summer temperature measured under the same conditions at some pavement depth. The corresponding minimum winter temperature at the same pavement depth could then be determined from Figure 20, after which the procedure as described could be followed step by step. In this case however, as shown in Figure 13 of reference (14), because of the lag of from one to two hours between the time when the maximum surface temperature and the times when the maximum temperatures at different depths occur, it is suggested that if used, the maximum summer temperature employed with this design method should be the maximum summer temperature measured at some pavement depth like 2 inches or 4 inches.

SUMMARY

1. Paving asphalt temperature susceptibility is illustrated and defined.
2. The background of a method for the determination of paving asphalt temperature susceptibility in terms of pen-vis number (PVN) values is described.
3. An example is included to demonstrate how engineers can make paving asphalt temperature susceptibility work either for them or against them when designing asphalt pavements in a warm climate that is free from frost.
4. An example is also included to show how engineers can make paving asphalt temperature susceptibility work either for them or against them when selecting paving asphalts for pavements that are to avoid low temperature transverse pavement cracking in a cold climate.
5. A method is described for the selection of paving asphalts for pavements for heavy, medium and light traffic, that will simultaneously avoid low temperature transverse pavement cracking in winter, and

provide adequate pavement stability, and resistance to pavement rutting in summer.

6. This method indicates that harder paving asphalts of low temperature susceptibility, PVN = 0.0 to -0.5, should be selected for pavements for heavy traffic, while somewhat softer paving asphalts of medium temperature susceptibility, PVN = -0.5 to -1.0, could be chosen for pavements for medium traffic, and still softer paving asphalts of high temperature susceptibility, PVN = -1.0 to -1.5, for pavements for light traffic.

7. Based on measurements of pavement temperature with pavement depth by The Asphalt Institute at Washington, D.C., and by the Manitoba Department of Highways and Transportation jointly with Shell Canada Limited at Ste. Anne, Manitoba, a chart has been developed that indicates the minimum winter pavement temperature at any pavement depth in any given area, and the corresponding maximum summer pavement temperature at the same pavement depth in the same area when the minimum pavement temperature at any given pavement depth is known or is estimated.

8. Primarily on the basis of this chart, a method is described to enable paving asphalts to be selected for pavements that will avoid low temperature transverse pavement cracking at the minimum winter pavement temperature that has been measured or estimated for any given region, and that will at the same time provide pavements of adequate stability for the same pavement layer, for fast traffic in any category, heavy, medium or light, at the corresponding maximum summer temperature in the same region.

9. It is demonstrated that adequate pavement stability for fast heavy, medium, or light traffic at maximum summer pavement temperatures, with associated avoidance of low temperature transverse pavement cracking at corresponding minimum winter pavement temperatures, can be achieved by the proper selection of combinations of paving asphalts of different temperature susceptibilities for surface, binder and base course layers.

10. This method for selecting proper paving asphalts of different penetrations at 25°C and temperature susceptibilities for surface, binder, or base courses, that will avoid low temperature transverse pavement cracking at minimum winter pavement temperatures, and at the same time will provide adequate pavement stability at maximum summer pavement

temperatures for heavy, medium, or light traffic, requires the use of only Figures 20, 18, 7, 8 and 10 in that order.

When the minimum winter pavement temperature at some pavement depth has been measured or estimated, Figure 20 can provide the minimum winter pavement temperature and corresponding maximum summer pavement temperature for this or any other pavement layer.

From Figure 18, for the minimum winter pavement temperature for each pavement layer, corresponding values for PVN and penetration at 25°C (77°F) for paving asphalts that just avoid low temperature transverse pavement cracking are indicated, from which a paving asphalt with a combination of PVN and penetration at 25°C (77°F) satisfactory for the specified traffic category, heavy, medium, or light, can be selected. The base temperature for this selected paving asphalt can be obtained from Figure 7. Figure 8 indicates the modulus of stiffness for the selected paving asphalt for the time of loading at a point on the pavement by fast traffic, and for the maximum summer temperature for each layer that has been obtained from Figure 20. From Figure 10, the modulus of stiffness of a pavement layer containing this paving asphalt can be determined for the same temperature and time of loading.

11. To summarize very briefly, it has been demonstrated how engineers can make paving asphalt temperature susceptibility work either for them or against them when selecting paving asphalts for:

(a) asphalt pavements of adequate stability for traffic in warm climates free from frost,

(b) asphalt pavements that will avoid low temperature transverse pavement cracking in cold freezing climates,

(c) asphalt pavements of adequate stability for heavy medium, or light traffic at maximum summer pavement temperatures that will at the same time avoid low temperature transverse pavement cracking at corresponding minimum winter pavement temperatures.

(d) asphalt pavements for surface, binder and base courses that will avoid low temperature transverse pavement cracking at minimum winter pavement temperatures, and that will at the same time provide adequate pavement stability for fast heavy, medium, or light traffic at corresponding maximum summer pavement temperatures.

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Rut Wear Measurements on an Epoxy-Asphalt Bridge Deck Surface in Vancouver, British Columbia

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ABSTRACT

Epoxy extended asphaltic cement concretes have certain unique properties for resisting wear from vehicles using rubber tires, steel studded tires or snow chains. Our experience with three different epoxy asphalt systems on one bridge is documented. This bridge deck surface is adjacent to a mountain side community which has periodic snow falls requiring vehicle tire traction devices. Primarily the success of a 35mm thick epoxy extended asphaltic concrete bridge wearing surface is assessed over a nine year period. The traffic volumes average over 60,000 vehicles per day. The pessimistic prediction traffic rutting indicates the 12mm terminal depth value of traffic caused rutting will be acheived within ten years of placement. This early linear extrapolation of existing wear rate curves may be overly pessimistic. A field trial of repair to a localized excessive rut wear section indicates that excellent results can be acheived if proper control is excercised during field repair.

RUT WEAR MEASUREMENTS ON AN EPOXY-ASPHALT BRIDGE DECK SURFACE IN VANCOUVER, BRITISH COLUMBIA

by

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B.C. Ministry of Transportation and Highways

INTRODUCTION

The Lions Gate Bridge connects a large residential community located on an ocean side mountain to the downtown central business district. The majority of the traffic using this bridge are cars, busses and some service vehicles. Since its original construction in 1938, the Portland Cement Concrete deck surface has worn down to the level where the grid steel on the main bridge span is exposed, and the reinforcement steel on the north bridge approach was in danger of deterioration. During the 1960's and early 1970's the increased usage of vehicle traction devices for overcoming lack of traction on snow and ice, greatly increased the wear of paved surfaces.

Because the bridge structure could not tolerate large increases in dead loading a relatively thin layer (10 to 15mm) of coaltar epoxy and natural aggregate was applied to the main bridge span in the summer of 1963. This material was mostly worn away in December 1964 during several days of heavy snowfall apparently caused by the "flailing" action of an abnormal use of car tire chains (photo 1). Subsequent laboratory tests with a flailing chain device duplicated the amount of wear; and indicated that normal hot mix asphaltic cement concrete would suffer approximately

twice the amount of rut wear than would epoxy-aggregate mixtures. This rate of wear, plus other concerns about this bridge's traffic volume capacity, led to the decision to have a field trial of replacement of the entire bridge deck. The north bridge approach (670 metres long) was replaced in 1975 with an orthotropic steel plate bridge deck covered with an average wearing surface thickness of 35mm of "Concresive Asphalt". This "Concresive Asphalt" is a special epoxy extended hot mix asphaltic cement concrete designed by the United States firm of Adhesive Engineering.

With the exception of the first two panels applied which suffered problems during application, the remainder of the approach viaduct using the epoxy extended asphaltic cement concrete above the orthotropic steel deck has proven to be a very effective product. The wheel paths of the curb lane of the first two poor panels were resurfaced with epoxy concrete in 1978. The epoxy concrete was a mixture of cold applied (Guardkote 250) epoxy and two different natural aggregate materials. Again rainfall during placement caused problems with one half of the repair work, with resulting increased wear on the improperly cured epoxy concrete.

The rate of wear measurements on both the good and bad bridge redecking materials provides the central thread for this paper. These rate of wear measurements are supplemented by other laboratory studies which relate to the overall assessment of the success to date of this system of bridge deck replacement.

PAST EXPERIENCE

It was originally considered that a thin layer of a highly resistant wearing surface would restore both the smooth ride and the original friction between the bridge deck and the vehicle tires. A thin layer of 10 to 15mm thickness would not overload the bridge design loadings. Several types of this wearing surfaces were considered, including natural aggregates with asphaltic binders, natural aggregates with epoxy binders, very hard synthetic aggregates with epoxy binders, and epoxy bound Portland Cement Concrete.

The thin Portland Cement Concrete overlay was not used because of less than satisfactory success by other agencies. While the hard carborundum type aggregates bonded with an epoxy appeared to have many advantages; the cost, limited availability and brittleness mitigated against its use. Previous maintenance experience with asphaltic cements and natural aggregates overlays on this bridge indicated problems with adhesion to the concrete deck, and with shoving during the hot weather. On the basis of past successes by other users it was decided to use a two component coal tar epoxy to bond the natural hard aggregate to the Portland Cement concrete and steel grid main bridge spans - and also to the Portland Cement concrete north viaduct. The two components of the coal tar epoxy were mixed, trowelled to an average depth of 3mm, and covered with an excess of natural birds eye sized (10mm to 2mm) aggregate. This mixture was then heat cured with heat lamps for 2 hours at an average temperature of 50 ± 5° C. Except for some equipment and manpower training problems at the beginning, of the project, this "coal tar chip seal" resurfacing was successfully applied in September 1963 and appeared to be very effective

until December 1964. At that time a unusually intensive and week long snowfall forced most vehicle owners to install snow chain traction devices for one week continually. This very unusual flailing-impact type of wear removed most of the epoxy bonded chip seal in December 1964. Photo 1 shows the results of this wear. Since it was impossible to exactly predict the next occurrence of an unusually heavy snowfall, it was considered that use of this thin resurfacing was not a wise statistical gamble. This possibility of another unusual snowfall, plus the need for a greater bridge deck width to reduce traffic congestion led to the next decision which was the replacement of the entire bridge deck, one section at a time. The north viaduct of the bridge was selected as the full scale test for replacement of the reinforced concrete deck with a wider orthotropic steel plate deck overlain with 35mm thickness of a special hotmix machine lard asphaltic cement concrete.

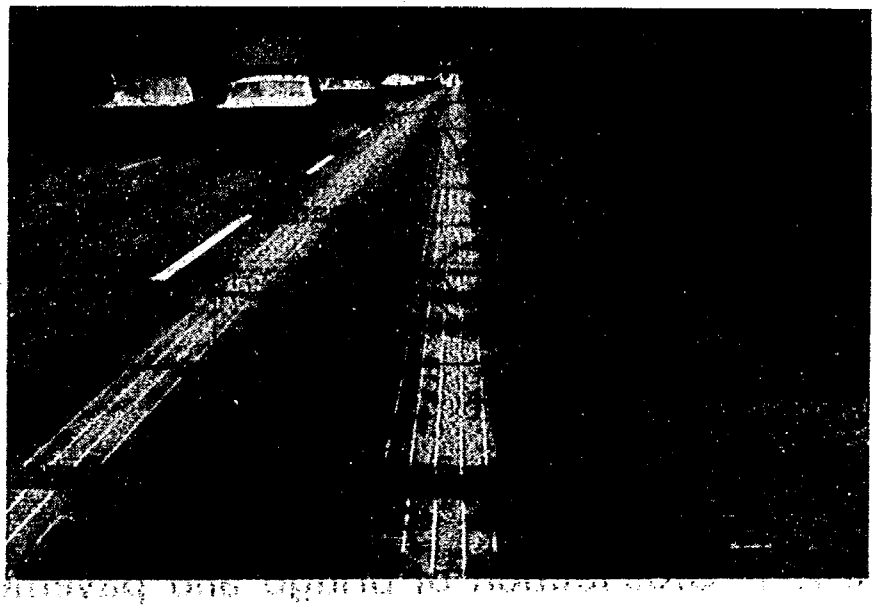

Photo 1 - Two years wear on 10 mm layer of Coal Tar Epoxy and Natural Aggregate

After detailed engineering studies it was decided to replace the bridge deck of the north viaduct with a prepaved steel plate orthotropic type, one section at a time. The wearing surface of the bridge deck was designed by Adhesive Engineering of California and consisted of a special epoxy extended asphalt binder mixed with natural local aggregates. This special wearing surface was applied to the orthotropic steel plate by normal road paving methods. Several sections of the full width bridge deck were placed on the ground surface below the existing bridge and paved in one continuous operation.

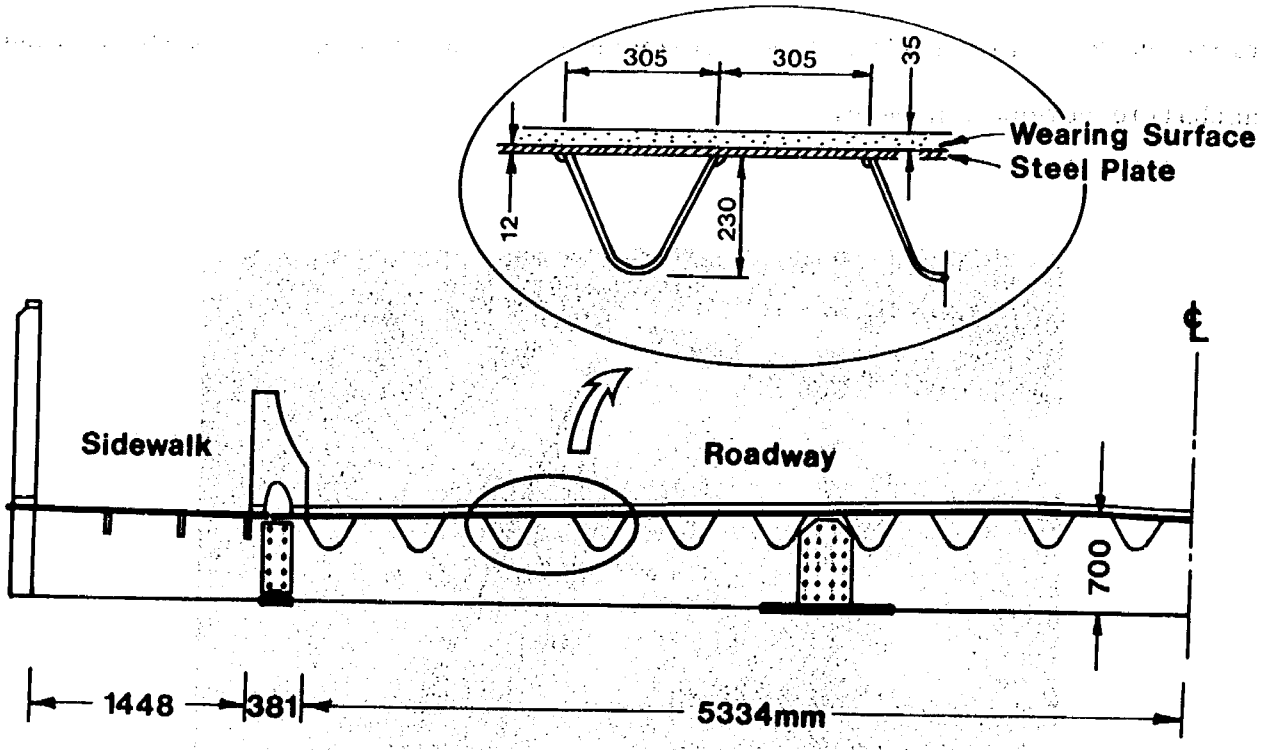

FIGURE 1: Crossection of bridge and pavement

The materials used on the new bridge deck are described in the following paragraphs.

The mineral aggregates used was made of a crushed local hard aggregate that met the following gradation:

Sieve Size	Percent Passing
3/8 inch	100
#4	75 to 95
#10	50 to 70
#20	36 to 55
#40	26 to 44
#80	14 to 28
#200	7 to 14

The epoxy extended asphalt binder consisted of two components. Component A was a diepoxy resin. Component B was a homogeneous mixture of petroleum derived asphalt and of epoxy resin hardeners.

The orthotropic steel deck was cleaned to white metal and immediately sprayed with inorganic zinc rich paint. A thermosetting two component epoxy was next applied to act as a bonding material for the following epoxy-extended asphaltic cement hot mix. It should be noted that this epoxy extend asphaltic cement hot mix stiffens with cooling as would a normal asphaltic concrete - but the epoxy component takes several weeks to complete its hardening, which provided it high strength and flexibility.

The prepaved replacement bridge deck sections (panels) were successfully replaced in the late summer of 1975. Only the first two panels (#1 and #2) had a lower quality pavement surface due to startup problems. It was decided to leave them in place and closely monitor any problems. Subsequent monitoring showed rapid wheel path wear and longitudinal cracking in the first two panels. In early September 1978

the two wheel plaths of panels 1 and 2 were replaced. This was both a repair under the contractors original guarantee, and a field test of the proposed repair method of the worn areas of the original paving. The repair consisted of machine planing a 600 mm wheel path width by removing about 15 mm of the remaining 22 mm thickness of original epoxy extended asphaltic cement concrete, and then replacing the gap with a mixture of local aggregates and fast curing cold applied epoxy. The epoxy was a two component oil extended epoxy marketted under the trade name "Guardkote 250".

The aggregates sources were different for each of the two wheel paths. The right (west) wheelpath of panels 1 and 2 used Jervis Inlet crushed gravel. The Jervis Inlet material is described as having 45% fractured faces and consisting of 52% porphyritic type volcanic material, 19% unweathered granitics, 6% slate; with the remainder consisting of weathered materials of the three types mentioned previously. The left wheel path of panels 1 and 2 used Colwood Bay crushed gravel. This material is described as having 94% fractured faces with 53% pophyritic type volcanic material, 34% unweathered granitics, 9% metamorphic/sedimentary materials, with the remainder weathered products of the foregoing. The final gradation of both products was as follows:

Sieve Size	Percentage Passing	
	Jervis(West/right)	Colwood(East/left)
1/2"	100	100
1/4"	99	99
# 4	88	87
# 8	60	57
#16	54	51
# 30	49	47
# 50	36	32
#100	20	18
#200	11	10

The premixed epoxy binder was added at a rate of 6.5% of weight of total mix.

The repairs of pavement in the west/right wheel path were completed in one night without any problems. The repairs of the east/left wheel path encountered rainy weather and were completed in less than ideal conditions.

PAVEMENT WEAR MEASUREMENTS

On the basis of theoretical computations and laboratory analog bending tests, it was decided that only an average of 12 mm of pavement wear would be acceptable. At this wear depth, remedial action would be necessary to avoid pavement failure through the action of longitudinal cracking. Therefore four transverse lines were established, one on each panel (bridge section). Each line consisted of three metal reference studs welded to the steel deck onto which a reference straight edge could be placed. A dial guage micrometer was used to measure the increase in distance between the reference straight edge and the wearing pavement surface for each wheel path. This gave us eight areas of wear. The wear readings were taken twice a year - in the spring (May) and in the fall (Sept.). This was based on the concern that various forms of vehicle wheel traction devices would greatly increase the pavement wear during winter snowfalls. Figure 2 graphically presents the amount of pavement wear. The line for wear readings starting in September 1976 represents the typical wear rate on the original epoxy extended asphaltic concrete surface. The steepest line starting in September 1978 shows the worst rate of wear in the wheelpath repaved with Colwood aggregate and epoxy (repair performed in wet weather conditions). The flattest line starting in September 1978 is

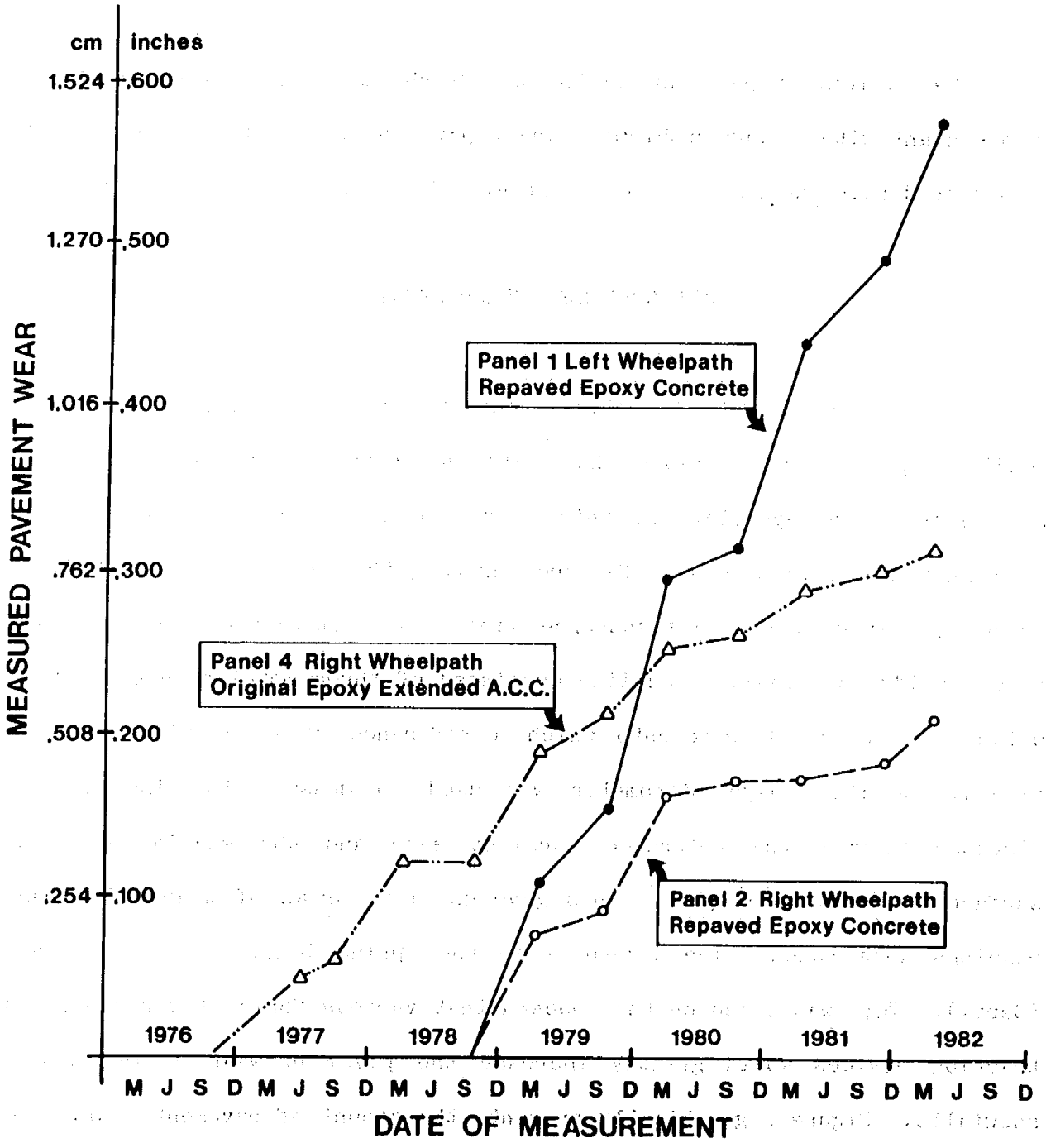

FIGURE 2: Comparison of wear rates for Epoxy – Asphalt – Aggregate Systems.

the best (lowest) rate of wear in the wheel path repaved with Jervis aggregates and epoxy (repair performed in dry weather conditions).

SUMMARY

After nine years of high volume traffic over the 670 metre long north viaduct, the original epoxy extend asphaltic concrete is providing excellent service.

The wear rate on epoxy-aggregate wheel path repairs varies from being equal to the average on the original epoxy extended asphaltic concrete; up to double that value. The latter would approximate wear on normal asphaltic cement concrete under extreme use of winter tire traction devices.

On the basis of available information, the extremely high rate of wear on panel I is very likely due to placing the wheel path repairs during rain. It very likely represents a worst case wear rate for a poorly mixed or poorly constructed epoxy concrete.

On the basis of the other wheel path wear rate it appears that excessive wear in the wheel path can be effectively repaired by machine planing out about a 600 mm wide trench in the worn area, and repaving this area with epoxy concrete.

The extrapolation of existing wear rates indicates that both the epoxy extended asphaltic concrete and the epoxy concrete paving on this bridge have a working life of 10 to 15 years. This is based on a 35 mm thick epoxy modified pavement not being worn below a 23 mm residual thickness.

The firm that applied the original epoxy extended asphaltic concrete redecking, predicted that the wear rate would lessen with time as the weaker surface aggregate was worn away. The wear rate curves appear to confirm this theory.

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On a Prediction Method of the Rutting Progress of Portland Cement Concrete Pavement in Cold Areas

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ABSTRACT

Portland Cement concrete pavements (P.C.C.P) in Japan in now generally designed at the view point of fatigue tensile strength of plain concrete slabs. However in cold areas the life of P.C.C.P. is often determinated by the rutting caused by wearing action of spiked tires before P.C.C.P. are cracked by tensile fatigue. This wearing action is not easily predicted because of various conditions; concrete materials, mix proportion, traffic conditions and so on.

In this paper, a prediction method of the rutting progress of P.C.C.P., based on the results of both various kinds of wearing tests with a traffic simulator machine and investigations of traffic chracteristics (speeds, axle loads etc.) in highways in service, is described and then the rutting prediction using this method is compared with the actual field studies of P.C.C.P. in service.

AS a result , the rutting value calculated with this prediction method can predict the field progress, so it is confirmed that this method can be effectively conducted to predict the life of P.C.C.P. in cold areas.

ON A PREDICTION METHOD OF THE RUTTING PROGRESS OF PORTLAND CEMENT CONCRETE PAVEMENT IN COLD AREAS

by

Mamoru Onodera, Takeaki Kato, Fumitoshi Koga, Kaoru Sakamoto

Nihon Doro Kodan (Japan Highway Public Corporation)

1/ Introduction

The total length of expressways in Japan has reached 3200km in March,1984 and it can be said that "age of expressways" has substantially come to Japan. The shere of concrete pavements, however, is still low ; no more than 200km (100km in Tunnels). Recently, the good durability of concrete pavements has been re-estimated and then the difference in total costs, which contain construction costs and maintenance expenditures, between concrete pavements and asphalt pavements has got smaller because of economical policies ; omittion of construction machines, decrease of concrete pavement depth, etc.. So, it is considered that concrete pavements will be widely adopted near future.

To study the abrasion resistance of concrete pavements in cold areas against studded tires and tirechaines, the laboratory of Nihon Doro Kodan (Japan Highway Public Corporation) has carried out the laboratory tests using Traffic Simulator Machine since 1970. Up to now, influences of tire-types, speed and wheel load as traffic conditions and cement types, aggregate kinds, mix propotion, age and casting methods as specimen factors upon the abrasion of concrete pavements have been studied.

In this paper, laboratory abrasion test results are summarized in each factor which influences on the abrasion of concrete pavements. Moreover, the prediction method of wear of concrete pavements is suggested on the basis of these laboratory test results.

2/ General of Experments

General of abrasion tests of concrete pavements is shown in Table-1.[1),2)] Traffic Simulator Machine, shown in Photo-1, was used as a test machine. Specifications of this machine are shown in Table-2.

Table-1 General of abrasion tests of concrete pavements

Test No.	Specimen Parameters: Materials (kinds) *1) Cement	Max. Size of Coarse Aggregate	Fine Aggregate	Coarse Aggregate	Mix Proportion	Traffic Parameters: Tire Type *2)	Wheel Load (ton)	Speed (km/hr)	Torque (kg-m)	Wheel Path Shift (mm)	Air Pressure of Tire (kg/cm^2)
1	N	25 40	4	6	Unit Cement Content 2 kinds	S-1	2.0	40	30	±60	6.0
2	N	40	1	1	Unit Cement Content 2 kinds	S-2	2.0	40	30	±60	6.0
3	N	40	1	1		S-3	2.0	40	30	±60	6.0
4	N	40	1	1		C-1	2.0	40	30	±60	6.0
5	N	40	1	6		C-2	0.5 1.0 2.0	40	30	±30	3.0 6.0
6	N	40	1	1		S-4	0.5 1.0 2.0	40	30	±30	3.0 6.0
7	H	40	3	3	Unit Cement Content 3 kinds	S-5	2.0	20 40 60	30	±50	6.0
8	H	40	3	3	Unit Cement Content 3 kinds	C-3	2.0	20 40 60	30	±15	6.0
9	M	40 25	3	3	Unit Cement Content 4 kinds Sand Percentage 3 kinds	N-1 S-6	2.0	60	30	±50	6.5
10	M	40 25	3	3	Unit Cement Content 4 kinds Sand Percetage 3 kinds	C-4	2.0	60	30	±15	6.5
11 12 13	M	13 40	2	2	Unit Cement Content 3 kinds Sand Percentage 3 kinds Air Content 3 kinds	S-7	2.0	60	30	±60	6.0
14 15	N H M	20 40 80	9	10	Unit Cement Content 6 kinds	S-8	2.0	60	30	±58	7.0

*1) N:Normal Portland Cement
M:Normal Portland Cment
H:Normal Portland Cment

*2) S:Studded Tire
C:Tire Chain
N:Normal Tire

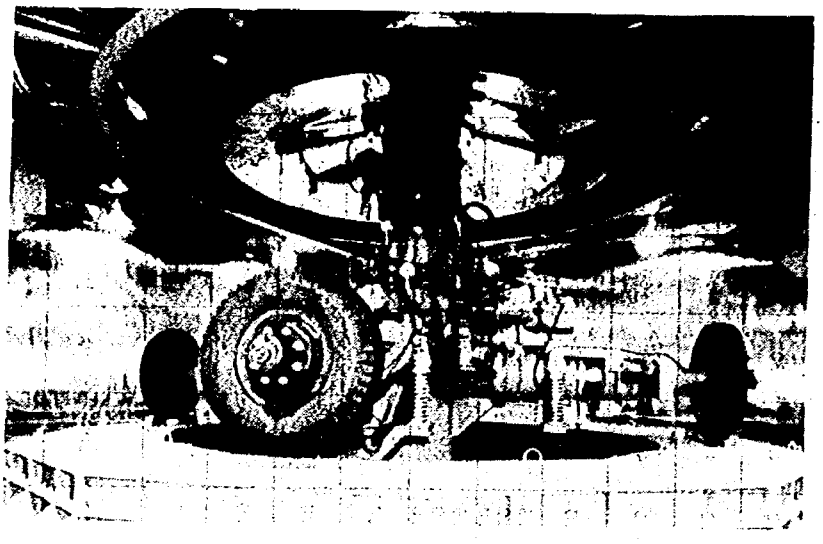


Photo-1 Traffic Simulator Machine

Table-2 Specifications of Traffic Simulator Machine

Diameter of Wheel Path (mm)	6000
Number of Wheels	4
Wheel Load (ton)	0-3.0
Speed (km/hr)	10-60
Tires	Truck-tire 10-20-14PR
Torque	Max.±6% Inclination

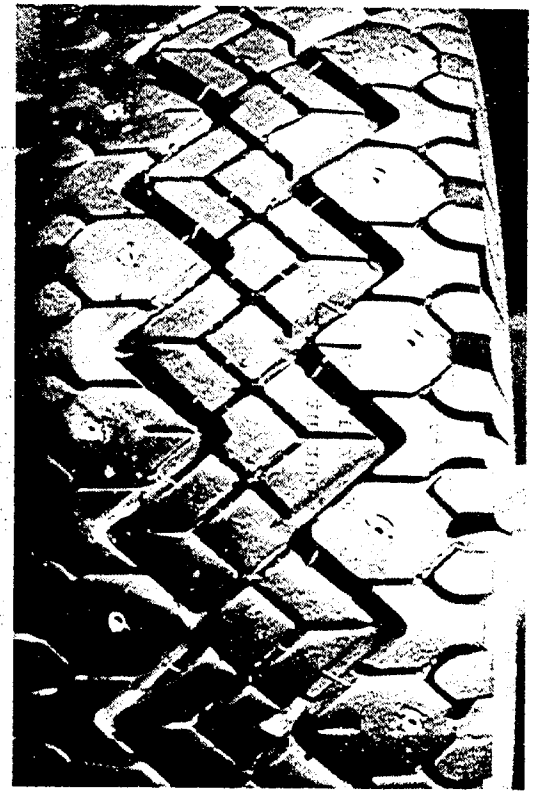
In these tests, materials, mix proportion, execution, surface texturing and strength as specimen factors and tire types (studded tires, tirechains), wheel load, taffic speed, torque, shift of wheel path, air pressure of tires and the number of tire passes as traffic parameters were studied. Surfaces of all specimens were moistured and the room temperature was adjusted to make the surface temperature 0°C.

3/ Influence of Traffic

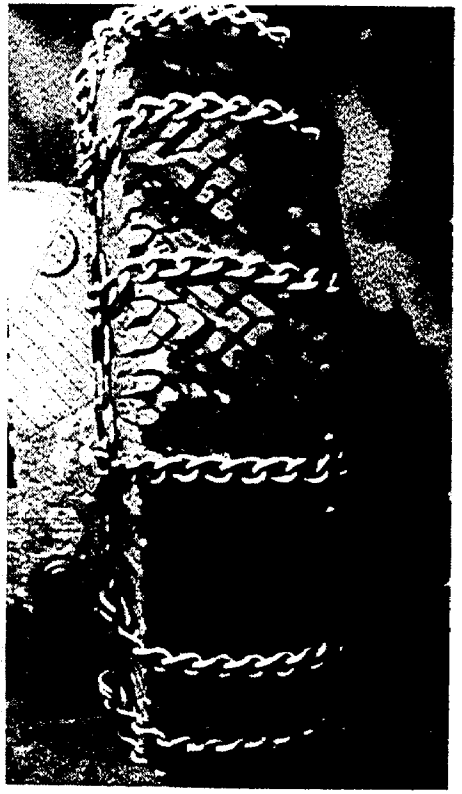
3.1/ Tire

Shapes of tires used in abrasion tests are shown in Photo-2. Figure-1 shows the wear characteristics VS. tire types. The average wear depth by tirechains is ten times deeper than that by studded tires within 30 thousand tire passes. This difference is caused by the difference of impact forces between studded tires and tirechains. The abrasion by tirechains is linearly increasing as well as test for abrasion of coarse aggregates is carried out. Photo-3 shows the wear charactetistics by tire types.

Nomal tire



Studded tire



Tirechain

Photo-2 Shapes of tires for abration tests

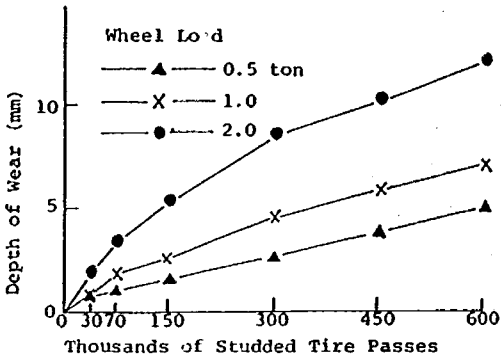


Fig-2 Wear rates by studded tire V.S wheel load

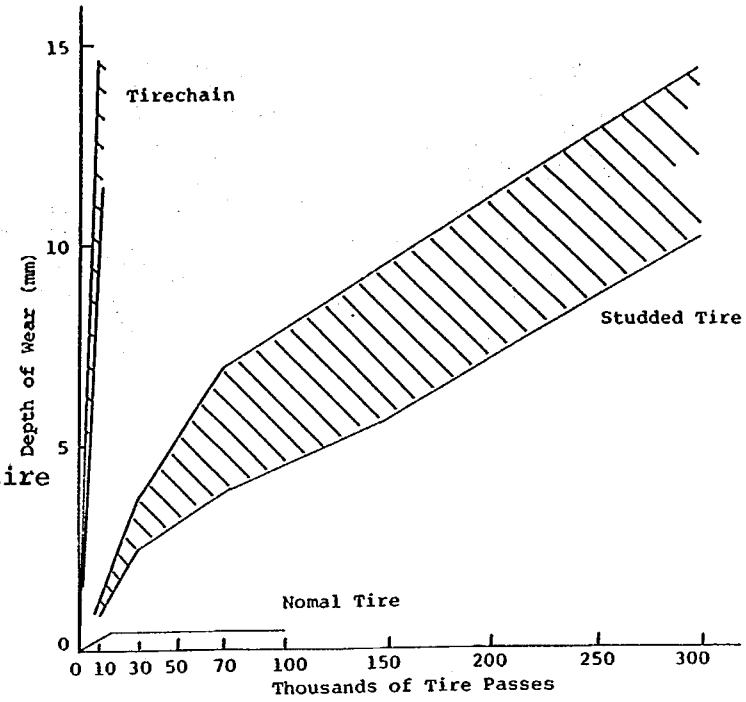


Fig-1. Wear rates by tire types(Cement Content : 320kg/m^3)

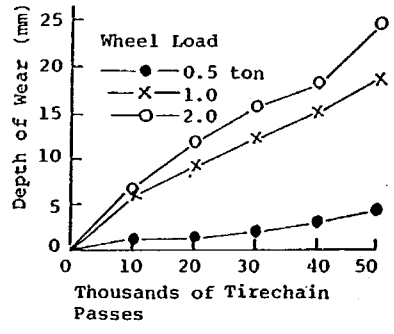
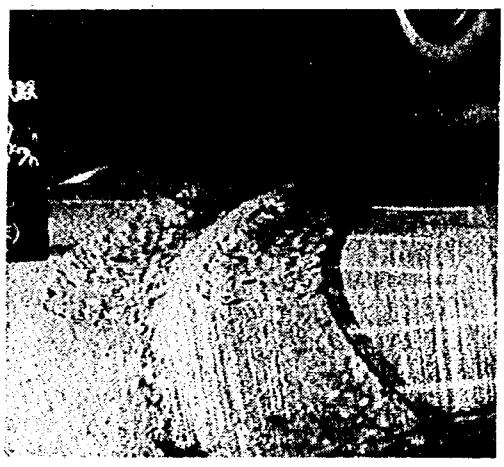


Fig-3 Wear rates by tirechains V.S Wheel load



Nomal tire (40thousand of tire passes)

Tirechain(40 thousand of passes)

Studded tire (600 thousand of passes)

Photo-3 Wear characteristics by tire types

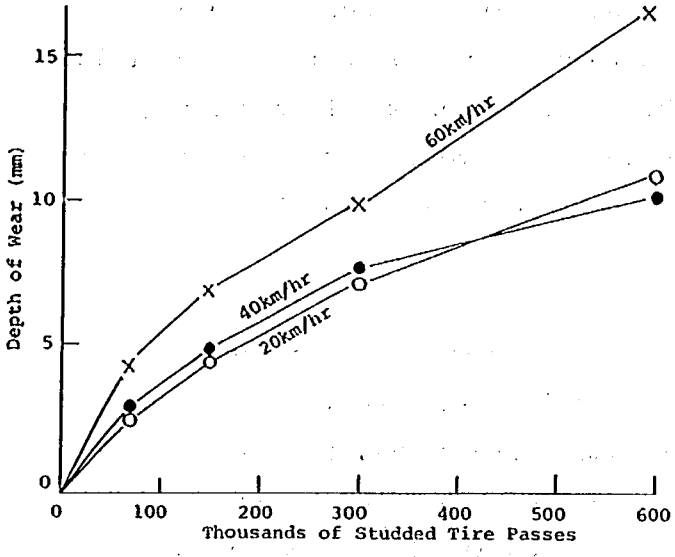


Fig-4 Wear rates by studded tires V.S traffic speed

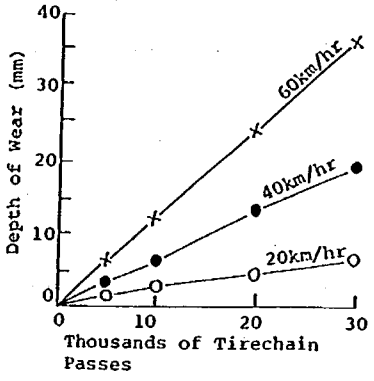
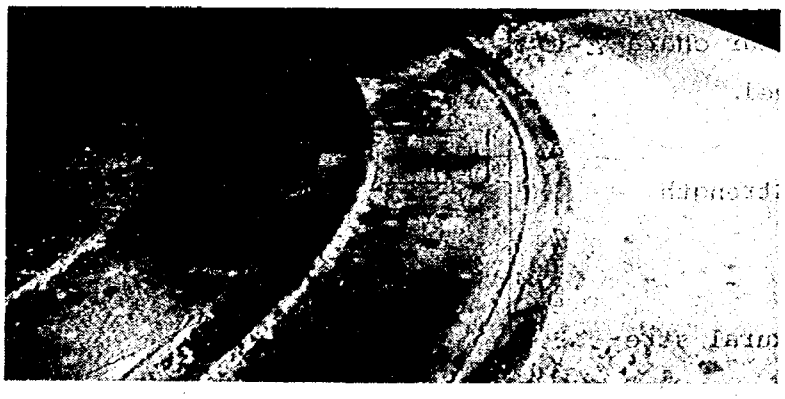
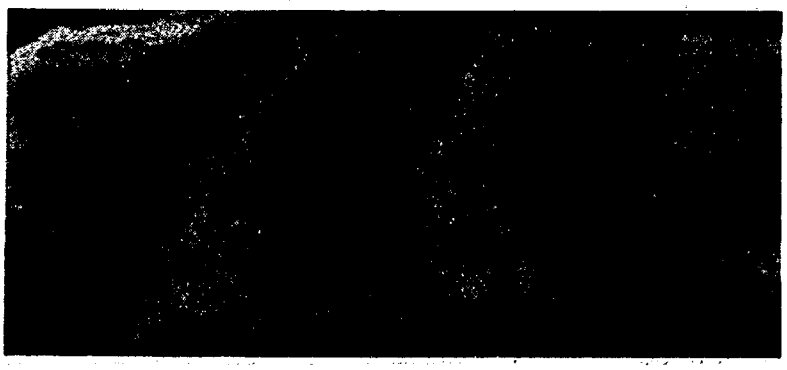


Fig-5 Wear rates by tirechains V.S traffic speed



Studded tire
(600 thousands of tire passes)

40km/hr 60km/hr 20km/hr



Tirechain
(30 thousands of tire passes)

40km/hr 60km/hr 20km/hr

Photo-4 Wear characteristics by traffic speed

3.2/ Wheel load

Wear characteristics VS. wheel load are shown in Figure-2 and 3. The abrasion by studded tires is increasing in proportion to wheel load. The increase of abrasion by tirechains is rather small over 1 ton of wheel load. As this cause, it is considered that impact forces by tirechains, which induce the abrasion, are not in proportion to wheel load.

3.3/ Traffic Speed

Figure-4 and 5 show the abrasion VS. traffic speed. The abrasion by studded tires at 20km/hr is not clearly distinguished from that at 40km/hr. The abrasion at 60km/hr is 1.3-1.8 times larger than that at 20km/hr. The wear depth by tirechains has linear relation with tirechain passes and the relation between the abrasion and speed is clearly observed. Concretely the abrasion at 60km/hr and that at 40km/hr is 6 times and 3 times larger than that at 20km/hr respectively at 30 thousand tire passes. Photo-4 shows wear characteristics by traffic speed.

4/ Infuence of Concrete Strength

4.1/ Flexural Strength

The relation between flexural strength and the abrasion by studded tires is shown in Figure-6. It is observed that the wear depth intends to decrease as flexural strength increases.

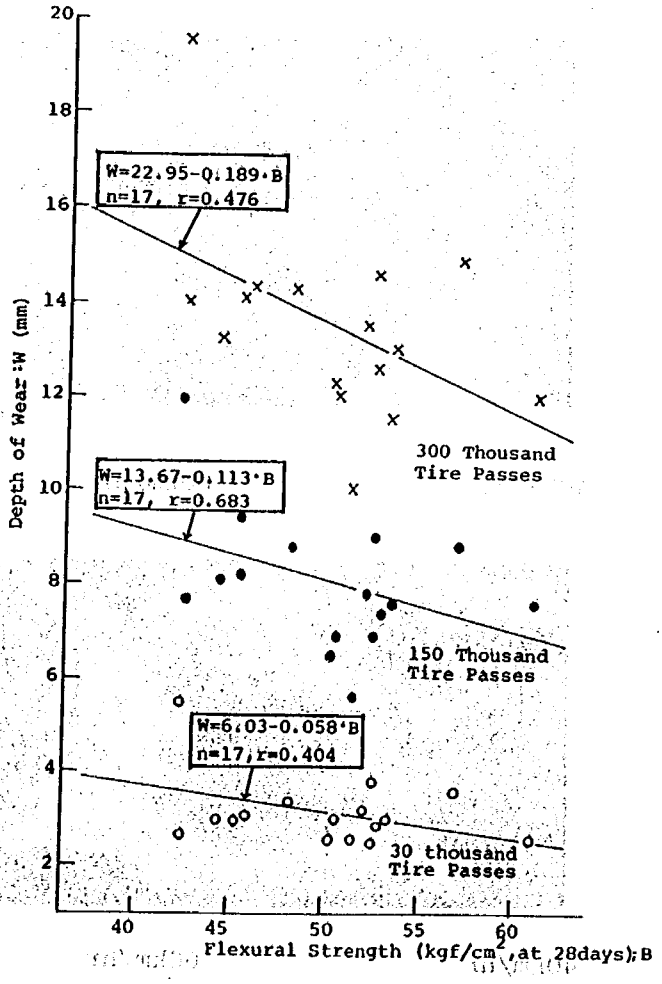


Fig-6 Relation between wear rates by studded tires and flexural strength

5/ Influence of Aggregates

5.1/ Max. Size of Coarse Aggregates

Figure-7 shows wear depth VS. max.size of coarse aggregates. It can be observed that the wear depth intends to decrease as max.size of coarse aggregates get larger.

It is considered that this result was caused by increase of the abrasion resistance due to larger flexural strength induced by fewer water content.

5.2/Abrasion Loss of Coarse Aggregates

Wear depth VS. abration loss of coarse aggregates is shown in Figure-8 and 9. It can be observed that wear depth by studded tires intends to increase as abrasion loss of coarse aggregates increases. However, this tendency was not obvious in wear depth by tirechains.

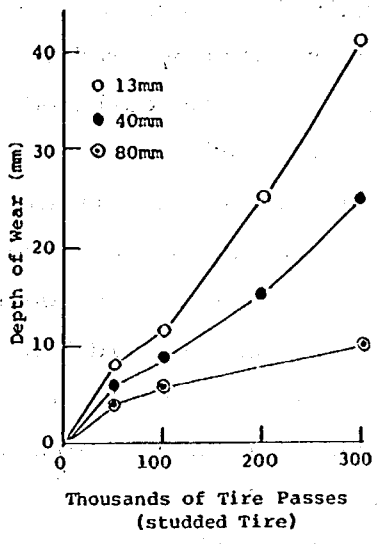


Fig-7 Depth of wear VS. max. size of coarse aggregates

6/ Prediction Method of Wear Progress

6.1/ Prediction of Wear

Structural of design concrete pavements are conducted on the basis of the fatigue design of a plain concrete slab where stress occurs because of traffic load and out-door temperature changes. The design life of concrete pavements is 20 years. However, in the area where there are a lot of vehicles equipped with studded tires or tirechains in winter, a lot of examples that the life of concrete pavements is decided by wear of concrete slab have occurred. In Nihon Doro Kodan Specification, purposed depth of wear which requires repairs in 25mm. So, if the prediction of wear progress can be carried out, the pavement life in cold area will be decided.

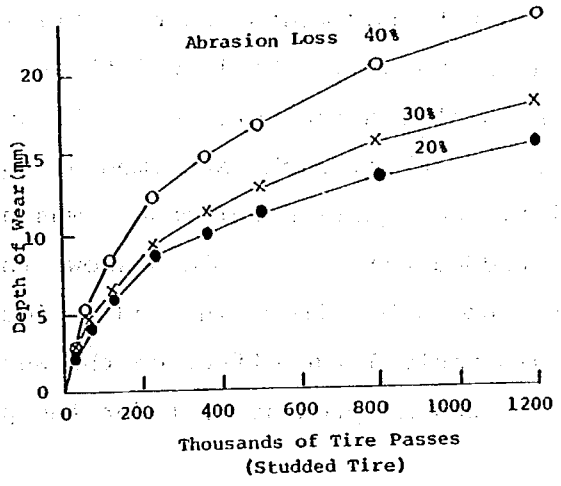


Fig-8 Depth of wear VS. abrasion loss of coarse aggregates (studded tires)

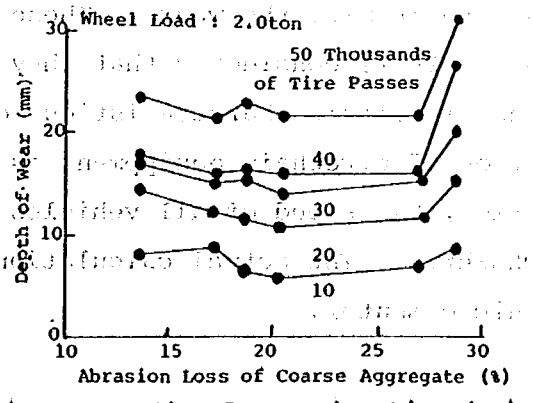


Fig-9 Depth of wear by tirechains VS. abrasion loss of coarse aggregates

6.2/ Analysis of Abrasion Test Results

From test results by Traffic Sumilater Machine, each data of 12 parameters (x1. to x12) was gathered and arrenged to each wear (Y) and then using an equation ($Y=ax_1^{b1}. x_2^{b2}$ ----), multiple regression analyses (multiple correlation analysis and partial correlation analysis) were carried out. Table-3 shows 12 parameters as independent variables and meen, maximum, and minimum of earch parameter. The wear as a dependent variable has the unit of section area and the width of the wear is supposed to be 30cm.

Table-4 shows the results of multiple regression analysis between the wear by studded tires and various parameters. The facter which has the largest influence on the wear is as a mater of course the number of tire passes and the next one is wheel load. On the contrary, materials, mix proportion, and strength have a little influence on the wear besides flexural strength has fairly large influence on that. So it can be considered that the wear of concrete pavements can be sufficiently predicted by traffic conditions (traffic volume, vehicle weigth and traffic speed) and flexural strength. This is also actual proved by the analysis (standard deviation and corelation coefficient) of each experimental equation for case-1 and case-7. From this point, the equation for case-6 was adopted as that for the prediction of wear by studded tires. Table-5 shows the results of multiple regression analysis between wear by tirechains and various parameters. Traffic conditions have considerably large effect on the wear by tirechains. So, the equation for case-6 was adopted as that for the prediction of wear by tirechains.

6.3/ Prediction Method of the Depth of Wear

From the investigations mentioned above, Figure-10 is supposed as the prediction method of the depth of wear. Table-6,7,8,and 9[3),4),5)] shows the traffic data to predict the wear. These data were survered and gathered on expressway. It is considered that they can be effectively used to estimate the general traffic characteristics on expressways. Now, as the traffic regulation of tirechain equippment is usually carried out at 50km/hr speed requlation, the speed of all vehicles equipped with tirechains is supposed to be 50km/hr. The actual calculation of wear can be conducted using an electronic computer.

6.4/ Examples of the Calculation of Wear Depth

Figure-11 shows the results of the wear calculated using the method shows in Figure-10. Calculations were carried out for 4 cases : large-sized cars with studded tires or tirechains and small-sized cars with studded tires or tirechains. From Figure-11, it can be observed that daily traffic volume which will cause 25mm depth of wear after 20 years is about 30 large-sized vehicles with tirechains, about 700 small-sized vehicles with tirechains, about 1 thousand large-sized vehicles with studded tires, and about 90thousand small-sized vehicles.

6.5/Comparison between field surveys and caluculated values.

Figure-12, and 13 show the comparison of the wear progress with the passage of time between results surveyed in the follow-up researches and values calculated by the prediction method of wear progress described above. Now., the calculation conditions are shown in Table-10. Althougth the comparison is carried out only at 2 or 3 years after these expressways were open to traffic, it is observed that calculated values are considerably consistent with surveyed results except having somewhat different shapes due to the difference of traffic conditions. Thus, it is considered that the changes of wear with the passage of time can be predicted more correctly if traffic conditions are accurately surveyed.

7/ Summary

Based on laboratory tests of influences of concrete mix proportions, surface texturing, traffic conditions, etc. on the abrasion resistance of concrete pavements, new effective prediction method of wear proggress with the passage of time has been developed and used to compare results surveyed in expressways in service and values calculated by this method. As a result, it is confirmed that this method can be effectively used to predict wear progress of concrete pavements in cold areas. After this, especially, taking note of traffic conditions, wheel load data, speed data, and wheel path data will be gathered and arranged to improve this method.

Table-3 Experiment Condition (Parameters to predict Wear Concrete Surfaces)

Items / Abbreviated Words / Unit / Case of Experiment		Y	X1 , X2 , X3 , ---												
		Weared Section Area	Traffic Condition			Surface Condition (Concrete)									
						Concrete Materials								Concrete	
						Cement	Fine Aggregate			Coarse Aggregate					
			Number of Passing Times	Wheel Load	Speed	Flexural Strength at 28 d	Specific Gravity	Fineness Modulus	Unit Weight	Max. Size	Absorbed moisture	Abration Loss		Flexural Strength at 28 Days	Ages of Test Specimen
		AS,AC	N	W	V	CB28	Sp	SFM	SW	GMS	Gq	GL		B28	D
		cm2	times	ton	Km/hr	kgf/cm2	----	----	kg/l	mm	%	%		kgf/cm2	day
Case of Studded Tires S (Number of Surfaces =127)	Mean	26.5	----	1.76	42.0	69.0	2.60	2.81	1.72	39	1.2	18.8		56.1	150
	Maximum	53.3	1.2 million	2.00	60.0	70.2	2.63	3.41	1.85	60	4.3	43.1		93.1	365
	Minimum	8.7	0	0.50	20.0	67.3	2.58	2.19	1.62	15	0.4	10.3		34.1	7
Case of Tirechains C (Number of Surfaces =103)	Mean	21.8	----	1.71	43.3	68.8	2.55	2.78	1.72	38	1.1	19.0		52.5	93
	Maximum	48.9	50 thousand	2.00	60.0	72.6	2.62	3.01	1.83	40	2.0	28.7		80.5	271
	Minimum	4.8	0	0.50	20.0	67.3	2.56	2.58	1.62	15	0.5	13.5		42.1	28

N.B.) AS:Section Area weared by Studded Tire (Case of 300thousand times)

AC:Section Area weared by Tirechains (Case of 10thousand times)

Table-4 Multiple regression analysis between wear by studded tires and various parameters

Items / Y	X1 , X2 , X3 , -----											Number of Data	Inspection of Experimental Expressions				
	Traffic Condions			Cement	Fine AggReates		Coarse Aggreates			Age	Strength		Y/Y				Correlation Coefficient
	N	W	V	CB28	SFM	SW	GL	GMS	GQ	D	B28		Mean	Standard Daviation	Max.	Min.	
Case-1	0.955 (1)	0.733 (2)	0.250 (5)	0.202 (7)	0.265 (4)		0.182 (8)	0.118 (9)	0.110 (10)	0.239 (6)	0.352 (3)	769	0.996	0.232	1.949	0.348	0.996
Case-2	0.954 (1)	0.73 (2)	0.290 (4)	0.181 (7)	0.253 (5)		0.159 (8)			0.230 (6)	0.331 (3)	769	0.996	0.234	1.942	0.331	0.965
Case-3	0.953 (1)	0.722 (2)	0.263 (4)	0.186 (7)	0.235 (5)					0.226 (6)	0.377 (3)	769	0.997	0.236	1.908	0.326	0.964
Case-4	0.954 (1)	0.756 (2)	0.283 (4)		0.250 (5)						0.397 (3)	769	0.999	0.242	1.912	0.329	0.933
Case-5	0.949 (1)	0.784 (2)	0.258 (4)							0.105 (5)	0.324 (3)	769	1.002	0.249	1.906	0.336	0.960
Case-6	0.952 (1)	0.782 (2)	0.243 (4)								0.374 (3)	769	1.003	0.251	1.962	0.331	0.960
Case-7	0.955 (1)	0.612 (2)	0.199 (5)			0.272 (4)					0.407 (3)	769	0.998	0.239	1.866	0.331	0.963

Experimental Equations *1)

Case-1 $As=10^{5.160}\cdot N^{0.608}\cdot W^{0.717}\cdot V^{0.247}\cdot CB^{-1.589}\cdot SFM^{0.636}\cdot GL^{0.198}\cdot GMS^{-0.146}\cdot GQ^{-0.073}\cdot D^{-0.081}\cdot B^{-0.714}$

Case-2 $As=10^{4.374}\cdot N^{0.606}\cdot W^{0.707}\cdot V^{0.282}\cdot CB^{-1.414}\cdot SFM^{0.661}\cdot GL^{0.163}\cdot D^{0.079}\cdot B^{-0.665}$

Case-3 $As=10^{4.937}\cdot N^{0.608}\cdot W^{0.697}\cdot V^{0.253}\cdot CB^{-1.469}\cdot SFM^{0.570}\cdot D^{-0.078}\cdot B^{-0.751}$

Case-4 $As=10^{0.846}\cdot N^{0.607}\cdot W^{0.734}\cdot V^{0.273}\cdot SFM^{0.613}\cdot B^{-0.751}$

Case-5 $As=10^{1.044}\cdot N^{0.602}\cdot W^{0.793}\cdot V^{0.247}\cdot D^{0.025}\cdot B^{-0.621}$

*2) Case-6 $As=10^{1.044}\cdot N^{0.607}\cdot W^{0.783}\cdot V^{0.229}\cdot B^{-0.691}$

Case-7 $As=10^{9.545}\cdot N^{0.610}\cdot W^{0.629}\cdot V^{0.182}\cdot B^{-0.730}$

*1) Exprimental Equations

$Y=a\ X1^{b1}\cdot X2^{b2}\cdot X3^{b3}\cdot$----------

*2) An equation suggested to predict the depth of wear.

N.B.) N:Number of tire passes ,W:Wheel load ,V:Speed ,CB28:Flexural Strength of cement at 28 days ,SFM:Fineness modulus of sand ,Sw:Unit weigth of sand , Gl:Abration loss of coarse aggregate ,GMS:Max. size of coarse aggregate , GQ:Content of absorbed moisture of coarse aggregate ,D:Day , B28:Flexural strength of concrete at 28 days.

Table-5 Multiple regression analysis between wear by tirechains and various parameters

Items / Y	X1 , X2 , X3 , ------											Number of Data	Inspection of Experimental Expressions				
	Traffic Conditions			Cement	Fine Aggregate		Coarse Aggregate			Age	Strength		Y/Y				Correlation Coefficient
	N	W	V	CB28	SFM	SW	GL	GMS	GQ	D	B28		Mean	Standard Daviation	Max.	Min.	
Case-1	0.953 1	0.678 3	0.786 2			-0.128 7			-0.279 4	-0.141 6	-0.152 5	620	1.033	0.357	3.027	0.390	0.964
Case-2	0.952 1	0.744 3	0.787 2						-0.273 4	-0.087 6	-0.137 5	620	1.032	0.354	3.009	0.387	0.964
Case-3	0.959 1	0.768 3	0.802 2						-0.276 4		-0.139 5	620	1.030	0.357	3.039	0.393	0.964
Case-4	0.954 1	0.652 3	0.773 2			-0.042 4					0.015 5	620	1.032	0.437	4.635	0.424	0.961
Case-5	0.984 1	0.726 3	0.769 2							-0.097 4	0.021 5	620	1.035	0.429	4.500	0.428	0.961
Case-6	0.956 1	0.749 3	0.784 2								0.020 4	620	1.032	0.434	4.585	0.425	0.960

Experimental Equations *1)

Case-1 $As=10^{5.026 \cdot N^{1.021} \cdot W^{0.936} \cdot V^{1.28} \cdot Sw^{-1.502} \cdot Gq^{-0.476} \cdot D^{-0.082} \cdot B^{-0.522}}$

Case-2 $As=10^{-0.017 \cdot N^{1.019} \cdot W^{0.999} \cdot V^{1.282} \cdot Gq^{-0.478} \cdot D^{-0.043} \cdot B^{-0.522}}$

Case-3 $As=10^{-0.015 \cdot N^{1.019} \cdot W^{0.963} \cdot V^{1.248} \cdot Gq^{-0.475} \cdot B^{-0.478}}$

Case-4 $As=10^{0.412 \cdot N^{1.033} \cdot W^{0.914} \cdot V^{1.198} \cdot Sw^{-0.441} \cdot B^{0.046}}$

Case-5 $As=10^{-1.041 \cdot N^{1.016} \cdot W^{0.983} \cdot V^{1.249} \cdot D^{-0.050} \cdot B^{0.062}}$

*2) Case-6 $As=10^{-1.058 \cdot N^{1.029} \cdot W^{0.942} \cdot V^{1.208} \cdot B^{0.059}}$

*1) Experimental Equations;
$Y=a \cdot X1^{b1} \cdot X2^{b2} \cdot X3^{b3} \cdot \cdots\cdots\cdots$

*2) An equation suggested to predict the depth of wear.

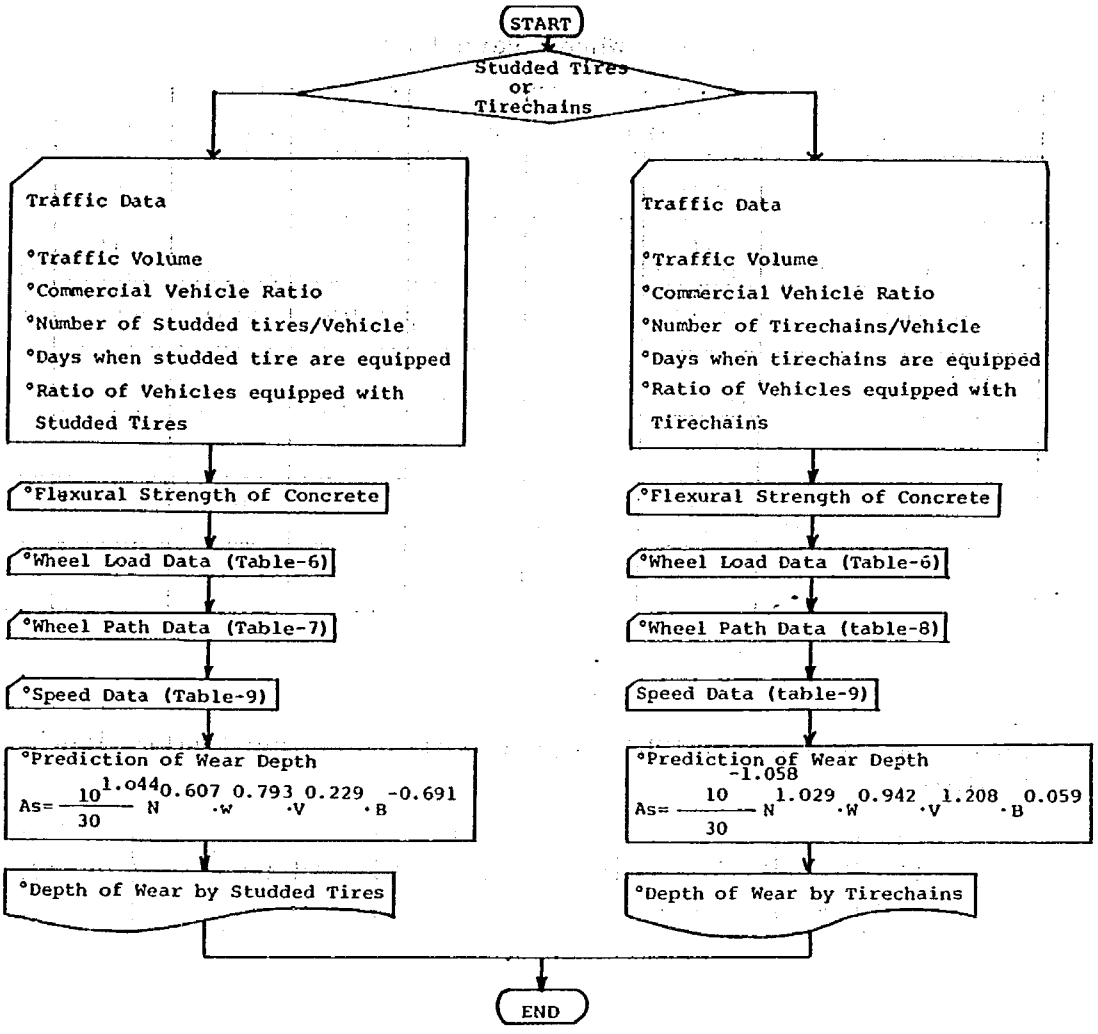


Fig-10 Flow chat of the prediction method of wear depth

Table-6 Wheel Load Data

Vehicle Types	Wheel Load (ton)	Frequency
Large-sized Vehicles	1.0(0 ~ 1.0)	0.1300
	2.0(1.1 ~ 2.0)	0.2200
	3.0(2.1 ~ 3.0)	0.3515
	4.0(3.1 ~ 4.0)	0.2080
	5.0(4.1 ~ 5.0)	0.0905
Small-sized Vehicles	0.25	1.0000

N.B) Wheels whose loads are more than 5 tons,are supposed as double tires,so these wheel loads are supposed as half of them in order to convert into single tires.

Table-7 Wheel Path Data

Large-sized Vehicles	Wheel Path (cm) (Length from Lane Mark)	1 (-60--31)	2 (-30--1)	3 (0-30)	4 (31-60)	5 (61-90)	6 (91-120)	7 (121-150)
	Frequency	0.005	0.020	0.075	0.125	0.155	0.055	0.025
	Wheel Path (cm) (Length from Lane Mark)	8 (151-180)	9 (181-210)	10 (211-240)	11 (241-270)	12 (271-300)	13 (301-330)	14 (331-360)
	Frequency	0.015	0.035	0.120	0.160	0.080	0.035	0.005
Small-sized Vehicles	Wheel Path (cm) (Length from Lane Mark)	1 (-60--31)	2 (-30--1)	3 (0-30)	4 (31-60)	5 (61-90)	6 (91-120)	7 (121-150)
	Frequency	0.005	0.005	0.015	0.025	0.035	0.060	0.065
	Wheel Path (cm) (Length from Lane Mark)	8 (151-180)	9 (181-210)	10 (211-240)	11 (241-270)	12 (271-300)	13 (301-330)	14 (331-360)
	Frequency	0.075	0.060	0.060	0.060	0.060	0.035	0.015

Table-8 Wheel Path data (for Tunnels, for both Large and Small-sized Vehicles)

Wheel Path (cm) (Length from Lane Mark)	1 (0-30)	2 (31-60)	3 (61-90)	4 (91-120)	5 (121-150)	6 (151-180)	7 (181-210)	8 (211-240)	9 (241-270)	10 (271-300)	11 (301-330)	12 (331-360)
Frequency	0.001	0.030	0.122	0.152	0.080	0.026	0.012	0.017	0.089	0.162	0.103	0.020

Table-9 Speed Data

Vehicle Types / Speed (km/hr)	Large-sized Vehicles	Small-sized Vehicles
≧ 60.0	0.065	0.010
60.1-70.0	0.215	0.075
70.1-80.0	0.470	0.245
80.1-90.0	0.200	0.340
90.1-100.0	0.050	0.220
100.1 ≦	--	0.110

N.B.) Speeds of vehicles equipped with studdedtires are supposed to be 10km/hr slower than each speed described in this table. Speeds of vehicles equipped with tirechains are supposed to be 50km/hr becauce max. speed is limited to be 50km/hr whenever equipment of tirechains is obliged.

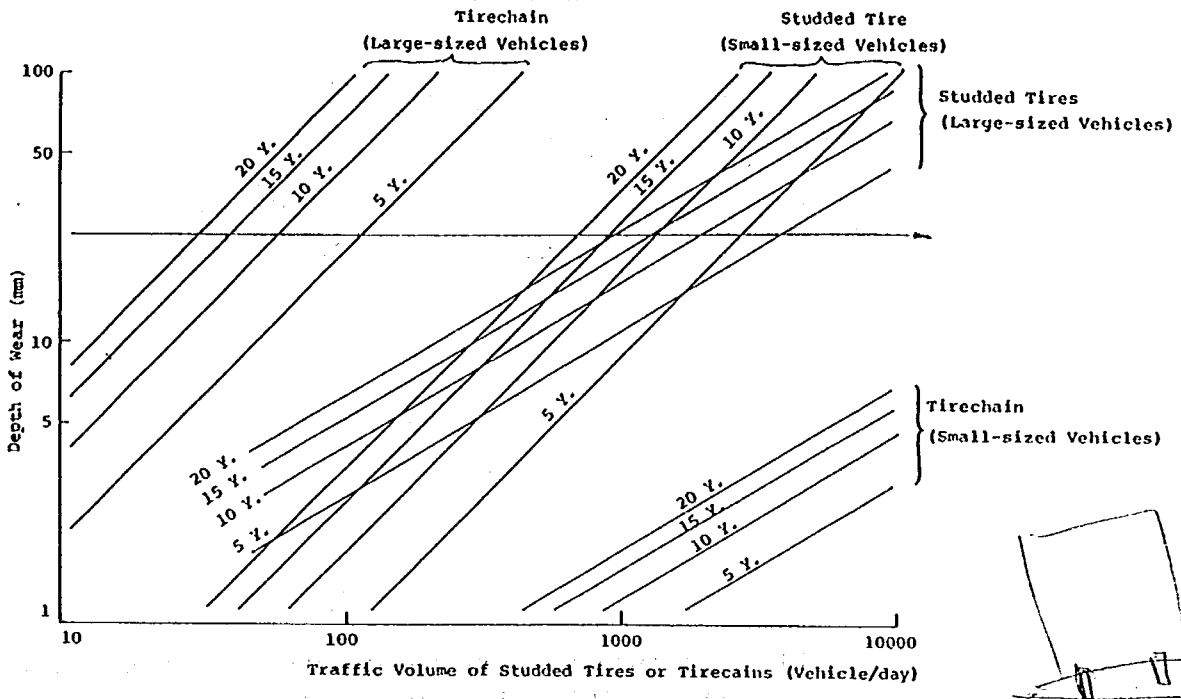
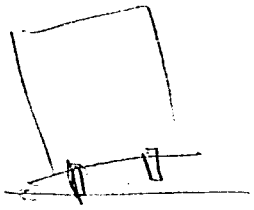


Fig-11 Results of calculation of wear depth



wheel path

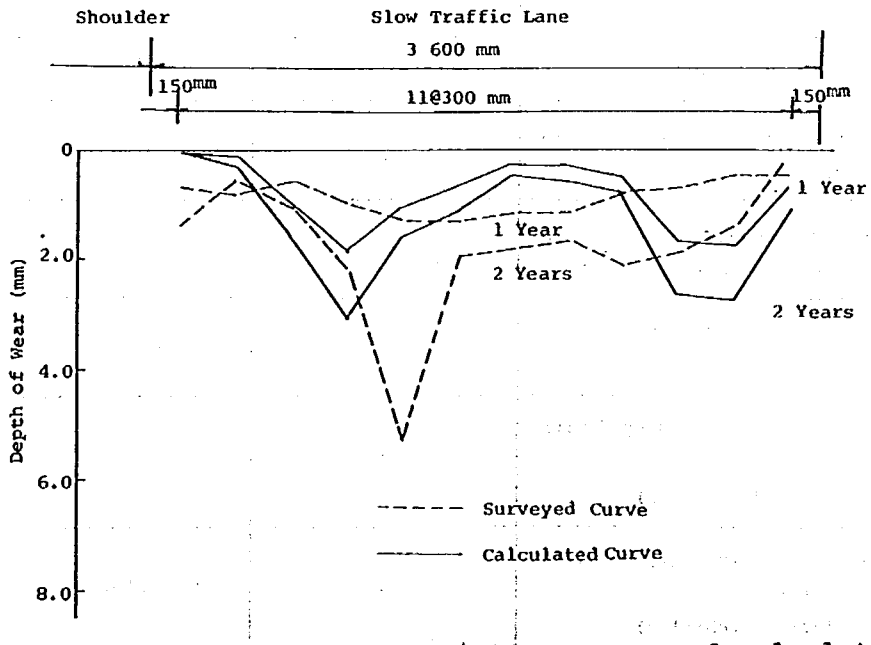


Fig-12 Comparision between field surveys and calculated curves (for Tohoku Expressway)

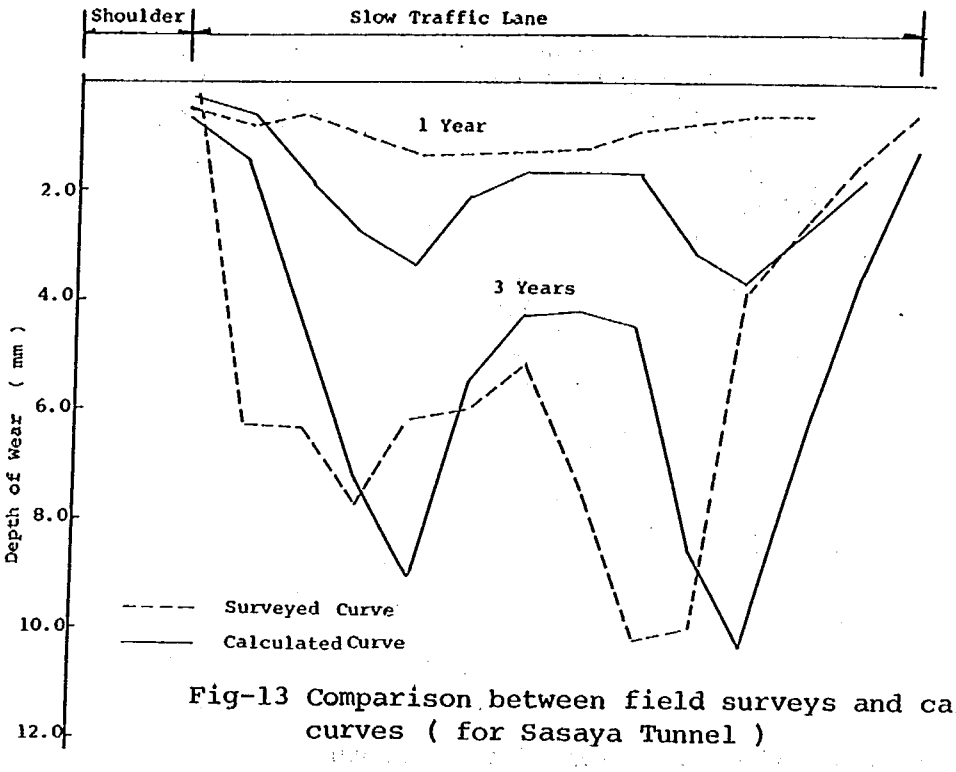


Fig-13 Comparison between field surveys and calculated curves (for Sasaya Tunnel)

Table-10 Conditions for calculating the depth of wear

Items	Tohoku Expressway	Sasaya tunnel
Traffic Volume (number of vehicles/day.lane)	2500.0	1898.0
Commercial Vehicles Ratio (%)	33.3	9.25
Number of Studded Tires/Vehicle (for large-sized vehicles)	2.0	2.0
Number of Studded tires/Vehicle (for small-sized vehicles)	3.0	3.0
Days When Studded Tires are equipped (day/year)	120	120
Ratio of Vehicles equipped with Studded Tires (%) (for large-sized vehicles)	20	20
Ratio of Vehicles eqipped With Studded Tires (%) (for small-sized vehicles)	20.0	20.0
Wheel Load Data	Table-6	Table-6
Wheel Path Data	Table-7	Table-8
Speed Data	Table-9	Table-9

Refereces

1. HOTTA, N., KANEKO, T., MIWA, H., etal, " A Study on the Abrasion Resistance of Cement Concrete Pavement Surface," The 1st-9th Reports of the Loboratory of Nihon Doro Kodan, 1970-1977 (in Japanese)
2. MIYAZAKI, T. and MURAKI,H., " A study on Wearing and Skid Resistance of Cement Concrete Pavement Surfaces, " The 1st-3rd Reports of the Laboratory of Nihon Doro Kodan, 1978-1980 (in Japanese)
3. " Report on Traffic Load Researches," Tokyo First Operation Bureau of Nihon Doro Kodan, March, 1980 (in Japanese)
4. " A Study on Relation between Slow Traffic Lane Width and Passing Traffic Lane Width," Nihon Doro Kodan, March, 1969 (in Japanese)
5. " A Study on the Actual Condition of Traffic in Highways," Highway Research Foundation of Japan, Feb., 1980 (in Japanese)

The Example of Research and Development of Improved Asphalt in Japan

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ABSTRACT

Rapid economic growth in Japan has caused an increase in the size of vehicles and the volume of traffic. Roads are now being used under regorous conditions. Moreover, new social needs of roads make it necessary to raise the qualitative level of asphalt pavement with regard to safety and noise countermeasures. The rigorous conditions of road use result in the flow or wear of asphalt pavement, and damage to pavement can no longer be left alone from the view point of traffic safety, noise and vibration. Thus, asphalt pavements more durable and useful than ever are in demand.
In Japan, generally the countermeasures for the flow and wear resistance of asphalt pavement consist of changing the gradings of mixtures and mix design including design asphalt content. But recently, improved asphalt containing such additives as thermoplastic resin, thermoplastic elastomer or some other improving agent has begun to be used. Though this improved asphalt is, on the whole, effective under suitable conditions, it is not necessarily adequate for use on the heavy traffic roads of today and the roads in cold, snowy districts and the development of still superior asphalt materials is being eagerly awaited.
To cope with these problems, the Public Works Research Institute and other research organizations are conducting the following research and development on improved asphalt by mobilizing all necessary government and private techniques.
1) "Research and development of semi-blown asphalt for flow-resistant paving" incorporating flow countermeasures for heavy traffic roads.
2) "Research and development of rubberrized asphalt for flow- and wear-resistant paving" incorporating both flow countermeasures for heavy-traffic roads and wear countermeasures for roads in cold, snowy districts.
This report describes the present state of the above research and development of improved asphalt.

THE EXAMPLE OF RESEARCH AND DEVELOPMENT OF IMPROVED ASPHALT IN JAPAN

by

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1. Research and Development of Semi-Blown Asphalt for Flow-Resestant Paving

1.1. Preface

This study concerns the "semi-blown asphalt for pavement" and has been developed with a view to improving the quality of asphalt as a way to reduce rutting due to the flow of asphalt pavement mextures and the possibility of its application to the pavement of heavy traffic roads. Since the start of this research in 1975, results having been achieved at different times and have been announced as a series of reports[1)-4)]. This paper is the final report on what has been generally concluded by now from the whole study.

Since about 1970, rutting due to the flow of asphalt pavement mixtures has steadily weakened in Japan and its countermeasures have caused concerned. This is the phenomenon where mixtures immediately under the wheel traveling locus move sideways little by little as they are subjected to repeated loads. If uneveness in the road surface resulting from the flow increases, repair aimed to assure the safety of vehicles using the roads is necessary even if there is no particular problem about the pavement structure itself. The presumed cause of the flow, as far as mixtures are concerned, is the viscoelasticity of asphalt because asphalt which bonds aggregates in the mixtures becomes prominently viscous in proportion as the temperature is high and the loading velocity is slow and its grip on the aggregates is lost. The impact of this outcome appears in the pavement in the form of such phenomena as the rapid progress of rutting in the hot season of summer and the fact that rutting is particularly great in the vicinity of an intersection or in a section with traffic congestion.

In the past studies, this has been coped with by means including 1) reduction of asphalt content, 2) use of a coarse aggregate grading and 3) use of hard asphalt. 1) and 2) are adjustments in the mix design. This change to the generally balanced mix of the past induces other damage, particularly cracks, but the problem has yet to be solved. Meanwhile, 3) may be conceived as a change made during 1966-75 from asphalt with a penetration of 80/100 mainly used in the past to 60/80 and a later attempt at 40/60. However, even the hardest of these asphalt types with the penetration of 40/60 cannot be always expected to be very effective in flow resistance[5]. Under these circumstances, we have sought to trial-manufacture an asphalt type with a high-temperature viscosity several times the level of the conventional pavement asphalt, carry out test paving with it and thereby not only confirm its flow resisting effect but also make it practical. This research and development dates back to 1975 and the research progress up to the present is as indicated in Fig.-1.1.

1.2. Japanese Pavement Environments

Foreign examples of similar studies have been rare, probably because the phenomenon of rutting due to flow does not pose so serious a problem in other countries as it does in Japan. In this chapter, we shall see the cause of the frequent occurrence of flow in Japan from the angle of Japanese pavement environment.

(1) Present Pavement Repair

As of April 1982, the total length of Japanese roads was about 1.1 million kilometers and the total length of Japanese paved roads (excluding low-cost pavement) was about 130,000km and thus, the pavement rate was 16%. Compared with the present state in the principal countries of Europe and America, the Japanese rate of pavement is considerably lower, as indicated in Table-1.1. But the proportion of pavement repair expenses in the overall cost of pavement is increasing yearly, topping 25% in FY 1980.

Data showing the details of pavement repair expenses by type of work are scarce but, according to a survey taken by the Road Bureau, Ministry of Construction, rutting countermeasures represent 56% of the amount of work of overlay pavement repair for designated sections of general national highways (total length : 12,000km) (Table-1.2).

Also, there is a report[6] that the reconstruction of about 75% of the total Tokyo Expressway area in the past was necessary because of rutting. Regarding Loop Road No.7 of Tokyo, which is typical of Japanese heavy-traffic

roads, its ratio of rutting countermeasures in the total area of repair work has topped 90% every year since 1975[7].
Against this background, much is expected of the development of materials effective for flow resistance as an urgent task.

(2) Roadway Width and Traffic Congestion

Table-1.3 compares roadway widths in the principal countries of Europe, America and Japan. Though there are some difference in survey years and width standards because of the limited availability of data, the table clearly shows that the Japanese roadway width is at a far lower level than that for the countries in Europe and America.
A small roadway width naturally often causes traffic congestion. Table-1.4 is an example of the analysis of traffic congestion on Japanese trunk roads. It indicates that, on the whole, 33% of all their sections see congestion during, at least, the morning and evening rush hours. A small roadway width and the congestion of motor traffic having been described above are major causes of rutting-the subject of this study-because the former results in the concentration of traveling loci of large vehicles at certain positions while the latter accelorates the flow of asphalt mixtures which are weak to slow moving cargoes.

(3) Traffic Load

Japan has about 38 million automobiles, or about twice the number in the European countries listed in Table-1.5. The value obtained by dividing this by the paved road length (Column E) -- which may serve as an index indicating traffic density for pavement -- is several times as large in Japan as in the other countries.
Table-1.5 shows that the Japanese possession of automobiles is characterized by the higher proportion of large vehicles, as compared with the other countries, and this has much to do with damage of pavement. To further clarify this characteristic, the number of large vehicles per kilometer of paved road length is shown in Column F of the table. If this is considered as an index of pavement destruction, it is 10 times more in Japan than in Europe and America -- an indication that Japanese pavements are exposed to extremely rigorous load conditions.
Column G of the table is for the annual total vehicle-kilometer. Here, assuming that large vehicles and passenger cars travel the same distance, the annual total vehicle-kilometer for large vehicles can be determined by multiplying this value by the commercial vehicle ratio. The value obtained

by dividing this by the total length of pavement (Column I) may be called as large vehicles passing a certain point of a paved road. This value, too, is about 10 times in Japan as in the countries of Europe and America.

(4) Temperature

The Japanese summer contains a considerably large number of days that temperature exceeds 30°C and these days sometimes continue for more than two weeks whereas, in Europe for example, the number of days that temperature exceeds 30°C is, indeed, few[8].

Temperature of the pavement rises with air temperature and, in the case of Japanese pavement, its surface temperature in summer reaches 60°C. In European instances of observation, the surface temperature of pavement generally is about 10°C lower than this and the number of days this temperature continues if fewer, compared with Japan. This difference of temperature appears also as the difference of temperature conditions in the wheel tracking test to check the high-temperature stability of asphalt mixtures, which are 45°C in Europe and 60°C in Japan. In view of the flow resistance of asphalt mixtures, a difference of 10°C at these temperatures is very large and, in terms of dynamic stability, this means a difference of 7-10 times[9].

Thus, it can be said that, from the view point of summer temperature, among other things, the Japanese asphalt pavement is in rigorous conditions liable to cause flow.

We have seen from the above description of the environments of pavement that the Japanese pavement is particularly under conditions more liable to accelerate rutting than in various other countries. Since these conditions will continue in the future, effective countermeasures in the form of easily controlable materials is in great demand mow.

1.3. Development of Semi-Blown Asphalt

In this study, we noted the high-temperature viscosity of asphalt with the purpose of obtaining mixtures that can effectively resist flow and trial-manufactured asphalt with increased viscosity. This chapter describes the progress of this effort and the properties of the trial-manufactured asphalt.

(1) Introduction of 60°C Viscosity

The flow of asphalt mixtures is largely governed by the consistency of asphalt. In this study, we adopted asphalt viscosity at 60°C (to be referred to hereafter as 60°C viscosity) as a measure of this consistency rather

than penetration and softening point, which have been conventionally used. This is for the following reasons:

1) Since the presumed maximum road surface temperature in Japan is 60°C, it is necessary to see the properties of asphalt at such a high temperature in order to study the phenomenon of flow.
2) Both the Marshall test and the wheel tracking test used to evaluate the high-temperature stability of asphalt mixtures are conducted at 60°C and data at this temperature are abundant.

When, under this policy, we investigated the 60°C vescosity of paving straight asphalt on the general market (to be referred to hereinafter simply as marked asphalt), we found that it is within the following range[2)]

Penetration 80/100 .....1,000～1,500 poise
Penetration 60/80 .....1,500～3,000 poise
Penetration 40/60 .....3,000～4,000 poise

(2) Trial Manufacture of Semi-Blown Asphalt

This study purports to increase the 60°C viscosity of asphalt and thereby improve its flow-resisting effect. Methods that can be used to improve asphalt in this manner include the follwoing[10)]: 1) distillation, 2) blending by propane deasphalting, 3) polycondensing of asphalt by adding catalyst, and 4) blowing of soft asphalt without using a catalyst. As the result of a comparative study of these methods, we decided to use 4) blowing as the method to produce asphalt with increased 60°C viscosity for which this study is intended for such reasons as the relative ease of viscosity adjustment, the relative freedom from dangers of low-temperature embrittlement and the economic feasibility of mass supply. The asphalt obtained by this process was named semi-blown asphalt.

A total of seven types of semi-blown asphalt was trial-manufactured by the blowing method after setting a target to increase 60 C viscosity to 4,000± 800 poise (two types), 8,000 ± 1,600 poise (three types) and 16,000 ± 3,200 poise (two tipes). Here, we deemed the tolerances of target viscosity to be about 20%. In trial manufacturing, we took care to avoid hindrance to low-temperature properties and workability by holding penetration below 40 and holding 180°C viscosity below 2 poise.

(3) Physical Properties of TrialManufactured Asphalt

As something to be used for practical paving, semi-blown asphalt must not allow viscosity increase to impair other properties required of paving asphalt. To confirm this, various specification tests were conducted for

seven trial-manufactured asphalt types and three marketed asphalt types. According to the results of these tests (Table-1.6), the characteristics of the trial-manufactured asphalt may be summarized as follows:

1) Each trial-manufactured asphalt type is within the range of target values set in advance for 60°C viscosity and its value is considerably higher, compared with the marketed asphalt.
2) Most trial-manufactured asphalt types satisfy the values that were specified by the Manual for Asphalt Pavement (1975 edition).

It became clear from these results that semi-blown asphalt could be manufactured by the blowing method and that it could be safely used as paving asphalt.

(4) Chemical Properties of Trial-Manufactured Asphalt

The molecular structure of asphalt is complex and, in it, many chemical components with different molecular weights exist together. The chemical properties of these components change as they undergo oxidation through the processes of blowing, hot mixing with aggregates, paving and use.

Table-1.7 shows the results of comparison of the chemical properties of the semi-blown asphalt used in the test paving (1st to 3rd) to be described in 1.5., marketed asphalt and blown asphalt. Iijima's 4-component separation[11] was used for composition analysis and GPC (Gel Permeation Chromatography) was used for the measurement of molecular weights and molecular weight distribution. The results of the test may be summarized as follows:

1) As can be seen from Fig.-1.2, composition analysis has revealed that semi-blown asphalt contains more asphaltenes and saturated hydrocarbon and less aromatic hydrocarbon than marketed asphalt. But this is the reverse, compared with blown asphalt, and compositionally, semi-blown asphalt is intermediate between these two types. In Fig.-1.2, AC-140 and AC-100 represent asphalt with 60 C viscosities of, respectively, 14,000 poise and 10,000 poise. Extracted asphalt I and II mean asphalt extracted from, respectively, the base course and the surface course.
2) In molecular weight and the distribution of molecular weights, also, semi-blown asphalt is intermediate between marketed asphalt and blown asphalt.

1.4. Properties of Mixtures Using Semi-blown Asphalt

Mixtures were produced, using the seven types of semi-blown asphalt and three types of marketed asphalt mentioned in the previous chapter, and the Marshall stability test and the wheel tracking test were conducted to see

their high-temperature stability, the high-velocity bending test to see their low-temperature brittleness and immersed Marshall stability test to see their stripping resistance.

(1) Marshall Stability Test

In preparing samples, we did the following, acting according to the Manual for Asphalt Pavement (1975 edition):

Manual for Grading: Grading of dense graded asphalt concrete

Asphalt content: Five types at intervals of 0.5% for a range of 5.0-7.0%

Number of times of compacting: 75 each for surface and reverse sides

The design asphalt content determined from the results of the Marshall stability test was about 6.0% regardless of asphalt types and the variation were small. So, in the 4.2-4 mixture test, asphalt content was made constant as 6.0%.

Fig.-1.3 shows the relation between the 60°C viscosity of asphalt and its Marshall stability (asphalt content = 6.0%). As can be clearly seen from the figure, the Marshall stability is in proportion to the 60°C viscosity. The high-temperature stability is also in proportion to this viscosity.

(2) Wheel Tracking Test

The wheel tracking test is a test to determine the high-temperature flow-resistance of asphalt mixtures. And it has been proved that the dynamic stability (to be abbreviated DS hereafter) that is obtained from the test is closely related to the actual rutting of pavement[12].

The materials and aggregate grading used in preparing samples were similar to those used in the Marshall stability test. The testing conditions used were according to the ordinary method.

Fig.-1.4 shows the relation between the 60°C viscosity of asphalt and DS. As can be seen from the figure, DS is in proportion to the 60°C viscosity.

The above two tests have disclosed that 60°C viscosity is closely related to the high-temperature stability of mixtures and that, therefore, the effect of flow resistance can be expected of semi-blown asphalt with increased 60°C viscosity.

(3) High-Velocity Bending Test

The semi-blown asphalt trial-manufactured in this study attaches importance to high-temperature stability. Yet, ignoring the importance of low-temperature brittleness, which is inseparably related to high-temperature stability is problematical from the viewpoint of resistance to cracking. To study this, we conducted a high-velocity bending test.

The mixtures were produced similarly to the wheel tracking test. The samples (4 x 4 x 30cm) were cut out, using a diamond cutter, and subjected to a bending test at a loading velocity of 13cm/s in a thermostatic chamber, using two-point support central loading. In a diagram plotted, using temperature as the axis of abscissas and bending strength as the axis of ordinates, from the results of the test conducted at temperatures variously changed from -10°C to 35°C, the temperature at which bending strength is maximal is called the brittle point and usually used as a yardstick for low-temperature properties. Brittling is difficult and the low-temperature properties are believed to be satisfactory in proportion as the brittle point is low.

Fig.-1.5 shows the relation between the 60°C viscosity of asphalt and the brittle point of mixtures. Though with considerable data variations, the brittle point lowers somewhat and the low-temperature properties move to the safe side if the 60°C viscosity is increased.

(4) Immersed Marshall Stability Test

As can be seen from Table-1.6, the trial-manufactured semi-blown asphalt is harder and less ductile than the marketed asphalt. Therefore, the stripping resistance between semi-blown asphalt and aggregates was apprehended. Generally, stripping resistance is determined by the immersed Marshall stability test. In this study, the properties of mixtures each made of 10 asphalt types were examined by this test. As the result, it was decided that the semi-blown asphalt involved no fear of stripping because the mean value of its residual stability was 85% and exceeded the prescribed value of 75% in the Manual for Asphalt Pavement (1978 edition).

It can be concluded from the above results that the semi-blown asphalt trial-manufactured in this study has the properties required of paving asphalt and, at the same time, can be expected to have the effect of flow resistance. But these are the results of an laboratory test on a material trial-manufactured by a small blowing unit and whether it is effective in actual paving can only be conjectured. Further, the chances of other damage that can possibly occur due to its increased 60°C viscosity have not yet been studied thoroughly. On the next phase of this study, it is necessary to construct test pavements using those materials and to observe their performance.

1.5. Inquiry by Test Pavement

The applicability of mixtures using semi-blown asphalt as pavement for an

existing road was now studied on the basis of the results of laboratory tests described in the preceding chapters. The main purposes of the test pavement were confirming the flow-resisting effect of semi-blown asphalt and finding the appropriate value of its 60°C viscosity.

(1) Quality Specification of Semi-Blown Asphalt

Prior to test paving, the quality specifications of the semi-blown asphalt were set as indicated in Table-1.8 (first specifications). For comparison, the table also lists the second and third specifications as revisions reflecting the results of the test pavements.

What most distinguishes these first specifications from the specifications of ordinary paving asphalt is that they provide for 60°C viscosity and use 14,000 ± 4,000 poise as the very high target value for it. This value was adopted for the following reasons:

1) The 60 C viscosity of the ordinary paving asphalt is 1,500 - 4,000 poise and even asphalt with a penetration of 40/60 of which the viscosity is highest in this group shows hardly any flow-resisting effect, compared with others. so, two to three times the 4,000 poise, namely, about 10,000 poise is considered necessary.

2) Judging from the past properties of paving asphalt remarkable for its cracks, using 20,000 poise or more for 60°C vescosity involves the danger of cracking.

3) This is the first attempt to adjust the 60°C viscosity of asphalt to a prescribed value of 10,000 poise or more by the blowing process. So, an error of 4,000 poise is unavoidable in in view of the accuracy of manufacture. As above, the figure of 14,000 ± 4,000 was temporarily set at the time by weighing various factors. So, it was adjusted by the results of the second and third test paving.

(2) Outline of Test Pavement

Test paving for a total area of about 120,000m^2 was carried out on the following three occasions with the cooperation of the Ministry of construction's various regional construction bureaus, Tokyo Metropolis construction bureau, Kanagawa Prefecture civil engineering department and Tokyo Expressway Public Corporation:

1st: FY 1977 9 places 37 engineering districts 41,538m^2
2nd: FY 1978 14 places 53 engineering districts 46,255m^2
3rd: FY 1981 11 places 11 engineering districts 32,171m^2

(3) 1st Test Pavement

Semi-blown asphalt with the first specifications set in this chapter as the target was manufactured and, paving of the foregoing scale was conducted, using this asphalt. For reference, asphalt with 8,000 poise as the target was also tested. Regarding types of mixtures, dense graded asphalt concrete and coarse graded asphalt concrete were mainly used for, respectively, the surface course and the base course and the asphalt content in most mixtures was the design asphalt content prescribed by the Manual for Asphalt Pavement.

The results of the test paving revealed that semi-blown asphalt is, indeed, effective for flow resistance. But as early cracking occurred at many-points and as we studied the situation, we found that, in most cases, the 60°C viscosity of the original asphalt (asphalt prior to heating and mixing) was in excess of its specified value. We also found that cracking was remarkable when the ratio of the 60°C viscosity of the original asphalt after the thin film oven test (to be referred to hereafter simply as viscosity ratio) was large. Thus, it was necessary to newly provide for viscosity ratio besides viscosity itself.

(4) 2nd Test Pavement

We set the viscosity ratio at less than 6.0 (second specifications) by referring to the results of the 1st test pavement, manufactured semiblown asphalt, taking utmost care to be in strict accordance with the 60°C viscosity specifications, and conducted the 2nd test paving. The results of this test were generally satisfactory with respect to not only flow resistance, but also crack prevention. However, detailed observation of individual data disclosed that there were the following tendencies:

1) Within the viscosity ratio specifications of less than 6.0, cracks developed if the ratio exceeded 5.0.
2) Cracks developed where the 60°C viscosity exceeded 15,000 poise.
3) Flow developed where the 60°C viscosity was less than 7,000 poise.

(5) 3rd Experimental Paving

From the results of the past test pavement, 60°C viscosity and viscosity ratio were revised to, respectively, 10,000 ± 2,000 poise and less than 5.0 as quality specifications (third specifications). These ranges of 60°C viscosities and viscosity ratios are believed to be appropriate as they take crack resistance into consideration while stressing flow resistance at the same time. But to confirm this, a 3rd test paving based on these specifications was carried out. The principal results of perfomance obser-

vation are shown in Fig.-1.6 to 9.

Fig.-1.6 shows the change of rutting for the cummulative traffic volume of heavy vehicles (N). In this figure, the results of the 29th Ministry of Construction Technical Research Congress are indicated with a dotted line for comparison with straight asphalt. The data, though with variations, clearly attest to the flow resisting effect of semi-blown asphalt and the amount of rutting can be estimated at about 1/2, compared with the case using straight asphalt.

In Fig.-1.7, changes in the amount of cummulative traffic volume of heavy vehicles and the cracking ratio by 60°C viscosity are indicated with average lines. In Fig-1.8, changes in the amount of rutting and cracking ratio by 60°C viscosity are indicated. The chart shows that rutting decreases with the increase of 60°C viscosity but that, the 60°C viscosity increases, the cracking ratio sharply increases. Thus, it can be said that, to prevent both rutting and cracking, it is desirable for 60°C viscosity to be adjusted to about 10,000 poise.

Fig.-1.9 shows the 60°C viscosity of extracted asphalt and its dynamic stability. It seems that the correct ranges of DS taking both rutting and cracking into consideration can be found from these charts.

1.6 Conclusion

The results having been obtained from this study are as follows:

(1) The study has clarified from the environments of pavement why flow is particularly remarkable in Japanese pavement.

(2) The study has indicated that it is logical for 60°C viscosity, a property of asphalt, to be used as the yardstick for the flow resistance as asphalt mixtures.

(3) The study has clarified that asphalt with 60°C viscosity increased to a prescribed value (semi-blown asphalt) can be manufactured by blowing.

(4) The study has developed semi-blown asphalt and proved by test paving that it is, indeed, resistant to flow.

(5) The study has clarified the appropriate range of 60°C viscosities taking resistance to both flow and cracking into account.

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2. Research and Development of Rubberized Asphalt for Flow and Wear-Resistant Pavement

2.1. Preface

The Japanese climate is remarkable for its distinct changes in temperature during the four seasons of the year and greatly differs between north and south and between the Japan Sea side and the Pacific side because of its topography which stretches north to south and holds long mountain ranges in the middle. Thus, climatic conditions in Japan vary widely by season and by district and the capacities required of asphalt pavement differ from district to district. Specifically, developing pavement with high flow resistance in hot summertime for warm districts and pavement with high wear resistance to vehicles equipped with tire chains or studded tires in winter-time for cold, snowy districts is a task requiring immediate attention. The research and development of rubberized asphalt designed for flow- and wear-resistant pavement is being conducted to meet this requirement of asphalt pavement. Briefing on part of it took place on the occasion of the first mini-workshop, "Paving in Cold Areas," Canada/Japan Science and Technology Consultations. And this is a follow-up report on the further progress made later by the research and development.

2.2. Research Method

In developing the rubberized asphalt for flow- and wear-resistant pavement (to be referred to hereafter as new binder), the target values for flow resistance and wear resistance were set as indicated in Table-2.1, for convenience use. Also, in developing the new binder, its physical property target values were set as indicated in Table-2.2, on the assumption that those values could well be realized for practical ends in view of the standard properties required of rubberized asphalt by the Manual for Asphalt Pavement, the standards for the surface pavement of Honshu-Shikoku bridges (draft plan) and the physical properties of improved asphalt sold on the market.

Tests for the research and development of the new binder were conducted for three years: FY 1981∿1983, as indicated in Table-2.3. Of these, FY 1981 research on the new binder was reported at the first mini-workshop[1]. So, here we shall see the results of research and development carried out in FY 1982∿1983. Since this report cannot cover all details for the reasons limited space, we shall only consider the important parts of the study. This extraction may make the report appear inconsistent and ifficult to understand in many respects and we must ask to be excused for this.

2.3. Mixture Test Using New Binders (first)

The range from which to select new binders was decided in Fig.-2.1 from the results of the FY 1981 study concerning binders. How the decision was made was reported in the first mini-workshop.

This report (FY1982) covers mixture tests conducted, using such binders as straight asphalt (to be referred to hereafter as "stas") and improved asphalt on the market, to comparatively study the improving effects of new binders according to the above-mentioned decision.

Table-2.4 shows the types of binders used for mixture tests while Table-2.5 shows related test items and methods.

In our mixture tests, hard sandstone was used as aggregates and the type of mixture used was dense graded asphalt concrete (maximum size of aggregates: 13mm). The results of these mixture tests were as follows:

2.3.1. Marshall Stability Test

From the results of the Marshall test in Table-2.6, stability at the optimum asphalt content. (OAC) is larger with mixtures using new binders than with mixtures using stas; thus, new binders were found to have the effect of improved asphalt. As a whole, OAC was within the range of 5.4∿5.7% (values prescribed by the Manual for Asphalt Pavement: 5∿7%) and, though it was a special rubberized asphalt, it either equalled or fell behind mixtures using stas in the range of asphalt contents.

2.3.2. Wheel Tracking Test

Fig.-2.2 shows how the relation between the moonie values of SBR and NR and their dynamic stabilities (to be referred to hereafter as DS) based on the results of a wheel tracking test designed to determine the flow resistance of mixtures appears when it is divided by the amount of rubber that is added. It can be seen from this that, in the SBR system, the moonie value is large and DS increases with the amount of rubber that is added. Particularly the case with a moonie value of 150 is characterized by the fact that, unlike the others, it has an exceedingly large DS value. NR, meanwhile, has a considerably smaller DS value than the SBR system although its moonie value is high at 200. Thus, the difference between polymer types has become distinct. DS also differs by the type of straight asphalt and there is also difference in DS after the addition of rubber. Thus, we have realized the importance of selecting raw material straight asphalt.

2.3.3. Ravelling Test

Fig.-2.3 shows the results of a ravelling test designed to determine the wear resistance of mixtures. Here, wear after a six-hour test on stas E60/80 is 1.62m^2 and expressed by ratios with this as 100%. From this, rubberized asphalt using rubber with a high, though varied, moonie value clearly has an improving effect since it reduces wear.

A high correlation was found to exist between the density of a mixture and its wear ($r = -0.91$) and we learned that a 1% increase of compaction had the improving effect to reduce wear by 0.3m^2. It can, therefore, be said that increasing density and compaction for the rubberized asphalt mixture, as for ordinary mixtures is important for wear-resistant pavement.

2.3.4. Bending Test

Table-2.7 shows the results of a bending test. It indicates that the bending strength was 1.1∿1.4 times, compared with the stas (65∿75kg/cm^2) in the low-temperature zone (-100°C) and the strain was about 1.3 times in the low-temperature zone, compared with the stas (6×10^{-3}∿7×10^{-3}), and about 85% in the normal temperature zone (10∿15°C), compared with the stas (19×10^{-3}∿22×10^{-3}). Further, in stiffness the value was approximate to that of the stas. Thus we have learned that low-temperature bending strength and strain are properties conductive to the improvement of cracking since these properties related to the cracking of pavement increase when rubber is thus added.

2.3.5. Splitting Test

Table-2.8 shows the results of a splitting test. It indicates that, when tensile strength was compared by a splitting test, it was 1.1∿1.2 times at -10°C and 1.2∿1.4 times at 60°C, thus attesting to an improving effect.

2.3.6. Workability Test

The new binder used is high-viscosity asphalt with great toughness and tenacity and is, therefore, expected to be more difficult to work on than the stas when making mixtures. So, we studied its workability, using a mixing torque meter illustrated in Fig.-2.4.

Test results concerning the stas and E70-7 are shown in Fig.-2.5 as examples of test results. The chart indicates that the torque is larger for E70-7 than for the stas at less than 160°C (meaning that E70-7 is more difficult to mix). But at 160°C, the torque is generally constant and, even considering errors, it can be presumed that uniform mixtures can be obtained if, say, 180°C is used for the mixing. It seems therefore, that the new binder

can be worked on even below the mixing temperature obtained from the temperature-viscosity curve drawn by a Saybolt Furol viscosity test.

2.3.7. Conclusion

We have learned from the results of our mixture test (first) using new binders that these binders are fairly promising from the view point of flow- and wear-resistance.

Table-2.9 lists mixtures that are characteristically effective in improving and those which are not effective. From the evaluation of these, E150-5, E125-7 and E70-7 can be selected as desirable binder combinations. However, E150-5 and E125-7 are uneven in melting with asphalt and have exceeding high-temperature viscosity. So, inevitably the final choice is E70-7. Though there is no problem about its solubility with asphalt, E70-7 considerably exceeds the stas in high-temperature viscosity and, considering its workability, there seemed to be room for improvement.

2.4. Mixture Test Using New Binders (second)

Asphalt with a 7% content of SBR latex having a moonie value of 70 was selected by a mixture test using new binders (first test) as a combination satisfying both the development target and the mixture requirements. But this new binder exceeded the stas in high-temperature viscosity and, therefore, involved extremely high mixing temperature as far as we could presume from the viscosity/temperature curve. Thus, we learned that, from the view point of workability, it was necessary to search for binder improvement aimed to reduce high-temperature viscosity.

In the admixture test using new binders (second test), therefore, we developed binders stressing reduction of high-temperature viscosity for the convenience of workability while maintaing the basic physical properties of E70-7, conducted an admixture test using something with a great improving effect and determined its characteristic value.

2.4.1. Test to Develop and Select New Binders

A binder developing and selecting test was conducted on 37 samples consisting of different combinations of rubber types and doses. And of these, the following were selected as binders maintaining the basic physical properties of E70-7 and having reduced high-temperature viscosity:

1) A composition of stas 60/80 plus a 5% addition of SBR latex with a moonie value of 70 and a 1% addition of SBR latex with a moonie value of 150. (To be referred to hereafter as E70-5/140-1)

2) A composition of stas 60/80 plus a 5% addition of hot polymerized SBR latex with a moonie value of 50 and a 1% addition of SBR latex with a moonie value of 150. (To be referred to hereafter as E50-5(H)/140-1.

2.4.2. Heating Separation Test on New Binders

New binders involved the fear of uneven solubility to asphalt because of a high moonie value and the great amount used. So, we conducted hot separation test in the laboratory to evaluate their properties. In the test, the new binders, when kept hot, showed greater top-bottom separation than the stas. But we learned that this separation could be prevented by reducing storage temperature or by stirring while in storage. So, this must be remembered in connection with the kettle storage of new binders having been manufactured at the plant.

2.4.3. Admixture Test

The mixtures underwent Marshall stability, and wheel and ravelling tests. They were tested similarly to the mixture test (first). A total of four binders was used: namely, two with reduced high-temperature viscosity - E70-5/140-1 and E50-5(H)/140-1 - plus two others: stas 60/80 and E70-7 used to compare the improving effects of the first two types.

Hard sandstone was used as aggregates, as in the case of the first mixture test. Three types of mixtures were provided for the test. They were dense graded asphalt concrete (20) (to be referred to hereafter as D(20)), dense graded asphalt concrete (13) (to be referred to hereafter as D(13)) and fine and gap graded asphalt concrete (13F) (to be referred to hereafter as FG(13F). The results of the mixture test using these werw as follows:

2.4.3.1. Marshall Stability Test

The OAC characteristic values in the Marshall stability test are shown in Table-2.10. From this, one can see that the stability of the new binders is not much larger in the case of FG(13F) than that of the stas but, in the case of D(20) and D(13), it is considerably larger and these binders have the improving effect. The OAC values of these binders are much the same as those of the stas in mixtures of the same grading.

When the water resistance of the new binders is evaluated from the results of a retained stability test, the retained stability of the new binders is 80∿93%, compared with 60∿74% for the stas. Thus, the new binders can be expected to have improved water resistance in the future.

2.4.3.2. Wheel Tracking Test

The results of wheel tracking tests using different binders are shown in Figs. 2.6~10. The following can be said from these results:

1) Relation Between Binder Content and Dynamic Stability

In both the stas and the new binders, DS changes with the change of asphalt content. Regarding the trends of change, DS decreases in some parts where the binder amount is small but most often DS decreases in proportion to the increase of asphalt content. This tendency is strong in the D(20) stas.

2) Relation Between Mixture Type and Dynamic Stability

The relation between mixture type and DS is that, with the exception of FG (13F) E50-5(H)/140-1, DS in all binders is high in the order of D(20)>(D(13) >FG(13F).

DS in mixtures using new binders is within the range of 3,000~7,000 times/mm versus 240~1,400 times/mm for the stas DS; thus showing a considerable improving effect. Further, whereas the difference by mixture types is large in the stas, in the new binders the difference is small. Also, DS is smaller in the two types with newly reduced high-temperature viscosity than in E70-7 but their values are more than 3,000 times/mm and they are fairly promising binders as far as flow resistance is concerned.

3) Relation Between Mixture Compaction and Dynamic Stability

DS peaked on the plus side of 100% compaction and the value declined to 20~70% of the peak at 98% and 102% compaction shifted form 100% by ±2%. In actually working with the new binders, therefore, attention must be paid to this fact and care must be taken to obtain a right degree of compaction.

2.4.3.3. Ravelling test

Figs. 2.11~15 show the results of ravelling tests using the new binders. The following can be said from these results:

1) Relation Between Binder Content and Wear

It is said that, generally, the wear of mixtures decreases as the binder content increases. The stas shows the general tendency that, with both D(20) and FG(13F), wear decreases as binder content increases. By contrast, the new binders do not show the clear binder content-wear relation of the stas in any mixture. On the contrary, their tendency is that they are hardly affected by binder content.

2) Relation Between Mixture Type and Wear

In a ravelling test, wear six hours after the conduct of the test is used for evaluation but we also measured for reference sake wear 12 and 24 hours

after the test. Here, the wear of mixture types is evaluated from the wear six hours after the test.

The relation between mixture type and wear is that there is little difference of wear between D(20), D(13) and FG(13F).

Any mixture using a new binder wore 30∿50% less than the stas, thus showing a considerably improving effect. The wear of the two types of mixture with reduced high-temperature viscosity nearly equalled the wear of E70-7: thus, both types were found to be fairly promising in wear resistance as in flow resistance.

3) Relation Between Mixture Compaction and Wear

The tendency with all binders and all mixture types was that wear decreased as mixture compaction increased. This tendency is remarkable particularly with D(20) stas and E70-7 and FG(13F). By types of mixtures, the effect of compaction is greater with D(20) than with FG(13F).

2.4.4. Conclusion

We were able to see from the results of a mixture test using new binders (second test) that the two new binders: E70-5/140-1 and E50-5(H)/140-1 that were developed and selected as something having equivalent basic physical properties to E70-7 and also having high-temperature viscosity reduced for the convenience of work performance have equal or even better admixture properties than E70-7.

2.5. Quality Standards (Draft) for New Binder (Tsukuba No.1) Designed for Flow- and Wear-Resistant Paving

The quality standards (draft) for a new binder developed for use in flow- and wear-resistant paving have been decided as indicated in Table-2.11, based on research results accumulated for three years. And the new binder satisfy these quality standards (draft) has been named Tsukuba No. 1.

The procedure of paving work using Tsukuba No. 1 is not mentioned here but it was prepared as indicated in Table-2.3: Progress of Research and Development.

2.6. Conclusion from Three Years of Research and Future Tasks

The research and development of rubberized asphalt for flow- and wear-resistant paving was conducted for three years from FY 1981 to FY 1983. As the result, we were able to prepare not only the quality standards (draft) for Tsukuba No. 1 believed to fully accomplish the using targets listed at the beginning of this study as development targets but also a guide on paving work using this binder.

However, this study did not include confirmation of the applicability of Tsukuba No. 1 to the real road. It is, therefore, urgently necessary to carry out experimental work with Tsukuba No. 1 as soon as possible, check its workability and applicability and see if its quality standards (draft) are appropriate.

Acknowledgement

We take this opportunity to express our gratitude to those who permitted data for this study to be cited from the results of the "Research Concerning the Development of Rubberized Asphalt for Flow- and Wear-Resistant Paving," a joint research conducted by three organizations: Public Works Research Institute of the Ministry of Construction, Public Works Research Center (a foundation) and Japan Rubberized Asphalt Association[2].

References

1) Tada, Makita, Iijima and Sakamoto: Modified Asphalt in Japan and Research into the Development of New Rubberized Asphalts, the Proceedings of the First Mini-Workshop, "Paving in Cold Areas", Canada/Japan Science and technology Consultations, October 1982.

2) Research Committee Concerning Rubberized Asphalt for Flow- and Wear-Resistant Paving: Research Report Concerning the Development of Rubberized Asphalt for Flow- and Wear-Resistant Paving, FY 1981, FY 1982 and FY 1983.

Table-1.1 Comparison of Pavement Improvement Level

Nations	Road Length (1000Km)	Length of Paved Road (1000Km)	Percentage of Paved Roads (%)
U.S.A.	6,304	5,169	82.0
W. Germany	482	419	86.9
U. Kingdom	352	340	96.6
France	803	738	91.9
Japan	1,111	178	16.0

Note: Based on IRF 1981. Japanese Data is based on the Annual Report on Road Statistics in 1981 and excludes low cost pavement.

Table-1.2 Quantity of Overlay Work on National Highways

Regional Construction Bureau	Ovly for Crack (Km)	Ovly for Rutting			Ovly for Wear (Km)	Total (Km)
		Ovly (Km)	Cut & Ovly (Km)	Total (Km)		
Tohoku	16	3	93	93	-	112
Kanto	36	3	31	34	6	76
Hokuriku	5	10	13	23	20	48
Chubu	22	14	41	55	11	88
Kinki	31	11	59	70	-	101
Chugoku	37	-	31	31	-	68
Shikoku	24	3	3	6	-	30
Kyushu	58	1	23	24	-	82
Total	229	45	294	339	37	605
(%)	37.9	7.4	48.6	56.0	6.1	100

Note: Based on the statistics by Road Bureau of Ministry of Construction in 1981

Table-1.3 Comparison of Road Width

Nations	Road Class	Length		Percent	Year
U.S.A	Federal Highways of the 1st Class	Total	416,750		1978
		W>7.3m	335,489	80.5	
W. Germany	Federal Highways	Total	32,490		1976
		W>7m	21,420	65.9	
France	National Highways	Total	29,104		1977
		W>7m	22,380	76.9	
Japan	National Highways	Total	39,946		1980
		W>7.5m	6,521	16.3	

Note: 1.Reference
U.S.A. "Highway Statistics 1978"
W. Germany "Verkehr in Zahlen 1978"
France "Annuaire Stat ist igue pes Transport 1978"
Japan "All Japan Traffic Survey 1980"
2.Expressways are excluded and presumption is included.
3.Heavy vehicles can pass each other easily on the roads of 7m wide.

Table-1.4 Congestion of Major Roads

Category	Length of Each Degree of Congestion Km					Vehicles-Kilometer		
	Total	0.5-	0.8-	1.0-	1.5-	Total A	Congested B	A/B (%)
National Highways	39,946 (100)	27,111 (68)	18,003 (45)	12,677 (32)	4,394 (11)	284,843	156,371	55
Prefectural Highways	130,166 (100)	79,074 (61)	54,644 (42)	44,208 (34)	28,084 (22)	322,622	147,689	46
Total	170,112 (100)	106,185 (62)	72,647 (43)	56,885 (33)	32,478 (19)	607,465	304,060	50

Note: 1."All Japan Traffic Survey 1980"
2.Vehicles-Kilometer on congested sections means that the sections are of more than 1.0 degree of congestion.
3.The degree of congestion 1.0 means that traffic volume is equal to traffic capacity.
4.If degree of congestion is 1.0-1.5, traffic jams mainly occur at the peek time in the morning and the evening. If it is more than 1.5, jams last all day long.
5.Figures in ( ) is the ratio calculated by cumulative number.

Table-1.5 Number of Retained Motor Vehicles and Quantity of Running

Nations	Length of Paved Roads A ($\times 10^3$Km)	No. of Vehicles B ($\times 10^3$V)	No. of Heavy Vehicles C ($\times 10^3$V)	D=C/B (%)	E=B/A (V/K)	F=C/A (V/K)	Annual Running G ($\times 10^8$V·K)	H=GxD ($\times 10^8$V·K)	I=H/A ($\times 10^4$V)
U.S.A.	5,169	154,118	33,870	22.0	29.8	6.6	24,250	5,335	10.3
W. Germany	419	24,769	1,533	6.1	59.1	3.7	3,259	199	4.7
U. Kingdom	340	16,951	1,878	11.1	49.9	5.5	2,774	308	9.1
France	738	21,780	2,650	12.2	29.5	3.6	3,030	370	5.0
Japan	178	37,874	14,214	37.5	212.8	79.9	4,380	1,640	92.1

Note: 1.V:Vehicles K:Kilometer
2.IRF Edition 1981 and Annual Report on Road Statistics 1981 (Japan)
3.Length of paved roads includes prefectural and municipal roads excluding low cost pavements.

Table-1.6 Results of Tests of Asphalt on the Market and Experimental Asphalt

Materials / Tests	St. As on the Market			Semi-blown Asphalt						
	80/100	60/80	40/60	4000poise		6000poise			16000poise	
	No.1	No.2	No.3	No.4	No.5	No.6	No.7	No.8	No.9	No.10
Penetration (25°C,100g, 5 sec)	92	68	55	53	82	46	64	64	45	60
Softning Point (°C)	46.3	50.2	50.2	53.0	50.8	57.6	53.8	56.7	59.5	58.7
Ductility (15°C,cm)	150+	150+	150+	48	50	15	7	7	11	5
Weight Change Rate After Heating (%)	-0.02	-0.02	+0.01	+0.01	+0.01	+0.01	-0.01	-0.02	-0.01	-0.01
Penetration After Heating (%)	91.0	92.6	94.2	96.5	91.7	93.8	93.8	98.4	95.0	94.9
Penetration Ratio After Heating (%)	100.0	100.0	100.0	101.8	102.6	102.2	101.7	100.0	102.0	100.0
Weight Change Rate After TFOT (%)	+0.05	+0.06	-0.11	-0.06	+0.02	-0.04	+0.05	+0.09	+0.03	+0.09
Penetration Ratio After TFOT (%)	59.8	61.8	74.5	69.5	63.4	73.9	79.7	76.6	71.1	76.7
Solubility in Carbon Tétra-chloride (%)	99.68	99.86	99.98	99.93	99.89	99.66	99.77	99.88	99.76	99.96
Flash Point (°C)	316	316	350	286	288	336	304	306	312	290
Specific Gravity (25/25°C)	1.0245	1.0299	1.0379	1.0311	1.0122	1.0326	1.0001	1.0090	1.0314	1.0117
Viscosity at 60°C (poise)	1,290	2,430	3,110	4,220	4,530	9,100	8,350	8,380	14,550	13,430

Note TFOT:Thin Film Oven Test

Table-1.7 Results of Component Analysis on Asphalt

Items	Semi-blown Asphalt					As. on the Market		
	1st Test Pavement		2nd Test Pavement		3rd T. P.	St. As.		Blown
	Original	Extracted	Original	Extracted	Original	40/60	60/80	20/30
Viscosity at 60°C	16,200	201,000	15,500	460,000	9,500	3,500	2,200	8.9x10
Asphaltenes %	18.6	25.9	18.0	27.3	18.4	14.4	14.3	26.3
Resin	17.9	20.7	17.6	21.4	18.0	21.6	19.8	22.9
Aromatic Hydro-cbn	39.2	33.5	39.5	31.6	38.9	46.3	47.3	31.2
Saturated Hydro-cbn	24.0	15.6	24.4	15.9	24.2	16.7	18.2	16.1
M_w(2)	2,000	2,340	1,940	2,220	2,070	1,360	1,400	2,490
M_n(3)	780	820	740	790	770	710	750	890
M_w/M_n(4)	2.56	2.85	2.62	2.78	2.69	1.94	1.87	2.80

Note (1) Average Molecular Weight
(2) Weight Average Molecular Weight. It shows the existance of molecular with large molecular weight
(3) Number Average Molecular Weight. Average by Number of Molecules.
(4) Mw/Mn means the degree of distribution of molecular weight.
(5) Asphalt was extracted from surface course.

Table-1.8 Standard of Semi-blown Asphalt

Standard No. / Items	1st		2nd	3rd
	AC-140	AC-80	AC-140	AC-100
Viscosity (60°C,poise)	14,000 4,000	8,000 2,000	14,000 4,000	10,000 2,000
Viscosity Ratio (After TFOT/ Before TFOT)	-	-	6.0 or Less	5.0 or Less
Dynamic Viscosity (180°C,cSt)	200 or Less		200 or Less	200 or Less
Weight Change Rate After TFOT (%)	0.6 or Less		0.6 or Less	0.6 or Less
Penetration (25°C,100g, 5sec)	40 or More		40 or More	40 or More
Solubility in Trichloroethan (%)	99.0 or More		99.0 or More	99.0 or More
Flush Point (°C)	260 or More		260 or More	260 or More
Specific Gravity	1.000 or More		1.000 or More	1.000 or More

Note TFOT:Thin Film Oven Test

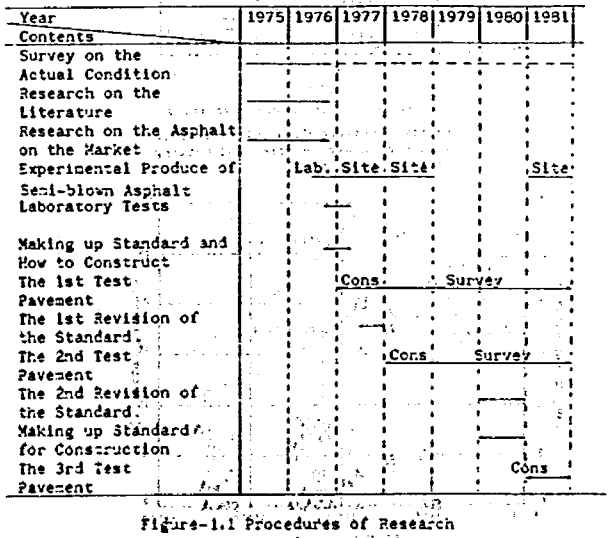


Figure-1.1 Procedures of Research

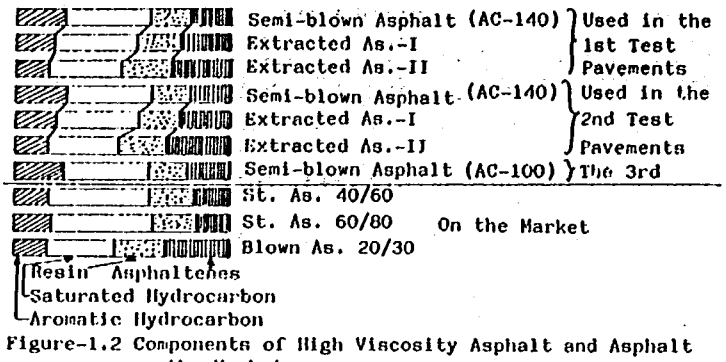


Figure-1.2 Components of High Viscosity Asphalt and Asphalt on the Market

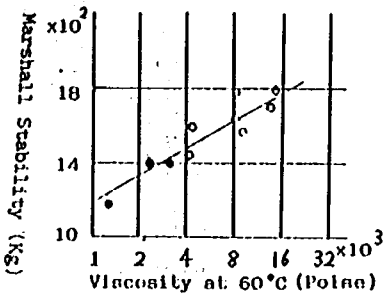


Figure-1.3 Relation Between Viscosity of Original Asphalt Marshall Stability

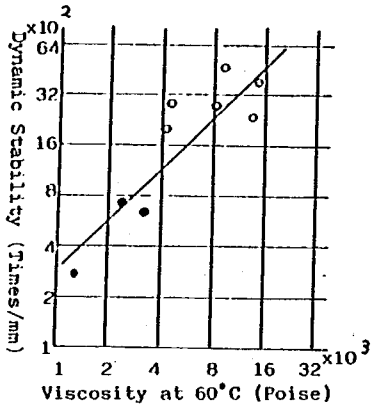


Figure-1.4 Relation Between Viscosity of Original Asphalt and Dynamic Stability

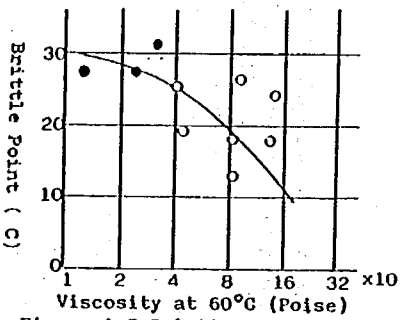


Figure-1.5 Relation Between Viscosity of Original Asphalt and Brittle Point of Mix

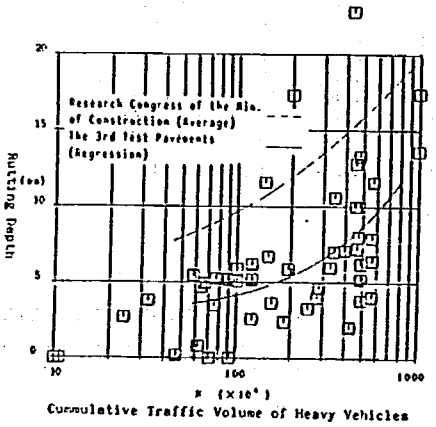


Figure-1.6 Relation Between Cumulative Traffic Volume of Heavy Vehicles and Rutting Depth (Intersection)

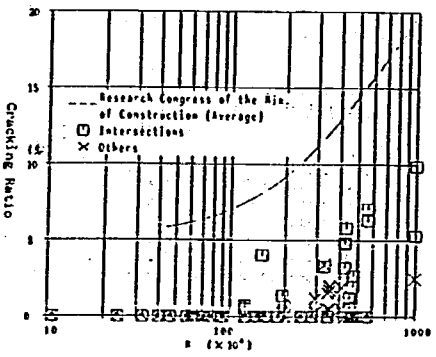


Figure-1.7 Relation Between Cumulative Traffic Volume of Heavy Vehicles and Cracking Ratio

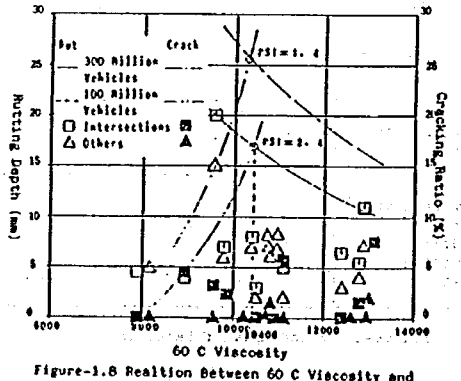


Figure-1.8 Realtion Between 60 C Viscosity and Rutting Depth or Cracking Ratio

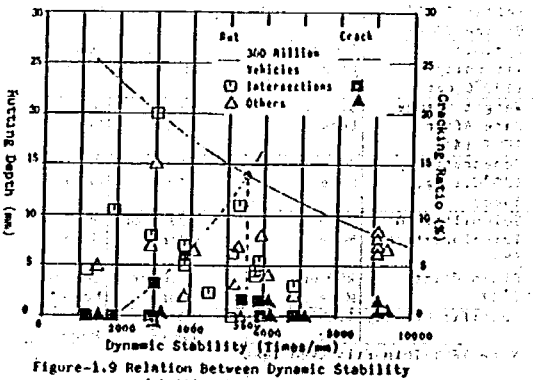


Figure-1.9 Relation Between Dynamic Stability and Rutting Depth or Cracking Ratio

Table-2.1 Target Values of Performance of New Binder

(1) Anti-Fluid and Anti-Crack
(Cumulative number of heavy vehicles is 10 million.)

Items	Measurement	Place	Present Condition	Target Value	Remark
Anti-Fluid	Rutting (mm)	Intersection	20	10 or Less	Target value is 50% of present damage.
		Others	12	6 or Less	
Anti-Crack	Cracking (%)		17	9 or Less	

Note 1) Road, Jan. 1981
2) Report of the 29th Technical Research Congress, Ministry of Construction

(2) Anti-Wearing
(Cumulative nunber of heavy vehicles are 1 million.)

Items	Measurement	Place	Present Condition	Target Value	Remark
Anti-Wearing	Wearing (mm)	-	15	12 or Less	Target value is 80% of present damage.

Note 3) Road, Aug. 1980

Table-2.3 Procedure of Development of New Binder

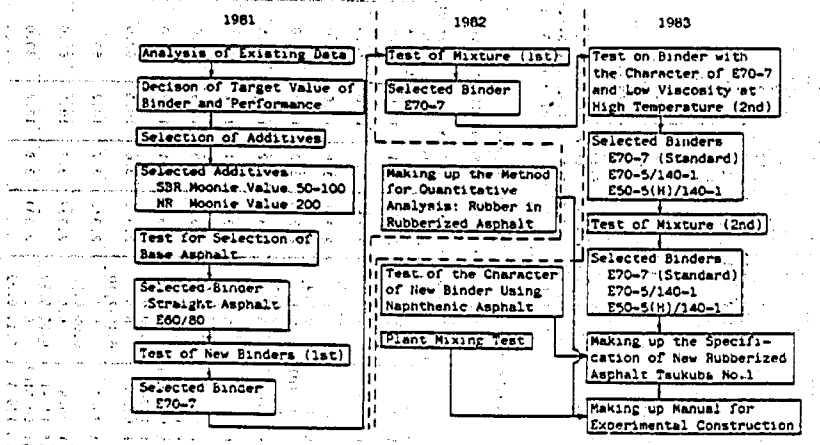


Table-2.2 Target Value of Character of New Binder

Items	New Binder Pre-mix Type		Rubberized As. on the Market Pre-mix		Standard for Rubberized As. Pre-mix		Standard for Hoshu-Shikoku Bridge Pre-mix	
	Target	Rem.	Rubber	Resin	60/80	80/100	I	II
Pen. (25°C, 100g,5sec)	50-100		65	65	60-80	80-100	60-100	60-100
Softening Point (R&B) °C	57 or More	PI=0.5 ~2.2	50.5	57.0	48.0-56.0	46.0-54.0	55-65	60-75
Ductility (7°C) cm	50 or Less		65	100 or More (25 C)	20 or More	50 or More	50 or More 1)	10 or More 1)
Pen. Ratio After Heating %	110 or Less		102	-	110 or Less	110 or Less	-	-
Pen. Ratio After TFOT %	65 or More	180 C 2.5 hr	75.2	87.2	55 or More	55 or More	65 or 2) More	65 or 2) More
Fraas Breaking Point °C	-12 or Less		-	-18	-	-	-12 or Less	-12 or Less
Toughness (25°C) Kgxcm	100 or More		91	138	60 or More	50 or More	120 or More	30 or More
Tenacity (25°C) Kgxcm	50 or More		56	87	30 or More	25 or More	100 or More	10 or More
Visco- 60°C Poise 4) sity 200°C cSt 5)	6000 or More 300 or More		4180 71	5750 120			4000 or More 200 or More 3)	16000 or More 200 or More 3)

Note 1) Temp. 10°C, 2) 180°C, 2.5Hr, 3) Unit SFS (cSt=2.16SFS) 4) CGS units are used.
5) Viscosity/Density=Dynamic Viscosity 6) TFOT:Thin Film Oven Test

Table-2.4 Kinds of Binders Used

Asphalt	Pen.	Rubber	Moonie Value	Addition (%)	Symbol
Σ	60/80	-	0	0	E 60/80
		SBR	50	5	E 50-5
			70	5	E 70-5
				7	E 70-7
			100	5	E 100-5
			125	5	E 125-5
				7	E 125-7
			150	3	E 150-3
				5	E 150-5
		NR	200	5	E NR-5
A		-	0	0	A 60/80
		SBR	70	7	A 70-7
D	40/60	-	0	0	D 40/60
Improved Asphalt on the Market	Rubberized Asphalt				G-c
	Resin Added Asphalt				F-a
	Rubber & Resin Added Asphalt				M-b

Table-2.5 Tests on Asphalt Mix and Methods

Test	Test Method	Test Condition
Marshall Stability Test	As. Pavement Manual	75 Blows to Each Side
Wheel Tracking Test	As. Pavement Manual	60°C 6.4Kg/cm
Ravelling Test	Public Works Research Institute Method	-10°C, Blow by Chains
Bending Test	Hoshu-Shikoku Bridge Pavement Standard	Span 20cm, Velocity of Strain 6.25×10^{-2}/sec
Splitting Test	Public Works Research Institute Method	Thickness 4.5cm,Loading Velocity 50mm/min
Workability Test	Mixing Torque Meter Method	Rotation 34, 46 and 68 r.p.m.

Table-2.6 Marshall Test Value at O.A.C.

Binder	O.A.C. % Common Range	Density g/cm³	Void Ratio %	V.M.A. %	Degree of Sutura-tion %	Stabi-lity Kg	Flow Value (1/100cm)	Stiff-ness (S/F)
E 60/80	5.7 5.25-6.20	2.364	4.0	17.0	77	1,370	31	44
E 50-5	5.5 5.10-5.90	2.370	3.8	16.5	77	1,480	29	51
E 70-5	5.5 5.05-5.95	2.369	4.0	16.6	76	1,580	30	53
E 70-7	5.5 5.10-5.80	2.371	3.8	16.5	77	1,570	33	48
E 100-5	5.6 5.20-5.90	2.364	4.0	16.8	76	1,600	34	47
E 125-5	5.7 5.15-6.25	2.362	4.0	17.1	78	1,460	33	44
E 125-7	5.4 5.05-5.75	2.373	3.9	16.4	75	1,530	36	43
E 150-3	5.5 5.05-5.90	2.369	4.0	16.6	76	1,350	30	45
E 150-5	5.6 5.25-6.00	2.359	4.1	17.0	75	1,560	34	46
E NR-5	5.5 5.05-5.90	2.366	4.2	16.8	75	1,490	29	51
A 70-7	5.6 5.15-6.00	2.369	4.0	16.8	76	1,470	30	49
A 60/80	5.6 5.20-6.05	2.364	4.2	16.9	75	1,190	24	50
D 40/60	5.7 5.25-6.05	2.367	3.8	16.9	77	1,370	34	40
G-c	5.6 5.10-6.15	2.361	4.1	16.9	76	1,450	32	45
H-b	5.5 5.15-5.90	2.363	4.3	17.0	75	1,560	31	50
F-a	5.6 5.10-6.10	2.365	4.0	17.0	76	1,440	31	46

Table-2.7 Results of Bending Test of Various Binders (Pre-mix type)

Binder	Items	Temperature (°C)				
		-10	0	5	10	15
E 60/80	Strength	64.7	75.3	85.7	92.7	66.5
	Strain	6.04	7.08	9.55	18.78	21.85
	Stiffness	11.14	10.78	8.98	5.12	3.08
E 50-5	Strength	72.6	104.1	129.5	75.6	61.6
	Strain	7.62	9.12	12.13	15.85	17.83
	Stiffness	9.64	11.43	10.67	4.78	3.46
E 70-5	Strength	75.3	80.1	120.7	101.5	65.6
	Strain	7.26	8.31	12.54	15.30	20.71
	Stiffness	10.64	9.71	9.64	6.23	3.25
E 70-7	Strength	72.7	98.4	119.8	90.7	64.7
	Strain	5.97	9.74	10.27	14.37	18.58
	Stiffness	12.20	10.16	11.69	6.34	3.52
E 100-5	Strength	70.4	90.4	102.6	90.1	70.2
	Strain	5.75	8.62	11.06	6.34	3.52
	Stiffness	12.52	9.74	9.35	4.95	3.66
E 125-5	Strength	76.3	89.2	130.7	97.3	63.7
	Strain	7.50	9.77	14.15	16.81	20.45
	Stiffness	10.50	9.39	9.41	5.82	3.23
E 150-5	Strength	71.5	109.5	108.9	83.4	48.9
	Strain	6.89	9.58	15.67	17.15	23.21
	Stiffness	10.95	11.53	6.99	4.87	2.12
E NR-5	Strength	84.6	105.4	130.9	83.9	57.4
	Strain	7.44	7.77	18.26	21.05	25.93
	Stiffness	11.83	13.61	7.21	4.02	2.25
G-c	Strength	75.3	81.9	127.9	94.7	69.1
	Strain	7.48	7.71	12.91	17.39	19.23
	Stiffness	10.13	10.68	10.02	5.56	3.57
H-b	Strength	74.6	94.8	103.2	119.5	86.7
	Strain	7.64	9.64	10.42	13.38	17.02
	Stiffness	9.76	9.83	9.90	8.93	5.09
F-a	Strength	82.9	98.6	141.6	104.9	78.3
	Strain	5.65	7.52	12.50	17.02	17.21
	Stiffness	14.66	13.22	11.37	6.21	4.60

Unit: Streingth Kg/cm²; Strain x10⁻²
Stiffness x10³Kg/cm²

Table-2.8 Results of Splitting Test of Various Binders (Pre-mix Type, Average)

Binder	Items	Temperature (°C)				Remark	
		-10	20	40	60	5°C	12°C
E 60/80	Load	2,593	1,019	200	86	2,413	1,713
	Deformation	1.05	2.40	1.70	1.60	1.51	2.07
	Strength	36.74	14.53	2.85	1.22	34.53	24.35
E 50-5	Load	2,657	1,044	256	111	2,650	1,650
	Deformation	1.11	2.08	1.72	1.69	1.91	1.97
	Strength	41.17	14.98	3.66	1.61	37.94	23.66
E 70-5	Load	2,980	1,101	278	113	2,607	1,917
	Deformation	1.28	2.20	1.83	1.57	1.52	2.07
	Strength	43.17	15.82	3.98	1.62	37.15	25.99
E 70-7	Load	2,762	1,033	287	121		
	Deformation	1.23	1.98	1.90	1.68		
	Strength	39.77	14.78	4.10	1.74		
E 100-5	Load	2,763	1,081	269	105		
	Deformation	1.04	2.19	1.68	1.47		
	Strength	39.85	15.49	3.88	1.51		
E 125-5	Load	2,748	1,060	260	102		
	Deformation	1.23	2.18	1.62	1.57		
	Strength	39.09	15.21	3.71	1.46		
E 150-5	Load	2,678	1,030	281	118		
	Deformation	1.14	2.17	1.75	1.56		
	Strength	37.96	14.72	4.00	1.68		
E NR-5	Load	3,155	999	234	95		
	Deformation	1.24	2.32	1.52	1.37		
	Strength	45.23	14.26	3.33	1.36		
G-c	Load	2,972	1,015	242	95		
	Deformation	1.10	2.13	1.55	1.34		
	Strength	42.40	14.44	3.44	1.34		
H-b	Load	3,200	1,226	302	130		
	Deformation	1.23	1.78	1.52	1.33		
	Strength	45.94	17.55	4.27	1.86		
F-a	Load	3,133	1,021	295	126		
	Deformation	1.41	1.92	1.50	1.33		
	Strength	45.16	14.59	4.23	1.82		
Cross Section at Breaking		Crack of Stones & Brittle Breaking of Mix	Crack of Stones & Ductile Breaking of Mix	Ductile Breaking of Mix	Ductile Breaking of Mix	No Difference Between Binders	

Unit Load(Kg), Deformation(mm), Strength(Kg/cm²)

Table-2.9 Evaluation of New Binder by Tests on Mixes

Tests		Test Result	
		Much Improvement	Little Improvement
Marshall Stability Test	Retained Stability	E 70-5(Plant-mix) E NR-5	E 150-5(Plant-mix)
Wheel Tracking Test	Dynamic Stability	E 150-5, E 125-7 E 70-7	E NR-5
Ravelling Test	Wearing	E 125-7, E 70-7 E 150-3, A 70-7	E 70-5
Bending Test	Brittle Temp.	E 150-5	E 100-5
	Breaking Strain	E NR-5	-
Splitting Test	Tensile Strength at 60 C	E 70-7 E 150-5	-
	Deformation at -10 C	E 70-5, E 70-7 E 125-5, E NR-5	E 100-5
Workability Test	Optimum Mixing Temperature (Assumed through Torque)	E 70-7	E 100-7 E 125-5 E 150-5

Table-2.10 Results of Marshall Stability Test at O.A.C.

Items / Mix	Binder	O.A.C. % Common Range	Density g/cm³	Void Ratio %	V.M.A. %	Degree of Satura-tion %	Stabi-lity Kg	Flow Value (1/100mm)	Stiff-ness (100kg/cm)
Dense Graded Asphalt Concrete (20)	E 60/80	5.1 4.75-5.45	2.381	4.1	15.8	74.5	1,360	26	57.0
	E 70-7	5.1 4.70-5.45	2.390	3.7	15.6	77.0	1,600	30	51.0
	E 70-5 E 140-1	5.1 4.70-5.40	2.391	3.8	15.6	76.0	1,630	29	57.0
	E 50-5(H) E 140-1	5.1 4.65-5.55	2.388	3.8	15.7	76.0	1,500	30	50.0
Dense Graded Asphalt Concrete (13)	E 60/80	5.8 5.35-6.30	2.355	4.2	17.5	75.5	1,190	25	45.1
	E 70-7	5.7 5.30-6.10	2.357	4.1	17.2	76.0	1,410	27	51.2
	E 70-5 E 140-1	5.7 5.20-6.10	2.364	3.8	16.9	77.0	1,450	29	49.0
	E 50-5(H) E 140-1	5.7 5.30-6.10	2.355	4.1	17.2	76.0	1,420	27	49.5
Fine & Gap Graded Asphalt Concrete (13F)	E 60/80	6.6 6.15-7.10	2.330	3.6	18.5	80.0	1,400	24	58.0
	E 70-7	6.4 6.05-6.75	2.340	3.7	18.2	80.0	1,590	25	64.0
	E 70-5 E 140-1	6.5 6.05-6.90	2.337	3.7	18.5	80.0	1,450	29	47.0
	E 50-5(H) E 140-1	6.5 6.10-6.90	2.334	3.8	18.6	80.0	1,460	30	49.0

Table-2.11 The Quality Standard of Tsukuba-No.1 (Draft)

Tsukuba-No.1 is rubberized asphalt of pre-mix type containing 4-10 weight percent SBR latex which is a mixture of rubber with low and high moonie value.

Items		Criteria	Test Method
Penetration (25 C, 100g, 5sec)		50-100	JIS K 2207
Softning Point (R&B) C		57 or More	JIS K 2207
Ductility (7 C) cm		50 or More	JIS K 2207
Pen. Ratio After Heating %		110 or Less	JIS K 2207
Pen. Ratio After TFOT %		65 or More	Standard of Honshu-Shikoku Bridge
Fraas Breaking Point C		-10 or Less	JIS K 2207
Toughness (25 C) Kgxcm		100 or More	Benson Method
Tenacity (25 C) Kgxcm		50 or More	Benson Method
Visco-sity	60°C Poise	10000 or More	Public Works Research Institute Method
	200°C cSt	300 or Less	JIS K 2207

Note JIS: Japanese Industrial Standard
TFOT: Thin Film Oven Test

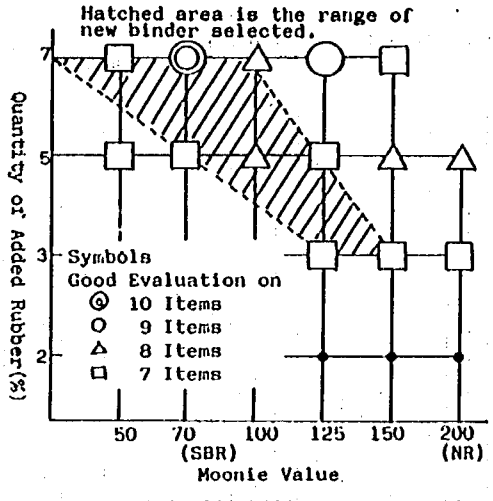


Figure-2.1 Applicability of New Binders

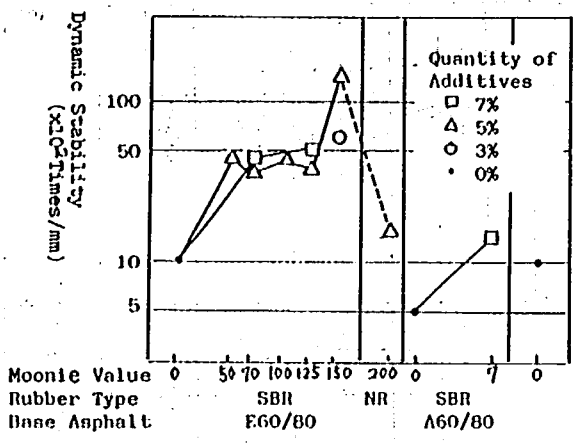


Figure-2.2 Dynamic Stability of Asphalt Mixes Containing Various Binders (O.A.C.,Pre-Mix)

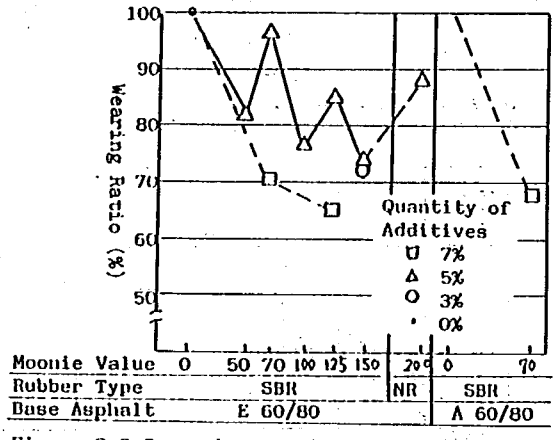


Figure-2.3 Comparison of Anti-Wearing Character to Basic Asphalt Mix

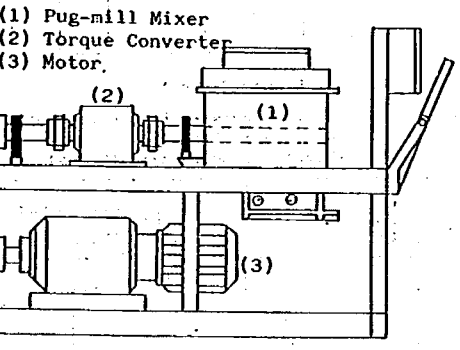
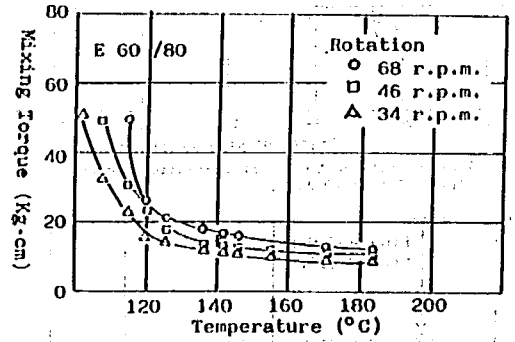


Figure-2.4 Mixing Torque Meter



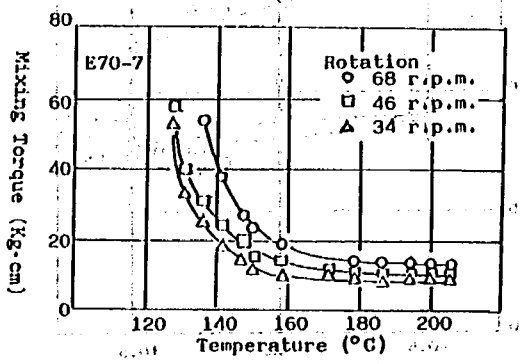


Figure-2.5 Relation Between Torque and Temperature

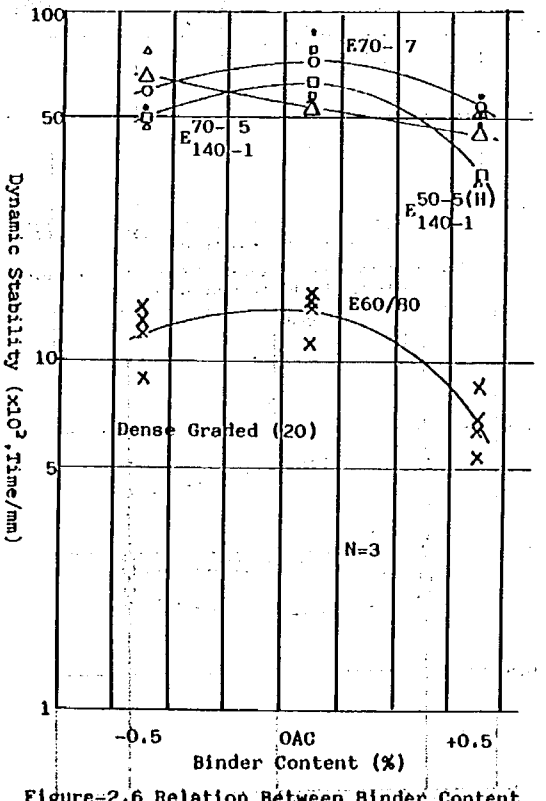


Figure-2.6 Relation Between Binder Content and Dynamic Stability (Degree of Compaction 100±1%)

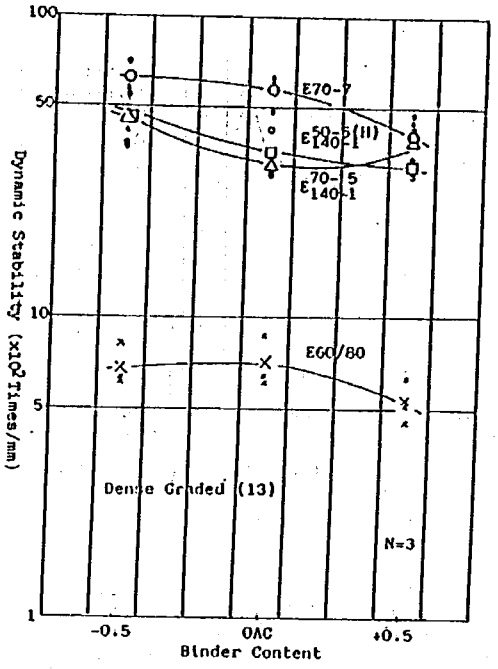


Figure-2.7 Relation Between Binder Content and Dynamic Stability (Degree of Compaction 100±1%)

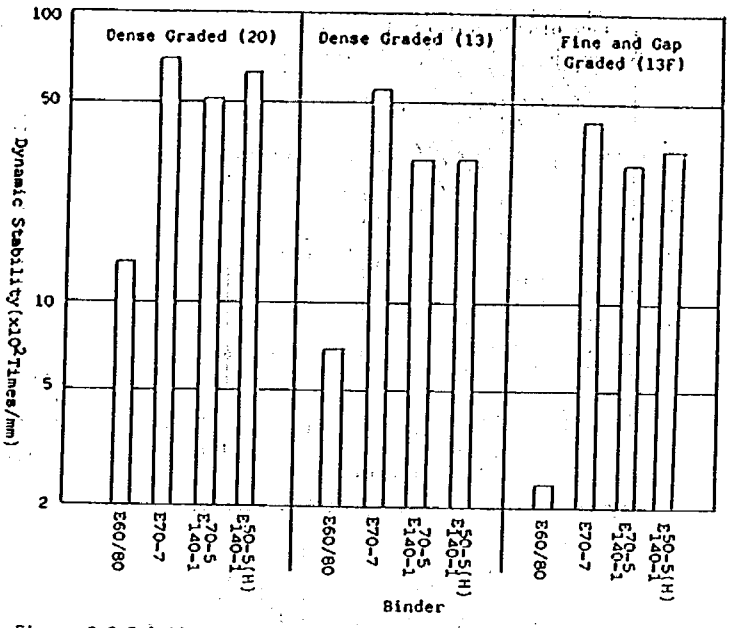


Figure-2.8 Relation Between Types of Mix and Dynamic Stability (O.A.C.,Degree of Compaction=100±1%,60°C,6.4Kg/cm²)

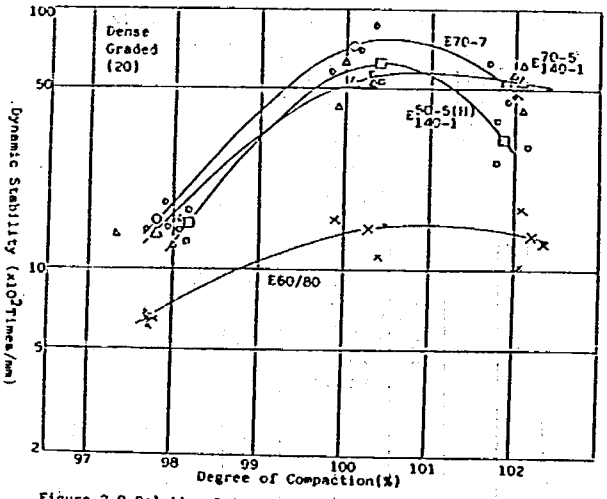


Figure-2.9 Relation Between Degree of Compaction and Dynamic Stability (Binder Content=O.A.C.)

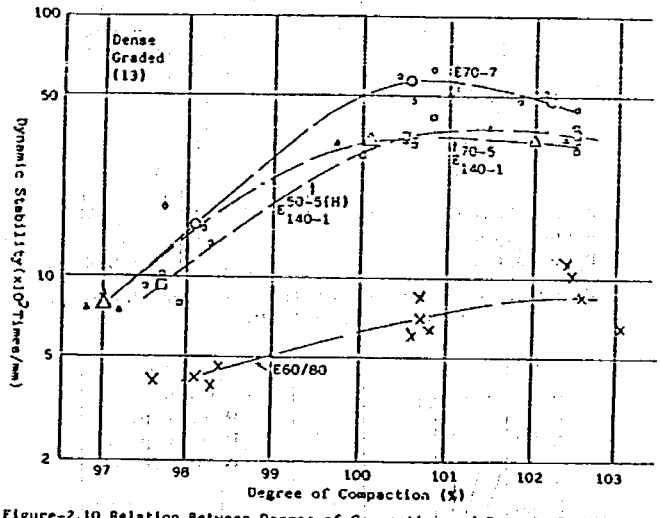


Figure-2.10 Relation Between Degree of Compaction and Dynamic Stability (Binder Content=O.A.C.)

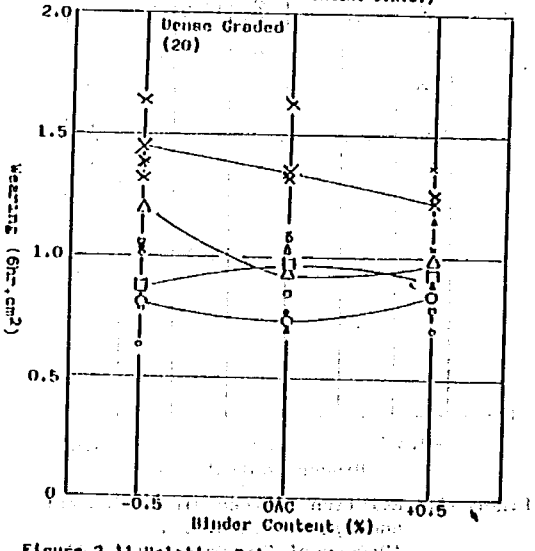


Figure-2.11 Relation Between Binder Content and Wearing (Degree of Compaction 100±1%)

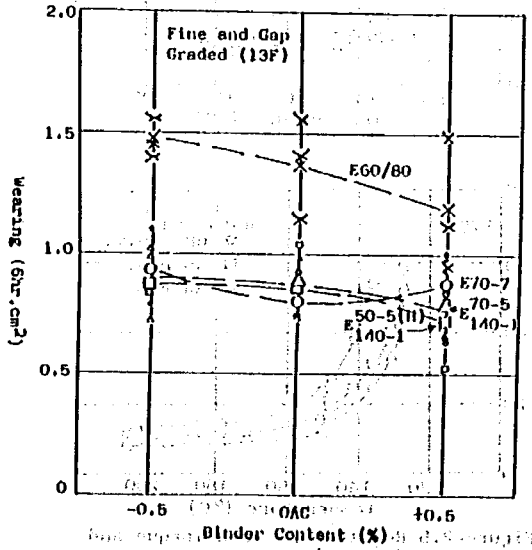


Figure-2.12 Relation Between Binder Content and Wearing Wearing (Degree of Compaction 100±1%)

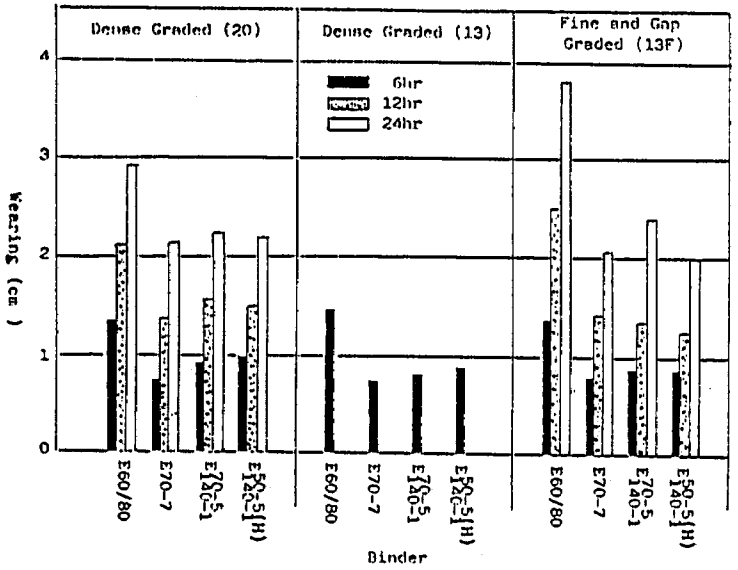


Figure-2.13 Relation Between the Type of Mix and Wearing (O.A.C., Degree of Compaction 100±1%,-10°C)

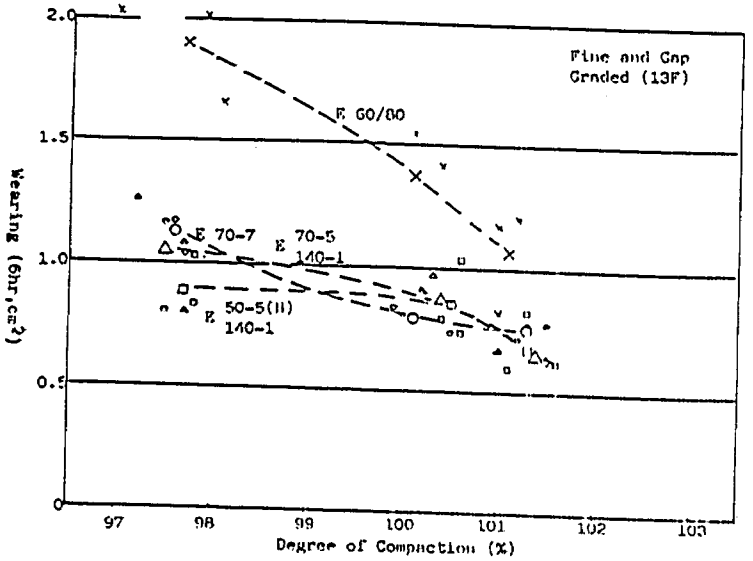


Figure-2.15 Relation Between Degree of Compaction and Wearing (Binder Content=O.A.C.)

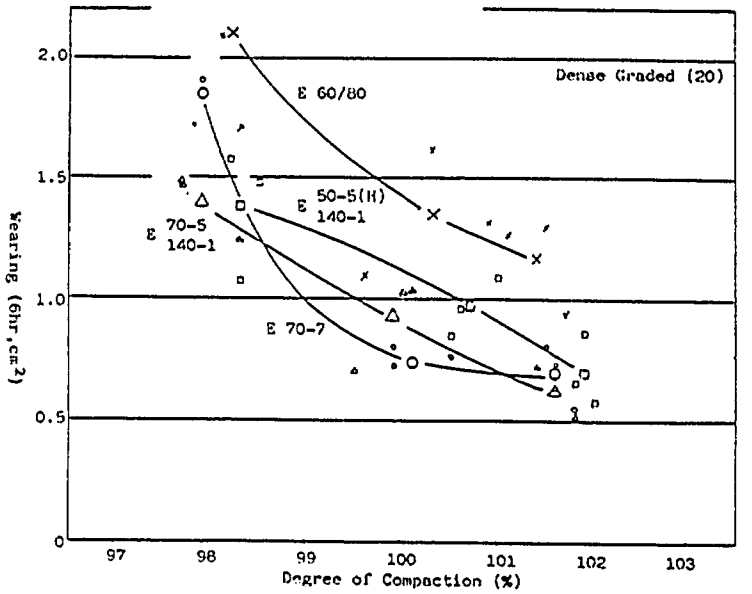


Figure-2.14 Relation Between Degree of Compaction and Wearing (Binder Content=O.A.C.)

Asphalt Pavement Overlays on Airports in Cold Regions

Mr. K. O. Anderson
University of Alberta
Mr. H. Heimark
Transport Canada

ABSTRACT

Numerous airports have been built within the past 30 years or so in the area under jurisdiction of the Western Region of Transport Canada. Many of these have asphalt pavement surfaces that have had or will require rehabilitation due primarily to roughness developed under the severe climatic conditions experienced in these regions. A description of pavement condition evaluation techniques used to establish the need for rehabilitation and construction history records on some projects is presented.

This paper is based on a runway overlay research project undertaken to study methods of asphaltic pavement rehabilitation with a view to minimizing reconstruction problems and costs, and extending the life expectancy of overlays. The scope of the work included a review of current literature regarding methods and materials for pavement improvement as well as several case histories of runways overlaid since 1980. A major part of the work involved a runway overlay project in 1983 at Yellowknife, N.W.T.

Problems encountered with overlays placed on badly cracked pavements include relatively large amounts of reflective cracking shortly after construction as well as during placement of the overlay. The joint sealing compound used to fill the cracks of the old pavement sometimes causes slippage and roughness to appear in the newly placed overlay. Several types of preoverlay preparation have been used to alleviate some of these problems. These types have included cold-milling, heater scarification and selective crack treatment. A test section using a non-woven polyester type geotextile was placed on the Yellowknife project.

Conclusions based on experience gained on recent overlays and the 1983 runway overlay project are given. Comparisons are made with other related studies concerning asphalt concrete pavements in cold regions.

ASPHALT PAVEMENT OVERLAYS ON AIRPORTS IN COLD REGIONS

K.O. Anderson
University of Alberta

and

Harry Heimark
Transport Canada

INTRODUCTION

Asphalt overlays provide one of the most cost-effective methods of improving existing pavements. Asphalt overlays can be used to strengthen existing pavements, to reduce maintenance costs and increase pavement life, to provide a smooth ride, to improve service to the user, and reduce safety hazards by improving pavement surface skid resistance. While these statements, contained in a recent publication of The Asphalt Institute (1) concern asphalt overlays for highway and street rehabilitation, they apply as well to airfields.

The objective of this runway overlay research project was to study methods of asphaltic pavement rehabilitation with a view to minimizing reconstruction problems and costs, and extending the life expectancy of overlays. The research program was undertaken prior to and in conjunction with the work to overlay runway 15-33 at Yellowknife Airport, N.W.T. during the summer of 1983.

The scope of the work included a review of current literature regarding methods and materials for pavement rehabilitation as well as inspection of runways overlaid within the past two or three years at Calgary International, Fort Smith and Hay River airports. This was in addition to the major part of the research project in connection with the work to improve runway 15-33 at Yellowknife.

A report of this runway overlay research project has been summarized and presented at a recent Specialty Conference on Cold Regions Engineering, held in Edmonton in April of this year (2,3).

OVERVIEW OF AIRPORT FACILITIES IN THE WESTERN REGION

Background

Many of Transport Canada's northern airports in the Western Region were built in the 1940's in support of the Canol Project and the Alaska Highway. The former project involved airports at Ft. Smith, Hay River, Yellowknife and Norman Wells. The latter involved Grande Prairie, Ft. St. John, Ft. Nelson, Watson Lake and Whitehorse. In the post-war years the thrust into the north was fuelled by natural resource development and the Federal Government's requirement for communication and transportation facilities in support of its expanding northern social programs.

The 1950's and early '60s saw the reconstruction of most of these small airports and the construction and paving of a number of new ones. In the late '60s, the emphasis shifted from provision of new facilities to the maintenance and upgrading of existing facilities. About this same period an equally important development involved the introduction of jet aircraft to almost all major northern centers. The implications to airport designers and operators were far reaching. Whereas light propellor aircraft could operate safely from rudimentary airfields, the faster and heavier jets required a much stronger and smoother surface. In addition, aircraft such as the popular Boeing-737, because of their low slung engines, are particularly susceptible to engine damage from ingestion of loose materials on the pavement surface. In other words, a runway which was eminently suitable for the DC-4 might prove totally unacceptable for 737 operations.

A map showing the current airports in the Western Region is shown in Figure 1. Except for the two International airports at Edmonton and Calgary, operations are such that the pavements are not heavily trafficked. The National airport at Yellowknife serves as a hub for many flights into the Arctic. In 1981 there were in the order of 25 average daily movements of jet and other heavier aircraft. Despite the relatively major level of activity in the MacKenzie Valley Corridor, only 5 or so average daily

movements of these aircraft were estimated for the two busier Regional airports of Normal Wells and Inuvik, for the period of 1960-1975.

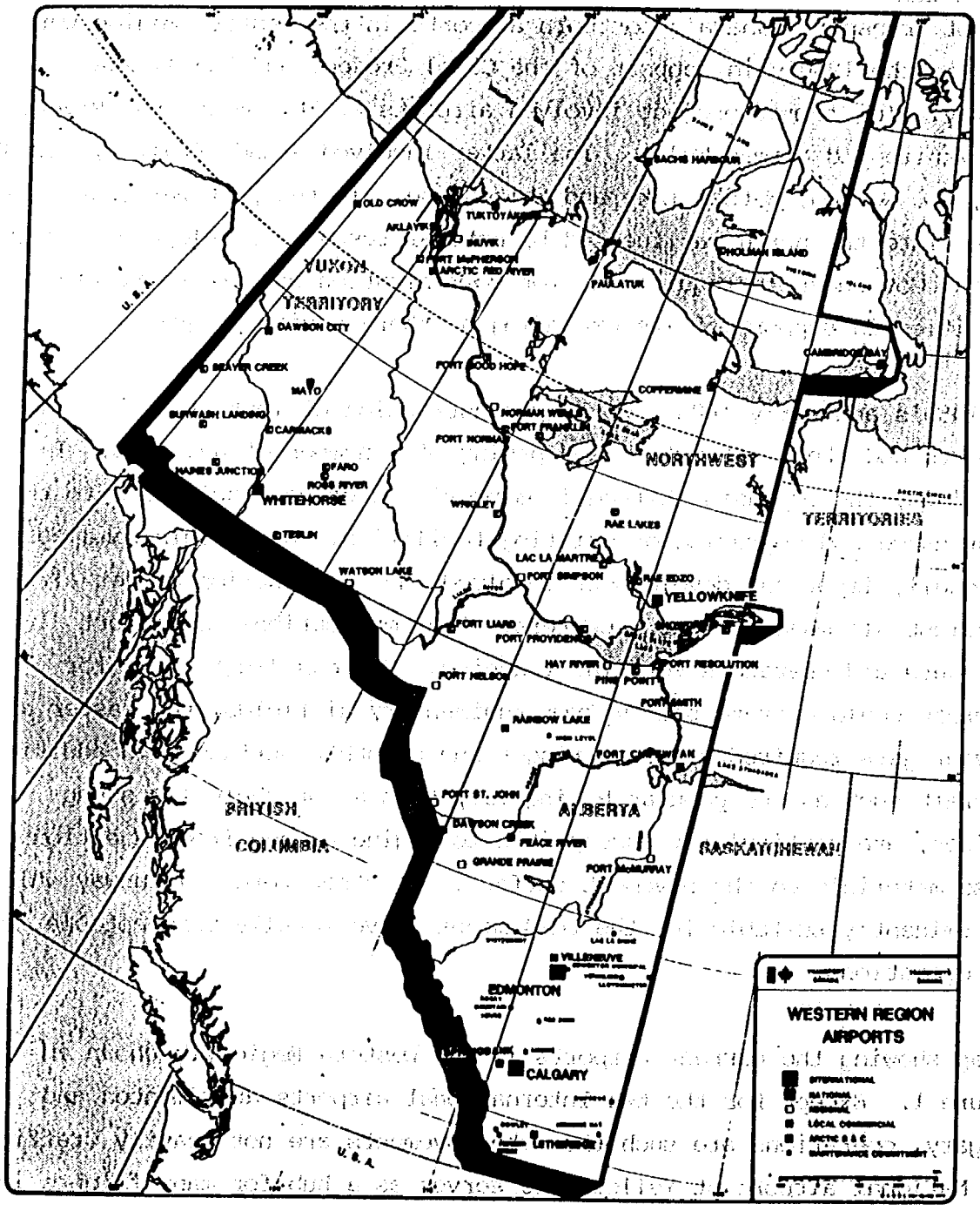


Figure 1. Western Region Airports - Transport Canada

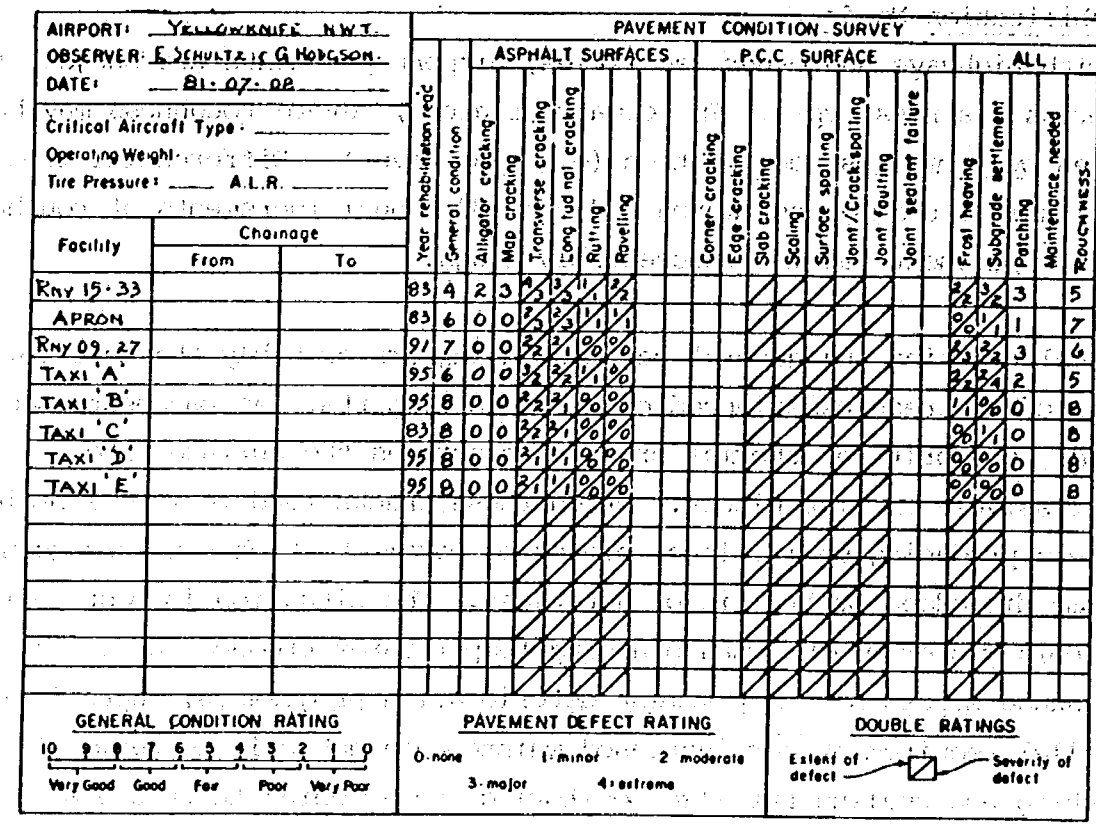
Rehabilitation Needs

As with highways and other facilities, airport pavements are subject to numerous distress types. In permafrost areas, severe roughness may be caused by permafrost degradation (e.g. Hay River). Differential frost heaving is a common phenomenon and can add annual increments of roughness until a pavement becomes unacceptably rough.

Thermal cracking and general pavement aging are probably the most common forms of distress experienced on northern airports. As pavements age and become more brittle, thermal cracking separates the surface into progressively smaller pieces. Often these cracks tend to form a braided pattern producing small, loose pieces of pavement which can subsequently become dislodged by traffic or jet blast. The situation is most serious in the cold winter months when plunging temperatures produce large and sudden shrinkage stresses. As pavement deterioration progresses, there is a parallel requirement for increased maintenance not only in terms of patching and sealing but also monitoring and sweeping to ensure a safe, debris-free operating surface.

Pavement Inspection and Condition Reporting

At Transport Canada Airports, maintenance personnel make inspections of the aircraft maneuvering areas at least once and perhaps as often as 15 times daily, depending on the amount and type of traffic, the age and condition of the pavement, weather and other factors. In addition to the maintenance checks, a more formal and in-depth inspection is carried out biennially at most sites and annually at Calgary and Edmonton International Airports. An example following standard procedures (4) is given in Figure 2. These "condition surveys" are carried out by professional and technical personnel. The reports identify the various types of distress and assign a numerical value to each type. Over the years, the reports can be studied chronologically to trace the decrease in pavement performance and to calculate the remaining useful years and life expectancy. A summary of airfield pavement data inventory for a typical airport such as Yellowknife, is given in Figure 3, as described in the latest procedures manual (5).



AIRPORT: YELLOWKNIFE N.W.T.
OBSERVER: E SCHULTZ & G HODGSON
DATE: 81-07-08
Critical Aircraft Type:
Operating Weight:
Tire Pressure: A.L.R.

PAVEMENT CONDITION SURVEY

Facility	Chainage From	Chainage To	Year rehabilitation req'd	General condition	ASPHALT SURFACES: Alligator cracking	Map cracking	Transverse cracking	Long. & trans. cracking	Rutting	Ravelling	P.C.C. SURFACE: Corner cracking	Edge cracking	Slab cracking	Scaling	Surface spalling	Joint/Crack spalling	Joint faulting	Joint sealant failure	ALL: Frost heaving	Subgrade settlement	Patching	Maintenance needed	Roughness
RNY 15-33			83	4	2	3	4/3	3/2	1/1	2/2									2/2	3/2	3		5
APRON			83	6	0	0	2/2	2/2	1/1	1/1									0/0	1/1	1		7
RNY 09-27			91	7	0	0	3/2	2/1	0/0	0/0									3/3	2/2	3		6
TAXI 'A'			95	6	0	0	3/2	2/2	1/1	0/0									2/2	3/3	2		5
TAXI 'B'			95	8	0	0	2/2	2/1	0/0	0/0									1/1	0/0	0		8
TAXI 'C'			83	8	0	0	2/2	2/1	0/0	0/0									0/0	1/1	0		8
TAXI 'D'			95	8	0	0	2/1	1/1	0/0	0/0									0/0	0/0	0		8
TAXI 'E'			95	8	0	0	2/1	1/1	0/0	0/0									0/0	0/0	0		8

GENERAL CONDITION RATING
10 9 8 7 6 5 4 3 2 1 0
Very Good — Good — Fair — Poor — Very Poor

PAVEMENT DEFECT RATING
0-none 1-minor 2-moderate 3-major 4-extreme

DOUBLE RATINGS
Extent of defect / Severity of defect

Figure 2. Typical Form for Pavement Condition Survey

AIRFIELD PAVEMENT DATA INVENTORY — Date - [illegible] /82 — AIRPORT - YELLOWKNIFE N.W.T.
TECHNICAL SUMMARY — Page - 1 OF 2 — SITE NUMBER - W - 154

Facility	Plan Code	Chainage(m)	Area ($10^3 m^2$)	Surf Type	Construction History: Layer Thickness (cm) PCC	AC	B	SB	Contract Number	Year	Est'd Rehab Year	Pavement Strength: Code	Streng. PLR	Assign. PLR
RNY 15-33	2	3+048.0 TO 3+139.4	4.2	A.C.		7.6	15.2		56202	1956	1983	S267t23	12(0.4)	12(1.0)
"	1	3+139.4 TO 4+663.4	69.7	A.C.		7.6	15.2		51668	1954	1983	S267t23	12(0.4)	12(1.0)
"	2	4+663.4 TO 5+335.1	30.7	A.C.		7.6	15.2		56202	1957	1983	S267t23	12(0.4)	12(1.0)
RNY 09-27	6	3+048.0 TO 3+627.1	26.5	A.C.		7.6 6.4	15.2		51668 76-154-1	1954 1976	1991	S267t36	12(0.7)	12(1.0)
"	6	3+672.8 TO 4+571.9	41.2	A.C.		7.6 6.4	15.2		51668 76-154-1	1954 1976	1991	S267t36	12(0.7)	12(1.0)
TAXI A	1	0+304.8 TO 0+518.2	4.9	A.C.		7.6	15.2		51668	1954	1983	S267t23	12(0.4)	12(1.0)
TAXI B	4	3+931.9 TO 4+282.4	8.0	A.C.		8.9	15.2		73-154-1	1973	1995	S267t31	12(0.4)	12(1.0)
"	5	4+282.4 TO 4+622.4	7.8	A.C.		8.9	15.2		76+154-1	1976	1995	S267t33	12(0.4)	12(1.0)
TAXI C	5	0+327.7 TO 0+472.4	4.7	A.C.		8.9	15.2		76-154-1	1976	1995	S267t33	12(0.4)	12(1.0)
TAXI D	5	9+156.4 TO 9+428.2	6.2	A.C.		8.9	15.2		76-154-1	1976	1995	S267t33	12(0.4)	12(1.0)

Figure 3. Airfield Pavement Data Inventory Summary

Aircraft Pavement Load Ratings

Airport pavement load ratings (PLR) define the aircraft which, from a pavement strength point of view, may operate at the airport without prior approval of the airport authority. The PLR are on a scale of 1 to 12 and are usually designated PLR xx. A tire pressure restriction given in MPa is shown in brackets following the PLR designation, if it applies. The PLR's are based on the subgrade soil strength and the pavement construction. The strength PLR and tire pressure for a runway (taxiway, apron) will be the minimum of all the assigned PLR's and tire pressures for any section of the runway (taxiway, apron). Load ratings assigned to different types of aircraft are based on the maximum and minimum weights of the aircraft, the percentage main gear load, the wheel spacing and the tire pressure. Aircraft may operate on airport pavements without prior approval only if the aircraft load ratings and tire pressure are less than or equal to the pavement load ratings and tire pressure restrictions.

METHODS OF REDUCING REFLECTION CRACKING

Previous experience has indicated that overlays placed on badly cracked pavements could be expected to experience a large amount of reflection cracking shortly after reconstruction. As well there have been problems with joint sealing compound on the surface causing slippage of the newly placed asphalt overlay. These are some of the problems that have been attempted to be corrected by using heater scarification prior to overlay.

Possible Alternative Treatments

In reviewing recent literature it was apparent that there were a number of methods that had been tried with varying degrees of success for controlling the reflection cracking problem in pavement overlays. Considering the extreme climate of Yellowknife and the short time available for planning and implementing any proposed methods for attempting to alleviate the reflection cracking problem, the following methods were the most promising and were given consideration.

Open Graded Hot Mix (Modified Mix). MacMaster and Blum (6) have reviewed various methodologies and techniques which the Ministry of Transportation and Communications of Ontario have employed in recent years to minimize reflection cracking in overlays. They have concluded that the most

effective technique employed to date in its attempt to control reflection cracking has been the use of modified open graded hot mixes as lower binder courses. The gradation of the modified mix is essentially 100% passing the 16 mm size with 5-15% passing the 4.75 mm (No. 4 sieve) and only 0-4% passing the No. 200 sieve. The large percentage of voids in the material helps to dampen the movement of the underlying slabs and reduces the stresses created in the upper binder and wearing courses. This mix has proved to be quite effective on concrete slab overlays and has been used on a cracked bituminous pavements in southern Ontario. The asphalt content of these open graded mixes is substantially lower than in conventional asphalt concrete and was set at approximately 3%.

Hensley (7) has reported on the use of open graded asphalt concrete base for the control of reflective cracking in Arkansas. They refer to their open graded mixes as a "crack relief layer". While they have used thicker overlays the minimum recommended thickness of their crack relief layer is 3 1/2 inches (90 mm approximately). Aggregates used for such crack relief layers have been made up of 100% crushed particles from a hard durable aggregate source. They recommend that the open graded mixture be daylighted, i.e. carried through to the shoulder in order to prevent accumulation of moisture vapour as well as providing free drainage to the outside edge of the pavement. They feel that stripping will not occur in the material as there are no fines in this open graded mixture to cause stripping of the asphalt cement.

Huffman (8) has also referred to the experience in Arkansas and has said that assessment of the ultimate durability of this overlay system is still required but the earliest placed sections have given years of satisfactory service. The total thickness of this system has precluded its use in many instances. Huffman has stated that one of the most promising systems for reflection crack control is an interlayer consisting of asphalt rubber mixture as a stress absorbing membrane (SAM). The principle of this system is that high stresses are relieved by the placement of this thin asphalt rubber membrane of low stiffness and high deformability. Because of the difficulties in implementing a system of this type this was not considered for the Yellowknife overlay project even though it may be promising.

Geotextiles and Fabrics. Use of geotextiles and fabrics as interlayers to control reflection cracking has been tested extensively both in the United States and Canada and many other areas. The results of this method varies with the type of fabric used and the climate of the testing area. Huffman (8) commented,

> "In recent years there has been increased usage of reinforcing fabric membranes for reflection crack control. Interlayers of this type have included fabrics of fiberglass, nylon, polyester, and polypropylene, often non-woven (spun bonded etc.).
>
> "Relative to fabric membrane performance several experimental projects in the United States with this construction indicate reduced reflection cracking after 5 years service. With asphalt pavements, fabric generally has been found not to be effective in stopping the transverse thermal crack reflection in cold climates."

Dykes (9) has also discussed the use of fabric interlayers to retard reflective cracking. He has concluded that "fabric interlayers will not of itself stop thermal cracking in severely cold climates however the sealed membrane remains intact in virtually every case". This was discussing cores taken in Montana and Wisconsin where the pavements had been subjected to cold temperatures. Long term benefits are realized by retarding the occurrence of secondary and advanced stages of cracking.

MacMaster and Blum (6) have also discussed the use of geotextiles in attempting to control reflection cracking in Ontario. Four types of fabrics have been tried namely a) Petromat b) Bidim C-28 c) Bituthene H.D.G. and d) Prepave. They have concluded that these geotextiles have been only marginally successful in a limited number of applications.

Eaton and Godfrey (10) have reported on the results of the attempts beginning in 1977 by the U.S. Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL) to study the rate of reflection cracking of an AC 2.5 asphalt cement overlay on a severely cracked runway at Thule airbase in Greenland. The following year, in 1978, fabrics were placed under another AC2.5 overlay on severely cracked portions of the taxi system

at Thule. CRREL chose to test the Phillips Fibers Corporation 4 and 6 oz. per yard^2 Petromat and the Monsanto 6 and 8 oz. per yard^2 Bidim fabrics. Their conclusions showed that regardless of fabric type or weight, reflection cracking could be expected through the fabric and a 2 in. thick AC 2.5 overlay. Slightly better performance was observed from the Bidim fabric. Eaton and Godfrey have commented further regarding the performance of fabric as follows, "Although fabric type or weight did not significantly influence the rate of reflection cracking in this study, ... the fabrics do serve as a waterproofing membrane." It should be noted that this installation was in an extremely harsh Arctic climate and was in a permafrost area. The Thaw Index is about 510 °F-days with a Freezing Index of over 8200 °F-days.

One of the new materials that has been found to perform satisfactorily in the laboratory under repeated loading is that of the high strength plastic geogrid known as TENSAR. Abdelhalim et al. (10) has described this material as a 50 mm by 50 mm mesh made by stretching a sheet of polypropylene plastic with holes punched through it, in both length and width directions. It has been termed a "geogrid" to differentiate it from the conventional geotextile fabrics. The material is said to have high tensile strength. The report of testing of TENSAR undertaken in 1981 in Ontario showed that the TENSAR mesh was very effective as a reinforcement in an asphalt concrete layer and minimized fatigue cracking. The behavior of TENSAR fiber under low temperatures was tested at the University of Waterloo in 1981. The TENSAR type used in this research was designated as AR-1 which had been developed to have the following properties: 1) high tensile strength 2) high modulus 3) low elongation 4) biaxial structure i.e. strength in both directions 5) a grid with desired width for pavement purposes.

Although TENSAR used as a pavement reinforcement under cold temperatures has not been tested in the field its properties based on laboratory testing suggested that it would be worthwhile trying on an experimental basis for the Yellowknife runway. Unfortunately this could not be arranged within the constraints of time available.

Asphalt Cement Selection. Many papers have been presented that show the importance of the asphalt cement as related to low temperature cracking performance. A recent paper by McLeod (11) summarized many years of experience by saying that the "two most important engineering properties of paving asphalts; penetration at 25°C and temperature susceptibility has been referred to and emphasized".

In a study reported on by Way (12) the prevention of reflective cracking in Arizona was described. Although in Arizona, this was located in a reasonably severe climate in a high desert region where a history of severe cracking problems had been reported. The temperature variations ranged from -18°C during the winter to 38°C during the summer. Some 18 different test sections were developed using various designs that appeared to give the most likely reduction in reflective cracking.

Five treatments were found to have significantly reduced reflective cracking:

- asphalt rubber membrane seal coat under asphalt concrete overlay
- asbestos plus 3% asphalt
- heater scarification with Reclamite (surface recycling)
- asphalt rubber membrane flushed into asphalt concrete overlay
- 200/300 penetration asphalt cement.

The conclusion that the 200/300 penetration asphalt from the Los Angeles basin (low temperature susceptibility) added to a 32 mm asphalt overlay and then covered with a 13 mm asphalt concrete finishing course was found to be one of the most effective treatments to reduce reflective cracking.

CASE HISTORIES

Calgary

In 1980, a section of runway 16-34 was rehabilitated by cold-milling. A strip 20 m in width and approximately 1500 m in length was removed to a depth of 64 mm and replaced with hot mix asphalt concrete. Construction history records show that runway 16-34 was constructed in 1956 with 22.9 cm of Portland Cement concrete. This was subsequently overlaid in 1963 with 10.2 cm of asphalt concrete and again overlaid in 1972 with 12.7 cm of asphalt concrete. Asphalt cement meeting the latest edition of CGSB

Specification 16-GP-3M grade 120-150 was used for the asphalt concrete.

The airfield pavement data inventory classify the soil to be CL(ML). Textural descriptions of the subgrade soil indicate it to be predominantly silty clay with varying amounts of sand and some gravel. The water table is from 1.2-3.1 m in depth and the frost susceptibility is rated high.

Air temperatures for the years from 1941-1969 ranged from a high of 35°C to a low of -43°C. The average maximum was 32.0°C and minimum of -34.3°C. The average Freezing Index was 1073°C-days. A pavement condition survey taken 81-07-08 indicated a general fair to poor condition rating with moderate alligator cracking and major map cracking. Transverse cracking was of extreme extent and major severity. Longitudinal cracking was of major extent and severity, with minor amounts of rutting and moderate ravelling.

Reflection cracking of the transverse cracks was observed to occur. The general condition of the surface in the center 20 m section appears to be quite good. These cracks have been routed and sealed.

Hay River

In 1981 a construction contract was awarded to improve runway 13-31 at the Hay River Airport, North West Territories. The contract essentially consisted of a 40 mm hot mix overlay on the runway which had experienced settlement due to isolated sections of permafrost being degraded. Both levelling courses and cold milling in isolated sections to a depth of 40 mm maximum were used to improve the profile prior to the 40 mm overlay being placed. Construction history records indicate that 914 mm of selected sand plus 229 mm crushed gravel plus 89 mm of hot mix surface was placed in 1967. Asphalt cement of 150-200 penetration grade was used for the original construction and the overlay in 1981.

The airfield pavement data inventory gives the soil classification as CL, ML, and SM. The textural description showed the subgrade soils to consist of sand, silty sand, sandy silt, and clay silt of a heterogeneous mixture. The frost susceptibility was considered high with ice lenses found between 1 m and 19 m. It was considered to be a permafrost site.

The average Freezing Index for this area is about 3150°C-days. The temperatures for the period of record from 1944 to 1970 ranged from a high of 34°C to a low of -47°C. The average minimum temperature was -44.9°C. The average Thawing Index was 1793°C-days.

A pavement condition survey taken in May, 1982 one winter after the overlay placement, indicated transverse cracking of moderate extent and severity, with longitudinal cracking of minor extent and moderate severity. Moderate severity was also noted for frost heaving and subgrade settlement, although the latter was only of minor extent.

Since the areas for cold-milling and levelling course placement were not confined to specific portions of the runway, no conclusions may be drawn concerning the relative merits of these two methods of treatment prior to overlaying.

Fort Smith, N.W.T.

In 1982, a construction contract was called to improve the runway 11-29, Aprons I and II, Taxi's A & C of the Fort Smith Airport, North West Territories. The contract tendered for two construction alternatives.

Alternative A - heater scarify and recompact 19 mm of the existing asphalt concrete and overlay with 65 mm ± hot mix asphalt concrete

Alternative B - plane 38 mm ± of the existing asphalt concrete and overlay with 65 mm ± hot mix asphalt concrete.

As a result of the tender closing, Alternative A, heater scarifying with overlaying was selected for construction. Asphalt cement of 150-200 penetration grade was used for the asphalt concrete. A tack coat of SS-1 emulsion diluted with water at a 1:1 ratio and applied at a rate not to exceed 0.5 ℓ/m^2 was specified. A test section approximately 63.5 m by 14.8 m was left unscarified and overlaid with asphalt concrete.

Construction history records show that runway 11-29, Taxi C and Apron I and II were constructed in 1956 while Taxi A was constructed in 1965. The pavement structure for runway 11-29 consisted of 83.9 cm crushed gravel and

8.9 cm hot mix asphalt concrete, and the pavement structure for Taxis A and C, Apron I and II consisted of 91.5 cm crushed gravel and 8.9 cm hot mix asphalt concrete.

The airfield pavement data inventory indicates the soil classification to be SM, SP. Textural description shows the soil as silt and sand with non-plastic fines. Thin layers of saturated unstable silt are encountered. The water table was not encountered in the first 6 m. The frost susceptibility is considered to be high but only minor heave was reported.

The Freezing Index for this area is about 3150 °C-days. The minimum temperatures since 1944 are -49°C with an average of -46.6°C. The average maximum temperature is 31.2°C.

A pavement condition survey taken in 81-07-07 indicated extensive cracking in Taxi C, Runway 11-29, Taxi A and Apron II. Both transverse and longitudinal cracking were described as major or extreme in extent with major severity. Extreme ravelling was found on the parallel taxi C. Most of the roughness was considered to be built in and longitudinal joints are poorly matched.

The heater scarifying equipment used by the sub-contractor consisted of two tractors which were rebuilt and improvised to perform the necessary operations. Each machine was identical having a refractory lined box at the front equipped with propane burners, two sets of independently hydraulically controlled scarifying teeth in the middle of the machine, and a heated vibratory screed with adjustable side extensions at the rear. The burners and tractor engines were fuelled by propane.

The specified depth of 19 mm was achieved on the average, however the depth did fluctuate, due to low or high areas in the surface. Scarifying with this type of equipment and to this depth did not achieve a high degree of levelling since not enough material was generated to enable this to be done.

The scarified material was compacted by a steel drum roller followed by a pneumatic-tire roller. The scarifying operation averaged 4 850 m^2 per day,

completing 126 122 m^2 in 26 days.

The openness and relatively low asphalt content of the scarified surface tended to absorb the tack coat. This was rectified by applying the tack coat only one or two hours prior to paving thus creating a better bond between the scarified asphalt and the overlay.

The product after scarifying was considered to be favourable, having achieved the initial goal of sealing cracks in the existing asphalt and mixing the excess crack sealant with the top 19 mm of the existing asphalt to prevent bleeding through the overlay. To verify the effects of scarifying, cores were taken through a crack before and after scarifying. The crack before scarifying was sealed approximately two thirds of the way through the asphalt concrete. After scarification the same crack was again cored and found to be sealed through to the base gravel.

Observations made of the pavement condition after one winter shows there is no apparent difference between the overlays placed on scarified or non-scarified pavements. Reflective cracks were seen to progress from near the centerline of the runway through both types of sections.

Summary of Experience

On reviewing the observations and reports made in these three case histories it can be concluded that:

1. Cracks have reflected through asphalt concrete which was placed following partial removal of an old surface by cold-milling methods to a depth of 64 mm.
2. Heater scarification was apparently successful in sealing cracks of the existing surfaces and mixing crack sealant with the top 19 mm of the existing asphalt. This process did not have much effect in levelling the surface, however.
3. There are no obvious differences in the reflection cracks observed on non-scarified as compared to scarified sections of the Fort Smith runway.
4. Partial cold-milling and levelling courses can be used together to help restore desired grades to meet required smoothness criteria.

RUNWAY OVERLAY AT YELLOWKNIFE

Background

The construction contract tendered for 1983 for the Yellowknife Airport, North West Territories was to improve runway 15-33, taxi A, aprons 1 and 2, and to pave portions of the ATB Access Road. The contract called for two alternative construction methods.

Alternative A - levelling course and overlay 75 mm ± for runway and 65 mm ± for apron and taxiway. Hot mix asphalt placed in two compacted layers of 40 mm maximum plus levelling course as required.

Alternative B - heater scarify existing pavement surface, levelling course and overlay. 65 mm ± for runway and 55 mm ± for apron and taxiway. Hot mix asphalt placed in two compacted layers of 40 mm maximum plus levelling course as required.

As a result of the bid prices, Alternative A, the conventional overlay was selected for construction. Asphalt cement of 150-200 penetration grade was specified for use.

Construction History. The construction history records show that most of runway 15-33 was constructed in 1954 with a portion of the runway extended in 1957. The pavement structure consisted of 152 mm of crushed gravel and 76 mm of hot mix asphalt concrete for both the 1954 and 1957 construction.

The properties of the asphalt cement used in the original paving at Yellowknife Airport are unknown, however supporting details of the 1957 construction of the runway extension list 2201 barrels of 120-150 penetration asphalt remaining from 1954 construction. This amount, together with 952 barrels of 150-200, was used for the 1957 construction. This would indicate that all of the original runway plus approximately 70% of the extension was constructed with asphalt cement of 120-150 penetration grade.

The airfield pavement data inventory for Yellowknife Airport indicates the soil classification to be SP, GP, SM, and SW. The textural description shows 1 to 1.2 m of gravel and sand over sand. Some pockets of silty sand

with clay traces, and some boulders were found. The water table is about 1.2 m in the northwest section to 2.4 m in the south end of the field in the fall of the year. Indications are that the silty areas are frost susceptible and severe heave has been reported on runway 09-27 near the intersection, with minor to moderate heave in other areas. This was not considered to be a permafrost site, however settlements observed on runway 09-27 suggest that isolated permafrost areas likely existed.

The Freezing Index for this area is about 3500°C-days. The minimum air temperatures since 1954 are -49°C with an average of -45.8°C. The average maximum temperature is 23.1°C. This gives a range of about 32°C maximum in the summer to a minimum of -50°C in the winter.

Pavement Condition. A pavement condition survey taken 81-07-08 indicated extensive cracking in runway 15-33. Transverse cracking was extreme in extent and of major severity. Longitudinal cracking was described as of major extent and severity. Frost heaving was reported as moderate with subgrade settlement major and moderate in several areas.

Alternative Treatments

It was recommended that test sections be incorporated in the improvements to runway 15-33. An area exhibiting large transverse cracks across the runway as well as numerous smaller sealed cracks of a blocky pattern type was selected in the vicinity of Station 4+000 as shown in Figure 4. A detailed crack map of the area was prepared and is presented in Figure 5.

In view of the experience reported by MacMaster and Blum (6) it was recommended that a geotextile such as Bidim C-28 or similar type non-woven polyester fabric be tried. Since Bidim C-28 could not be obtained in time for this project, a locally available geotextile of the same type was selected. This material is known by the trade-name Penroad and was labelled as a Mirafi geotextile. Roll dimensions were 65 m in length and 3.5, 7.0 and 9.15 m in width. Two fabric types P-50 and P-250, having weights of 200 g/m^2 and 340 g/m^2 respectively, were selected for use. Widths of 9.15 and 7.0 m were chosen, however it was later discovered that these widths were made by sewing successive 3.5 m strips together. Figure 6 shows the location of the geotextile fabric sections coordinated with

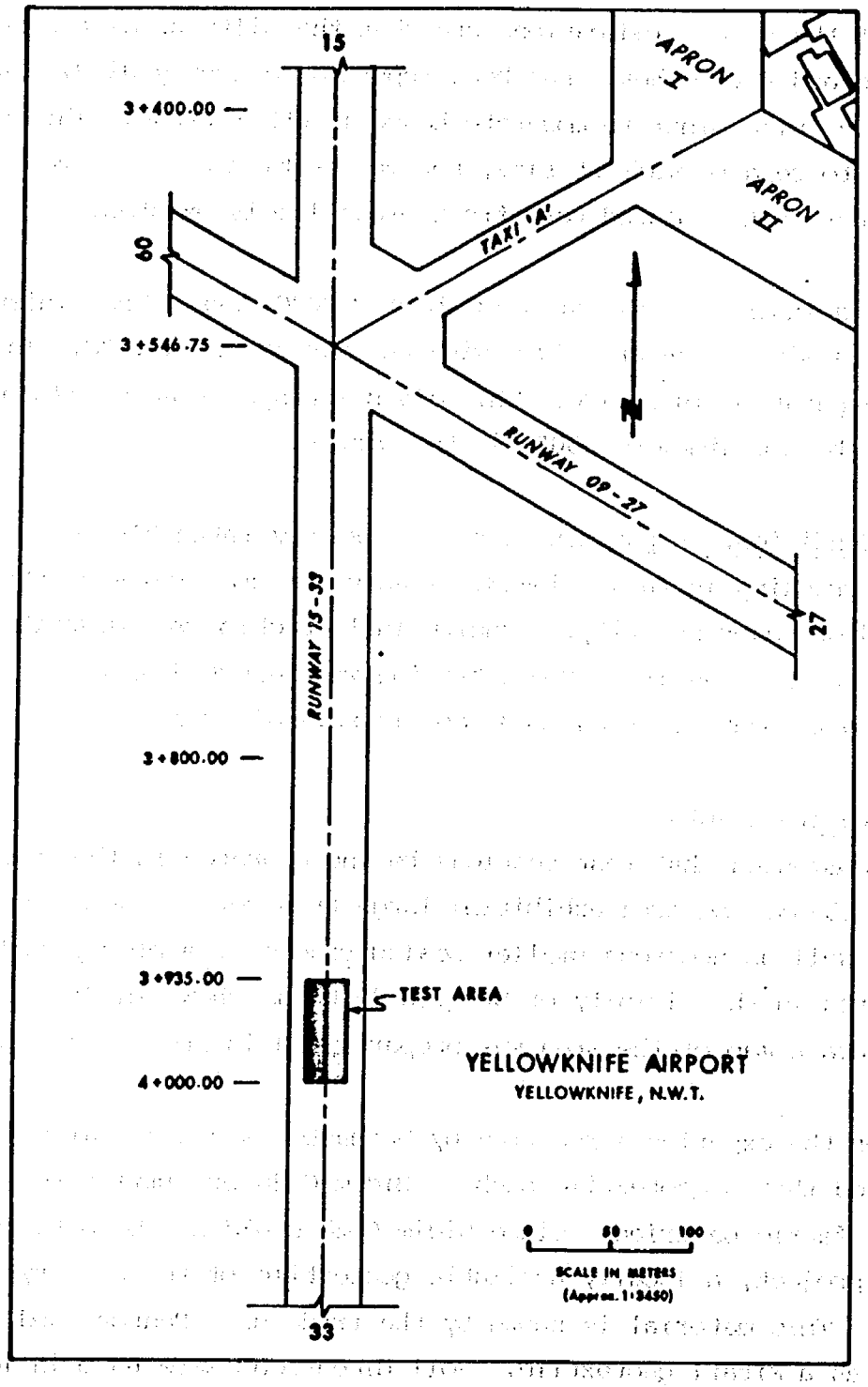


Fig. 4. Location of Test Area on Runway 15-33

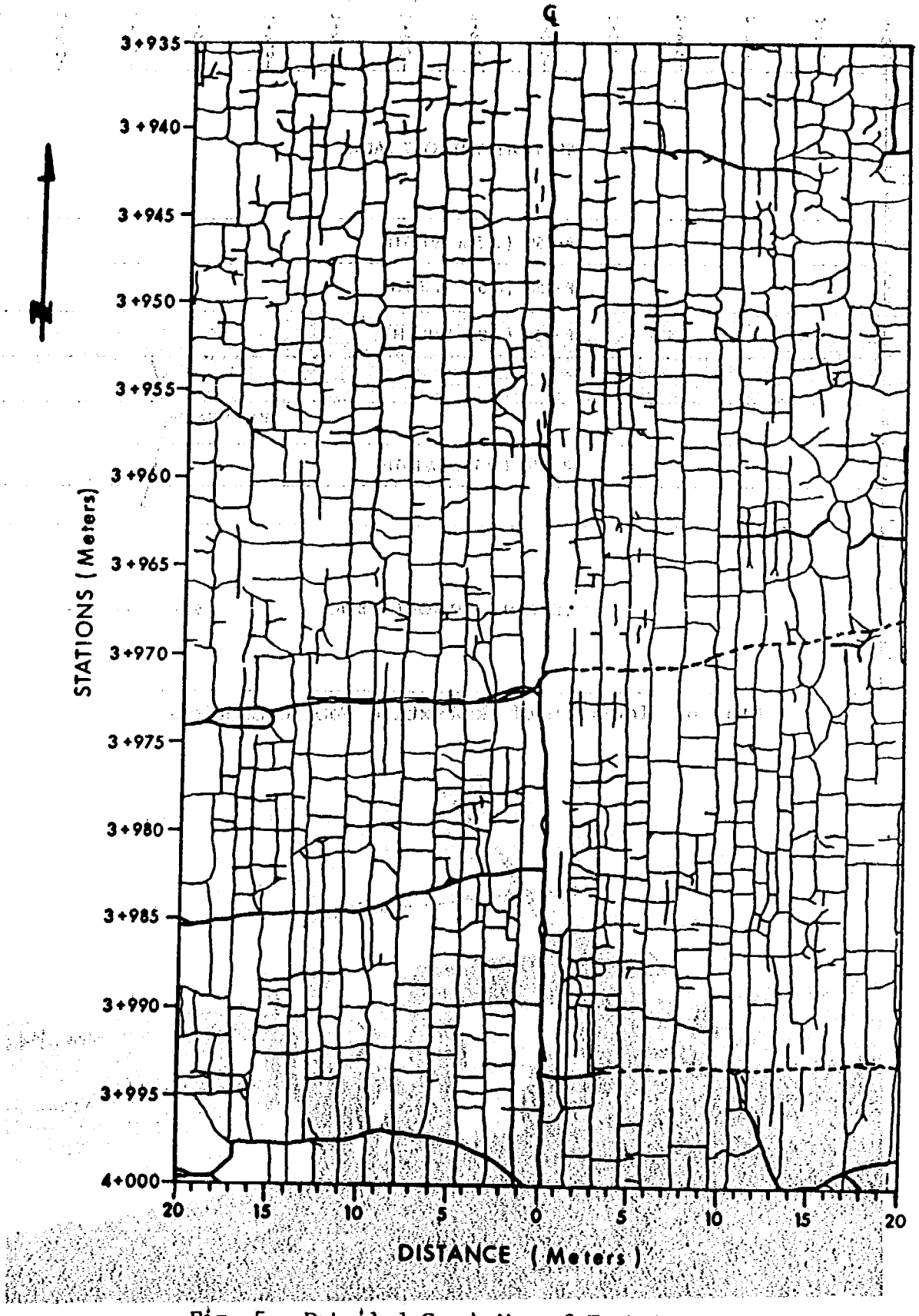


Fig. 5. Detailed Crack Map of Test Area

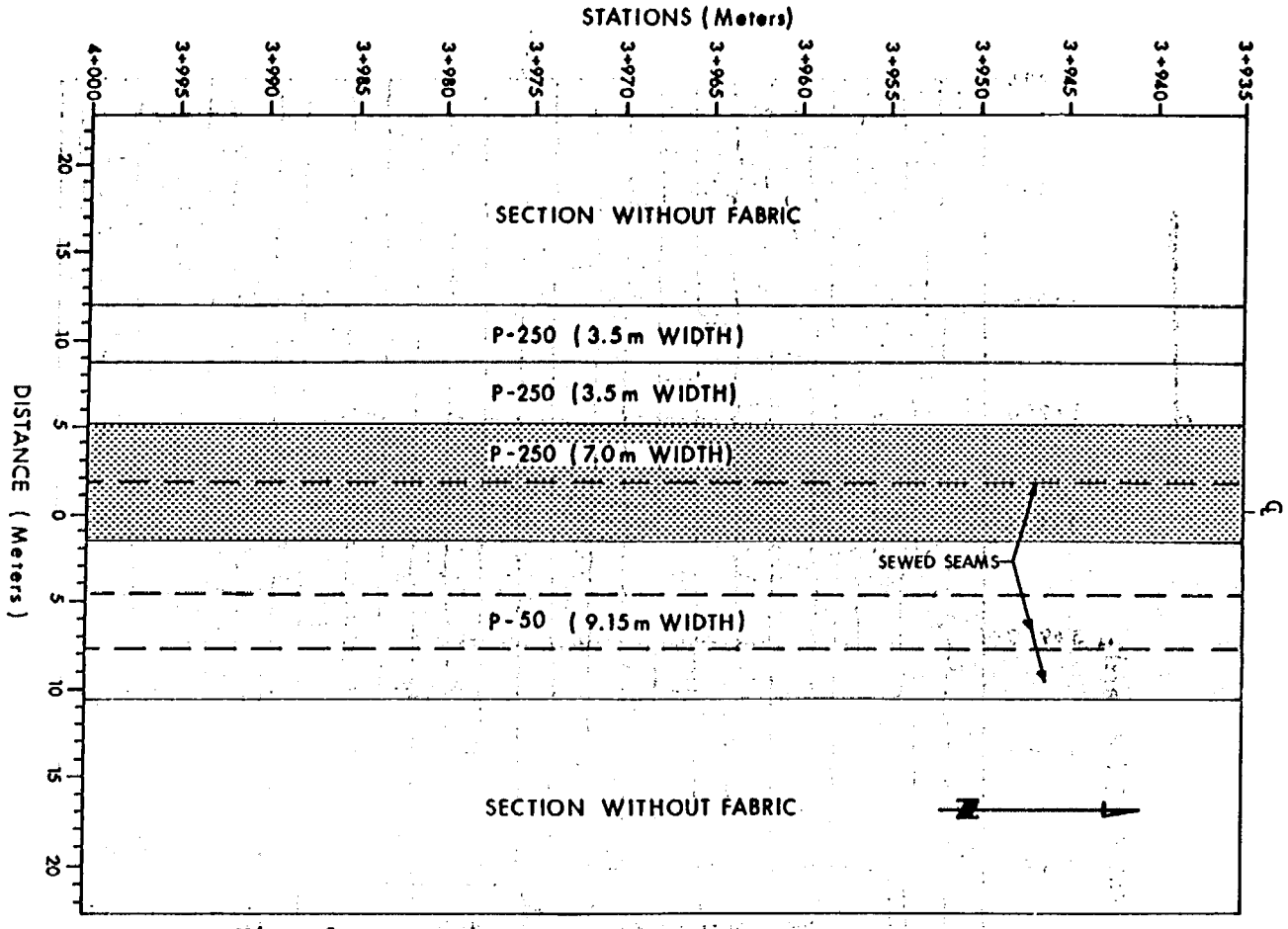


Fig. 6. Location of Geotextile Fabric Sections

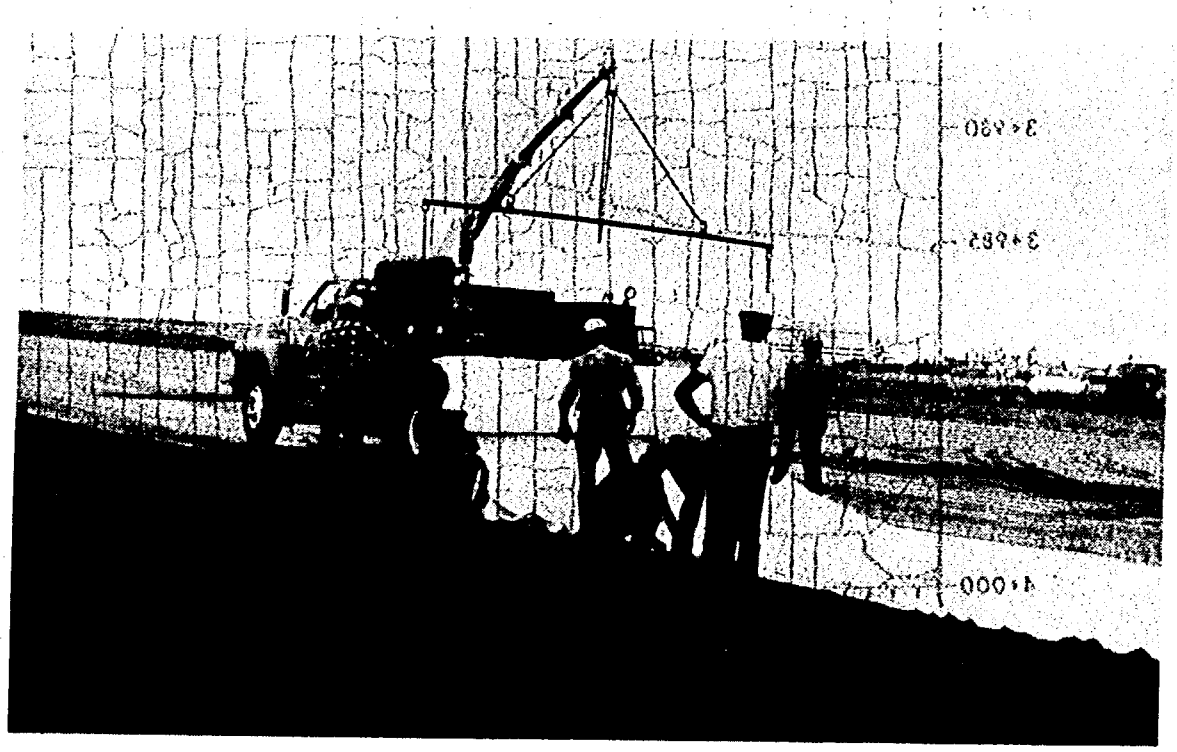


Fig. 7 Placing the Geotextile Fabric. 7 m width of P-250

scheduled passes and widths of the paver. Figure 7 shows the placing of the fabric, after completing the 3.5 m width of P-250 manually.

Selective treatment of some of the worst cracks was to be done prior to placing the levelling course. Approximately 800 m of severe cracking was mapped for treatment on runway 15-33. Two of these major cracks were located in the test area. The method of treatment was to heat the surface in the vicinity of the crack with a portable propane torch and to remove by hand the softened mixture of asphalt concrete and joint sealant. Figure 8 shows this operation.

Monitoring of Test Sections

Procedures described by Dykes (9) were used as a guide to formulate recommended procedures. He reported a typical application of asphalt cement of approximately 0.25 gal per sq. yd., residual asphalt. The rate applied must be sufficient to seal the fabric and provide bond to adjoining pavement layers, yet not "flush" to the newly paved surface.

Converting this rate to the metric equivalent gave 1.13 litres/m^2, and assuming a 60 percent residue for SS-1 emulsion, an application rate of 1.8 to 1.9 ℓ/m^2 was indicated. Since the normal rate of tack coat application was 0.5 ℓ/m^2, it was suggested two passes of the distributor be made at the normal rate. The fabric was then to be placed and rolled with a rubber tired roller. After this the need for additional tack coat could be determined visually and if necessary could be applied either by distributor or hand sprayer. In this way it was hoped that the correct application rate necessary to properly bond the fabric to the old pavement could be determined by observations after incremental applications.

Construction Sequence. The first layer of tack coat was applied with a distributor about 3 PM on 83-06-15. Due to rain delays the second layer of tack coat was applied at approximately 10:30 AM on June 17th. Approximately one hour after this application of tack coat, the first fabric section of P-250 of 3.5 m width was laid down manually. Numerous wrinkles developed in this first section. The second section of 3.5 m width P-250 was also placed manually and generally speaking was smooth.

Fig. 8. Portable Propane Torch

When the fabric was rolled it stuck to the pneumatic tires and caused wrinkles to develop. The time of initial rolling was approximately two hours after spraying the second tack coat. Because of this the rolling was stopped until the tack coat had cured sufficiently to prevent pick up by the roller tires. The 7 m and 9.15 m widths of P-50 fabric were placed in the afternoon using a truck mounted crane for handling the heavier rolls.

To remove the wrinkles in the 3.5 m P-50 section they were slit with a knife and edges overlapped. Where 15 cm overlap could not be achieved another piece of fabric was patched onto it. In areas where numerous wrinkles occurred, the fabric was cut across, lifted up and relaid. Again a 15 cm overlap was required. These overlaps were tacked in order to provide a bond between layers of fabric. Difficulties were experienced when the cut fabric was of irregular shape and sometimes narrower than required. Since there was not enough residual asphalt to seep through the fabric, another application of tack coat was placed on top of the fabric at about 10:30 PM. This freshly sprayed tack coat made the fabric very sticky causing the truck and paver tires to pick up the fabric. This resulted in many wrinkles and the 15 cm overlapping was lost in many areas.

The placement of the fabric showed that the folds at seams commonly were

placed in such a way that there could be up to three layers of fabric. To reduce this to a maximum of two layers, the seams in the uncovered portion of the 9.15 m wide section of P-50 fabric were folded back and tacked. Additional cutting and tacking of wrinkles and edges was completed before paving the remaining section of P-50 fabric. To prevent fabric pick up by the truck and paver tires a thin lift of asphalt mix was spread by hand in the wheel paths. This effectively prevented displacement of the fabric.

Observations on Test Sections. It can be expected that the numerous construction problems will affect the results of the experimental sections. The main problem seemed to be related to having the geotextile fabric properly tacked to the existing surface. The recommendation to use two light tack applications obviously was an error in judgment. This was compounded with the long delay between the first and second application and the short time for curing prior to placing the fabric.

Difficulties in placing the fabric were also not properly anticipated and as a result numerous wrinkles developed. The time constraint of having to have the runway open for operation the next day prevented proper construction procedures and corrective measures being taken when problems developed. The high elongation at rupture of the fabric, while being advantageous in some applications, was a detriment in this case. When the fabric stuck to the tires of the trucks and paver it tended to pull up and elongate. This caused overlapping folds to occur which resulted in sponginess and cracks to appear in the asphalt surface.

The comments of Sherman (13) have added meaning in the light of the experiences on this project.

> Construction practices can also affect the results of field experiments. When a new product or process is introduced, it takes a certain period of time for contractors and engineers to gain sufficient knowledge of placement to ensure that a uniform pavement can be produced and the performance can be compared to that of pavements placed using other products or processes. For example, various tack coats have been tried in the placement of fabrics: emulsified asphalt was used initially, but later it was found that a

penetration-grade asphalt provided the best method of sticking the fabric in place.

In hind-sight it would have probably been better to have used one heavy application of asphalt cement, near the recommended rate of 1.1 ℓ/m^2, and to take extra precautions so as not to place the fabric in such a way as to cause wrinkles. Use of fabric with sewn seams would also have been avoided, particularly the triple layer situations.

Observations in 1984

Discontinuous transverse reflection cracks spaced roughly 5 m apart have occurred in the test section. Similar longitudinal cracks are present on a somewhat closer spacing. The resultant pattern is not obviously different from adjacent pavement areas containing no fabric.

Some of the longitudinal cracks had developed a braided pattern which resulted in occasional break-outs. There was a lower incidence of pavement break-outs along cracks in the test section, but this phenomenon was not sufficiently common over the entire runway area to permit a confident comparison.

COST ANALYSIS

Construction costs for the 1982 overlay at Fort Smith and the 1983 Yellowknife project have been summarized from contract records.

Fort Smith

The average cost to heater scarify the existing surface, tack and place the 65 mm of asphalt concrete overlay came to \$8.62/m^2. The scarifying item alone came to \$1.28/m^2. The estimated cost of cold planing the surface to acceptable grade varied from \$1.28 to \$1.86/m^2 depending on the depth.

From these estimates it can be seen that the option of heater scarifying was the least costly to prepare the runway for an overlay.

Yellowknife

The average cost for the 75 mm overlay on runway 15-33 came to \$8.30/m^2. This included the major crack repair and ramping necessary. The crack

repair was about $0.30/m^2$ when averaged over the entire runway area. The unit cost was $56.80 per linear m of crack for a total of 560.2 m.

The estimated cost of the heater scarification alternative came to $9.13/m^2$, based on the contractor's tender prices. The scarifying item alone came to $2.17/m^2$.

The cost of the geotextile fabric only was $1.30/m^2$ for the P-50 and $2.30/m^2$ for the P-250. The total cost to supply and place the approximately 1400 m^2 of fabric came to $6.63/m^2$. Including the cost to repair the installation, the total came to $9.46/m^2$.

Cost Comparisons

As could be anticipated, the least costly alternative is the conventional 75 mm overlay with selective crack repair. The cost of selective crack repairing was far less than the heater scarification method to control the problem with the joint sealant material. As the necessary crack repair increases, the heater scarification method becomes more competitive with the selective crack treatment. There may be some added benefit as far as levelling is concerned and the treatment of all the cracks by scarifying.

The cost of cold-milling appears to be marginally more expensive when compared with heater scarification. In cases where there is sufficient structural strength, cold-milling to provide levelling as well as removing the excessive joint sealant material could be a viable alternative. This could provide correction to a greater depth than heater scarification and if combined with recycling methods may be of equivalent cost.

Even if the geotextile fabric P-50 or P-250 were to give some benefits in reflection crack minimization, the cost of application to an entire runway area would not seem to be justifiable. Because of the prevention of crack sealant from coming through the first overlay course, there may be some merit in considering using the 3.5 m width of P-50 fabric instead of the propane torch method of selective crack repair. This would of course require improved construction placement procedures, however a cost of approximately $20 per lineal m could be anticipated, using the prices obtained on this project.

CONCLUSIONS

Conclusions based on experience with recent overlays at Calgary, Hay River and Fort Smith airports are:

- Cracks have reflected through asphalt concrete which was placed following partial removal of an old surface by cold-milling methods to a depth of 64 mm.

- Heater scarification was apparently successful in sealing cracks of the existing surfaces and mixing crack sealant with the top 19 mm of the existing asphalt. This process did not have much effect in levelling the surface, however.

- There are no obvious differences in the reflection cracks observed on non-scarified as compared to scarified sections of the Fort Smith runway.

- Partial cold-milling and levelling courses can be used together to help restore desired grades to meet required smoothness criteria.

Conclusions drawn from the 1983 Yellowknife overlay project are:

- Selective crack repair using a portable propane torch and techniques developed on this project was successful and an economically viable method of pretreatment prior to overlays.

- The problems experienced in placing the non-woven polyester geotextile fabric will affect the results of the experimental sections. Experience gained may be useful in developing improved placement procedures.

- The geotextile fabric successfully prevented joint sealant from coming up through the asphalt overlay and therefore may be useful in selective major crack treatment.

- In the particular conditions under which it was used, it does not appear that this geotextile will significantly reduce the propagation of reflective cracks.

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Development of a New Design Method for Rehabilitation of Airport Pavements

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ABSTRACT

When airport pavements are damaged by traffic and environmental effects and their serviceability or load carrying capacity is insufficient, we consider some rehabilitation works for them. In Japan these rehabilitation works were conducted based on the subjective judgment of airport administrators up to now. But recently we have developed one new method to make decision for airport pavement rehabilitation strategy based on the quantitative evaluation of the surface conditions and the load carrying capacity.

Firstly, the Pavement Rehabilitation Index (PRI) equation has been developed for correlating surface condition with pavement engineers' opinions and each criteria of PRI for the necessity of rehabilitation to runway, taxiway and apron pavements has been decided.

Next, nondestructive evaluation method for load carrying capacity of airport pavements by use of Dynaflect has been proposed in stead of conventional destructive evaluation method.

In the new rehabilitation system the decision of necessity of rehabilitation is made based on the above-mentioned objective evaluations.

In this report some new overlay thickness design methods are also proposed. These are a thickness design method for asphalt overlay on asphalt pavements using Dynaflect deflections and a thickness design method for asphalt overlay on concrete pavements using the three-layer elastic theory.

DEVELOPMENT OF A NEW DESIGN METHOD FOR REHABILITATION OF AIRPORT PAVEMENTS

by

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1. Introduction

In airport pavements maintenance and rehabilitation keep the function of the pavements satisfactory and extend their service life once after the pavements are opened to traffic. Therefore, maintenance and rehabilitation strategies are quite important. However, the maintenance and rehabilitation of airport pavements in Japan have been carried out only on the basis of the empirical and subjective judgment of airport administrators and have not been necessarily systematic. From its kind of background, the development of a systematic maintenance and rehabilitation method has been eagerly desired to achieve the best value possible for the available funds.

The authors have been making researches on surface condition evaluation of airport pavements, structural evaluation, prediction of distress and performance, and maintenance and rehabilitation strategies.

Through these researches we have developed some new methods for the evaluation of airport pavements and the design of rehabilitation measures. The Pavement Rehabilitation Index (PRI) has been introduced for pavement surface condition evaluation and the criteria of PRI for the judgment of the necessity of rehabilitation to runway, taxiway and apron pavements have been decided. Nondestructive evaluation method for load carrying capacity of airport asphalt pavements by use of Dynaflect has been proposed in stead of conventional destructive evaluation procedure. Moreover, some new overlay thickness design methods have been proposed. A proposed thickness design method of asphalt overlays on asphalt pavements is based on the nondestructive structural evaluation using Dynaflect deflections.

From these kinds of new information we have proposed a new design system of rehabilitation of airport pavements. Main items in this design system are introduced in this report.

2. Fundamental Concept of Airport Pavement Rehabilitation

Airport pavements are maintained or rehabilitated when the necessary function cannot be fulfilled by the distress of the pavements due to aircraft traffic and environmental effects. Figure 1 shows the flow chart of the maintenance and rehabilitation works in airport pavements. The rehabilitation work which is dealt in this report is inside the dotted line in the figure.

The fundamental procedure of the proposed rehabilitation system consists of ① judgment of necessity of rehabilitation based on the evaluation of pavement surface condition, ②structural evaluation for the pavements in which rehabilitation is judged to be necessary, ③selection of appropriate rehabilitation measure, and ④design of the selected rehabilitation measure.

3. Asphalt Pavement

3.1 Pavement Evaluation

(1) Surface Condition Evaluation[1)]

It is very important to quantify pavement performance, because it helps engineers to recognize the degree of pavement distress and the necessity of rehabilitation. An equation for pavement performance evaluation has been developed for correlating pavement engineers' opinions with objective values of pavement surface conditions.

Forty two inventory sections (one section is $20 \times 30 m^2$), which have several types of distress, have been selected, and then the questionary to twenty four pavement engineers has been performed at each section, concerning the items shown in Table 1, the degree of distress and the necessity of rehabilitation (regarding as runway, taxiway and apron). On the other hand, mechanistic measurements also have been performed at each same section. The results of the questionary have been quantified by the Quantification Theory II.

The summary of the Quantification Theory II is the following. When one evaluation item (ex. rutting) is rated at A-category (or B- or C-category), the numerical value x_{11} is given (x_{12}, x_{13} respectively). Then the total score for each inventory section is,

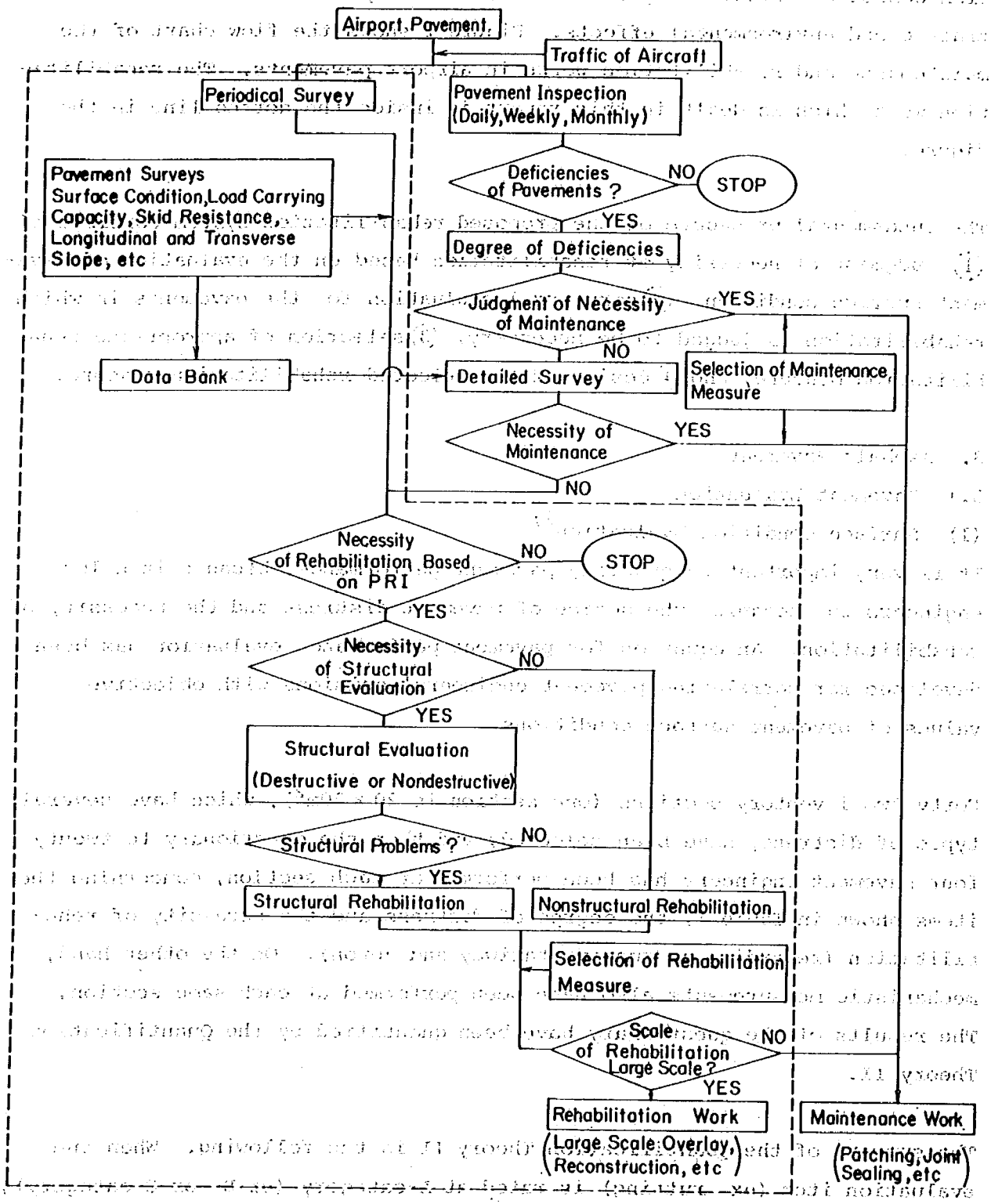

Figure 1 Flow Chart of Rehabilitation of Airport Pavements

$$\alpha_i = \sum_{j=1}^{R} \sum_{k=1}^{k_j} \delta_i(jk)\ x_{jk} \qquad (1)$$

where x_{jk} : numerical value given to the k-th category of the j-th item

$\delta_i(jk)$: = 1, if i-sample checks in the k-th category in the j-th item

= 0, otherwise

k_j : number of the categories in the j-th item

R : total number of items

Figure 2 shows the assumptive distribution patterns of α regarding each group. If the variance of each group (σ_W^2) can be minimized, and the variance between the groups (σ_B^2) can be maximized at the same time, the visual evaluation item (A, B and C) can be predicted and classified precisely by use of α obtained from Equation (1). Contrarily, since the correlation ratio η^2 will be expressed as $\sigma_B^2/(\sigma_W^2+\sigma_B^2)$, the procedure described above is equivalent to quantifing the category so as to maximize η^2.

Table 1 shows the score of each category in each item, which is obtained by the application of the Quantification Theory II to the results of the questionary. As the form of Equation (1) shows, the item which indicates greater differencies of the score given to A, B and C, has greater contribution to the total score of the inventory section. Table 1 also shows the difference of the score given to A and C as a range, and Table 2 shows the three items ranked in the range.

Next, the equation of performance evaluation has been obtained by use of the measured values regarding the items which have greater contribution to the total score. Considering the easiness of the measurements, the following three items are finally selected for it.

① Cracking ratio, CR(%) : cracking area/area of a section

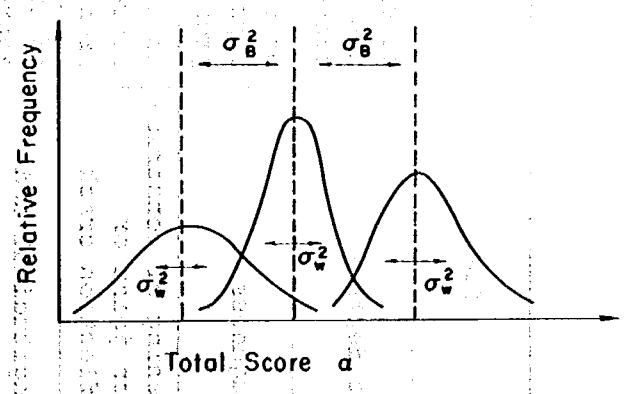

Figure 2 Distribution of Total Scores

Table 1 Scores Given to Each Category

Item	Category		Rating		Necessity of rehabilitation					
					Runway		Taxiway		Apron	
			Score	Range	Score	Range	Score	Range	Score	Range
(1) Rut depth	no or minor influence	A	-0.209	0.539	-0.279	0.732	-0.243	0.653	-0.200	0.604
	appreciable influence	B	0.058		0.700		0.051		0.003	
	pronounced influence	C	0.330		0.453		0.410		0.404	
(2) Roughness	no or minor influence	A	-0.431	1.147	-0.494	1.202	-0.397	1.093	-0.416	1.044
	appreciable influence	B	0.152		0.244		0.116		0.186	
	pronounced influence	C	0.716		0.708		0.696		0.628	
(3) Opening of construction joint or reflection crack	no or minor influence	A	-0.280	0.697	-0.165	0.499	-0.174	0.525	-0.180	0.507
	appreciable influence	B	0.012		0.040		-0.041		-0.023	
	pronounced influence	C	0.417		0.334		0.351		0.327	
(4) Longitudinal and transverse crack	no or minor influence	A	-0.189	0.414	-0.193	0.374	-0.150	0.262	-0.137	0.609
	appreciable influence	B	0.130		0.145		0.027		0.015	
	pronounced influence	C	0.225		0.181		0.476		0.472	
(5) Alligator crack	no or minor influence	A	-0.024	0.230	-0.079	0.400	-0.082	0.358	-0.145	0.558
	appreciable influence	B	0.000		0.076		0.091		0.177	
	pronounced influence	C	0.206		0.321		0.276		0.413	
(6) Scaling or pot hole	no or minor influence	A	-0.100	0.687	-0.022	0.151	-0.068	0.633	-0.050	0.512
	appreciable influence	B	0.158		0.033		0.044		0.012	
	pronounced influence	C	0.587		0.129		0.565		0.462	
(7) Surface texture	no or minor influence	A								
	appreciable influence	B								
	pronounced influence	C								
(8) Bleeding	no or minor influence	A	-0.010	0.214	-0.022	0.156	-0.021	0.360	-0.032	0.434
	appreciable influence	B	0.204		0.040		0.000		0.022	
	pronounced influence	C			0.134		0.339		0.402	

× 100

② Rut depth, RD (mm)

③ Roughness, SV (mm): mean slope variance obtained by 3-m profilometer

Table 2 Items Having a Greater Range

Total evaluation		Order	Item
Degree of distress		1	roughness
		2	opening of construction joint or reflection crack
		3	scaling or pot hole
Necessity of rehabilitation	Runway	1	roughness
		2	rut depth
		3	opening of construction joint or reflection crack
	Taxiway	1	roughness
		2	rut depth
		3	scaling or pot hole
	Apron	1	roughness
		2	longitudinal and transverse crack
		3	rut depth

The equation has been obtained by the following procedure. The total score of each observer has been calculated by substituting the score of each category in each item shown Table 1 into Equation (1), and the mean value of the total scores has been calculated as a dependent variable for each section. Subsequently, multiple regression analysis on the assumption of linear form has been carried out between the mean values and the measured values mentioned above as independent variable. If the independent variables are converted to exponential form, the coefficient of correlation becomes a little greater. However, since the difference is very small, the following linear form has been applied here.

$$y = a_0 + a_1 x_1 + a_2 x_2 + a_3 x_3 \qquad (2)$$

where y : total score

$x_1 \sim x_3$: measured values

$a_0 \sim a_3$: coefficients

The coefficients have been determined from a set of input data, and the resulting equation is,

$$y = a_0 + 0.0776CR + 0.00881RD + 0.113SV \qquad (3)$$

where $a_0 = -0.845$ for runway

-0.836 for taxiway

-0.847 for apron

The equation converted by linear transformation, which expresses a score 0 as the minimum value and a score 10 as the maximum value, is

$$y = 10 - 0.450CR - 0.511RD - 0.655SV \quad (4)$$

The value obtained from this equation has been named PRI (Pavement Rehabilitation Index). Table 3 shows the correlation between the criteria of the necessity of rehabilitation and PRI. Therefore, when it is necessary to evaluate airport pavements by the surface condition, PRI will be calculated by use of Equation (4) with measured values of cracking ratio, rut depth and roughness, and then the necessity of the pavement rehabilitation will be judged on the basis of Table 3.

However, it is difficult to evaluate the distress caused by one particular item by use of Equation (4) and Table 3. In this case, Table 4, which shows the criteria concerning each item, could be used.

Table 3 Criteria of PRI

Section	A, B	B, C
Runway	8.0	3.8
Taxiway	6.9	3.0
Apron	5.9	0

A: Rehabilitation is not required

B: Rehabilitation is desirable in the near future

C: Rehabilitation is required immediately

Table 4 Criteria for Each Category

Item	Section	Criteria	
		A, B	B, C
Cracking ratio CR (%)	Runway	0.1	6.5
	Taxiway	0.9	12.7
	Apron	1.9	17.0
Rut depth RD (mm)	Runway	10	38
	Taxiway	17	57
	Apron	22	70
Roughness SV (mm)	Runway	0.26	3.64
	Taxiway	0.91	6.57
	Apron	1.50	8.63

(A, B, C are the same meaning in Table 3)

(2) Structural Evaluation[2)]

Only nonstructural rehabilitation to improve pavement surface condition is enough for the pavements in which some rehabilitation is necessary from the pavement surface condition evaluation but which have not any structural problems. Contrarily, in case that pavements have some structural problems, structural rehabilitation to improve pavement load carrying capacity must be carried out. Consequently, some structural evaluation is necessary to the pavements which may have some structural problems.

Until recently, the structural evaluation of airport pavements was conducted based on destructive test procedures. However, since these destructive tests are time-consuming and costly, they are not suitable for airport pavement having many constraints such as limited time for investigation or construction.

In this decade, researchers have developed nondestructive test procedures. Using a nondestructive procedure has the advantage of rapid evaluation with a minimum of disturbance to normal traffic operation. Dynaflect, a nondestructive test device developed and widely used in the United States for measuring pavement surafce deflection, has been chosen for our research and the new method for structural evaluation of airport asphalt pavement using Dynaflect has been established.

Deflections of asphalt pavement are affected by both frequency of dynamic load and temperature. For Dynaflect measurement, the frequency is fixed at 8 Hz and further frequency adjustment is unnecessary. The influence of temperature, on the other hand, must be corrected to compare data taken at various temperatures.

In order to adjust measured deflections to their values at the standard surface temperature, 20°C, temperature adjustment factors shown in Figure 3 have been derived from experiment. These factors are applicable to only Dynaflect Maximum Deflection (DMD), which is the deflection of No.1 geophone. To show their accuracy, some examples of the application of these factors are given in Figure 4. The results show that more than 95% of the estimated values are between 100 ± 15% of the measured value.

Pavement thickness is designed as a function of the type of aircraft and traffic volume for given subgrade conditions. This means that surface deflections of the pavement are also influenced by the design load and the design coverage of the pavement. DMDs measured on some airport asphalt pavements are plotted in Figure 5 with their converted values (at 20°C) as ordinates against Equivalent Design Gear Load (EDGL) of the pavement on which DMDs have been measured as abscissas.

In this paper, a EDGL is defined as the allowable gear load for 5,000

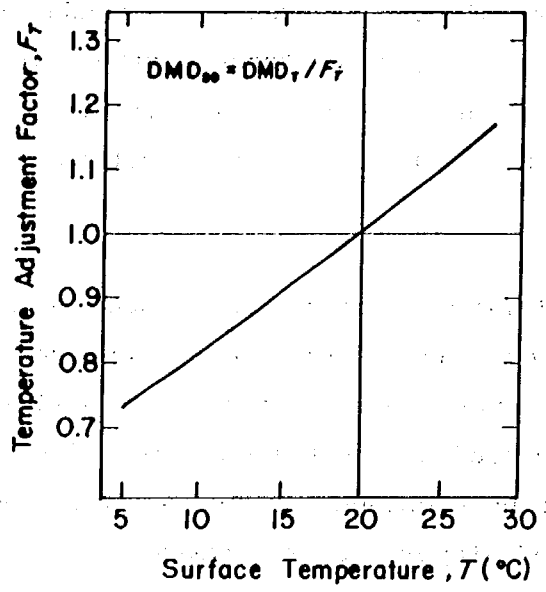

Figure 3 Temperature Adjustment Factors

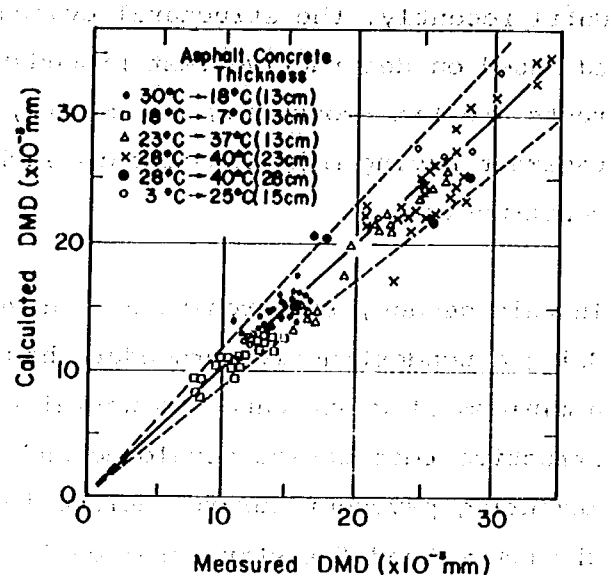

Figure 4 Relationship between Measured and Calculated DMDs

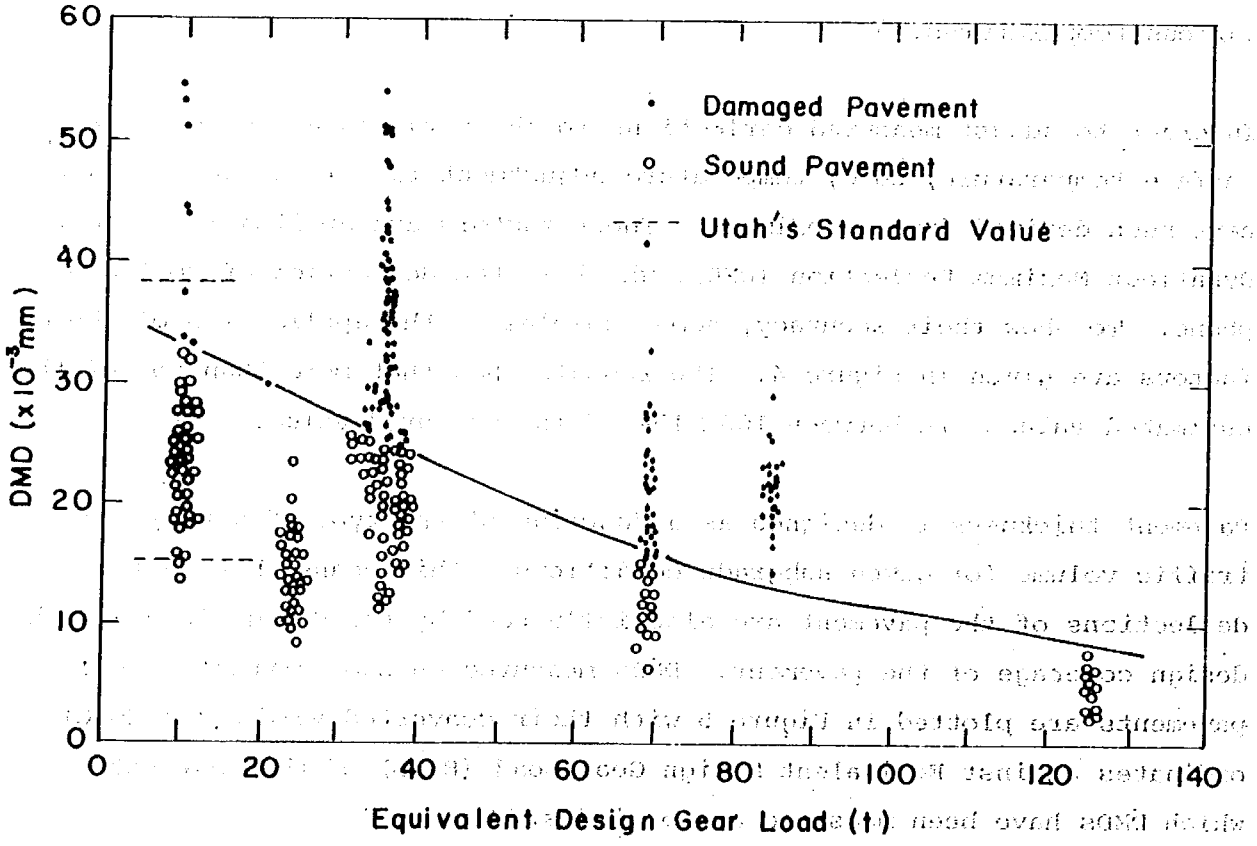

Figure 5 Relationship between DMD and Equivalent Design Gear Load

coverage which the pavement under consideration can sustain for its design CBR and the pavement thickness. For example, according to the CBR design method, 170 cm is required as a pavement thickness for 20,000 coverage of a Boeing-747-200B (Gear Load: 82.5 t) for 5% of design CBR. This pavement thickness is equal to the required thickness for 5,000 coverage of 115 t gear load. This gear load, 115 t, is the EDGL of this pavement.

From Figure 5, it can be seen that DMDs measured on structurally damaged pavements are generally higher than those on sound pavements, though these data are scattered because of the difference of the degree of distress. The solid line in the figure is drawn to separate these data into two parts. A pavement with DMD beyond this line might be regarded as being structurally damaged and requiring some rehabilitation. Standard values presented by the Utah State Department of Highways are also shown in the figure for comparison. A DMD indicated by this line is defined as a standard DMD for the corresponding EDGL and can be used as a critical deflection to estimate the structural adequacy of asphalt pavement.

From these results, nondestructive structural evaluation for airport asphalt pavements has become possible.

3.2 Selection of Rehabilitation Measure

As described in section 3.1, there are two types of distress in airport asphalt pavements, that is, structural and nonstructural. Consequently, there are two types of rehabilitations. One is a structural rehabilitation to improve pavement load carrying capacity and another a nonstructural rehabilitation to improve only pavement surface condition.

Rehabilitation measures for improving only pavement surface condition are in general an overlay or a partial replacement of the upper part of an existing pavement. Some definite cases are as follows.

① When pavement smoothness is decreased due to longitudinal deformation and/or depressions, the rehabilitation is normally an overlay to improve the roughness. In cases that there are problems when an overlay pavement is fitted to the surrounding pavements or structures, a removal of the upper part of the existing pavement and a replacement with new materials should be considered in place of an overlay.

② For rutting due to only plastic flow of a surface and/or binder courses, a partial replacement of the rutted part or an overlay is applied.

③ In case that a surface course has cracks due to aging but there are no structural problems, a removal of the surface course and a replacement of the part is generally applied.

④ On the occasion of the opening of construction joints in a surface course without any structural problems, a removal and a replacement of the part is applied.

⑤ When the skid resistance of a runway is not enough for safety, a grooving or a removal of rubber deposits should be firstly considered and at the next step an overlay or a replacement of the surface course is considered.

Rehabilitation measures to improve load carrying capacity are generally a replacement of the damaged part of an existing pavement, an overlay or a full reconstruction in airport asphalt pavements. Some definite cases are as follows.

① In cases that the only upper part of an existing pavement such as a surface and/or binder courses or a base course is damaged, a removal of the damaged part and a replacement of the part with new materials is conducted on the consideration of the degree of the distress and the apprehension of reflection cracks in an overlay.

② Overlays are mostly applied in cases that a pavement may have been damaged by loading or environmental effects or a pavement in good condition may require strengthening to serve heavier aircraft.

③ It should be considered whether the surface course of an existing pavement can be used for the binder course of an overlay pavement.

④ In some instances, an overlay is carried out after a partial replacement of an existing pavement.

⑤ When a partial replacement of an existing pavement or an overlay is not enough to maintain the stability or durability of a pavement, a full reconstruction of an existing pavement is carried out.

Since a long time can not be generally taken for rehabilitation in airport pavements with relation to the operation of the facilities, it should be considered to apply the rehabilitation measures that can achieve satis-

factory effects in as short time as possible. In some instances of rehabilitation works such as those in runway pavements, they must be carried out during the night and the facilities must be opened to traffic the next morning. Then, in such cases, the rehabilitation measures are constrained by the operational limitations and may not always be the best one.

3.3 Design of Rehabilitation Measures

The most usual rehabilitation measure in airport asphalt pavements is an overlay. The design method of asphalt overlays to improve load carrying capacity is mainly dealt in this section. The design of the other rehabilitation measures such as reconstruction can be done by the similar procedure in new pavement construction.

Until recently, the asphalt overlay thickness design was conducted based on the mechanical evaluation for subgrade and each layer of an existing pavement through destructive tests. However, since this procedure has such defects that were mentioned in section 3.1(2), the new overlay thickness design method based on the nondestructive evaluation of existing pavements has been researched and we have developed a quite simple and convenient procedure as follows.[2)]

Figure 6 is the flow chart of the overlay thickness design procedure proposed herein.

① Prepare input data, such as a set of Dynaflect measurements, the thickness of the asphaltic layer of the existing pavement (h_1), surface temperature (T), and allowable DMD (DMD_0). Selection of DMD_0 is very important. At this point, the standard DMD shown in Figure 5 may be used for this purpose.

② Adjust DMD to its value at 20°C using Figure 3.

③ Compare DMD at 20°C with DMD_0. If the former is greater than the latter, go to the next step.

④ Dividing the existing pavement into an upper layer (asphaltic material) and a lower layer (non-asphaltic material), calculate each elastic modulus using the two-layer elastic theory. In this step, the moduli of asphaltic material at T°C and non-asphaltic material of the existing pavement are obtained.

⑤ By Equation (5), calculate the modulus of deformation of asphalt con-

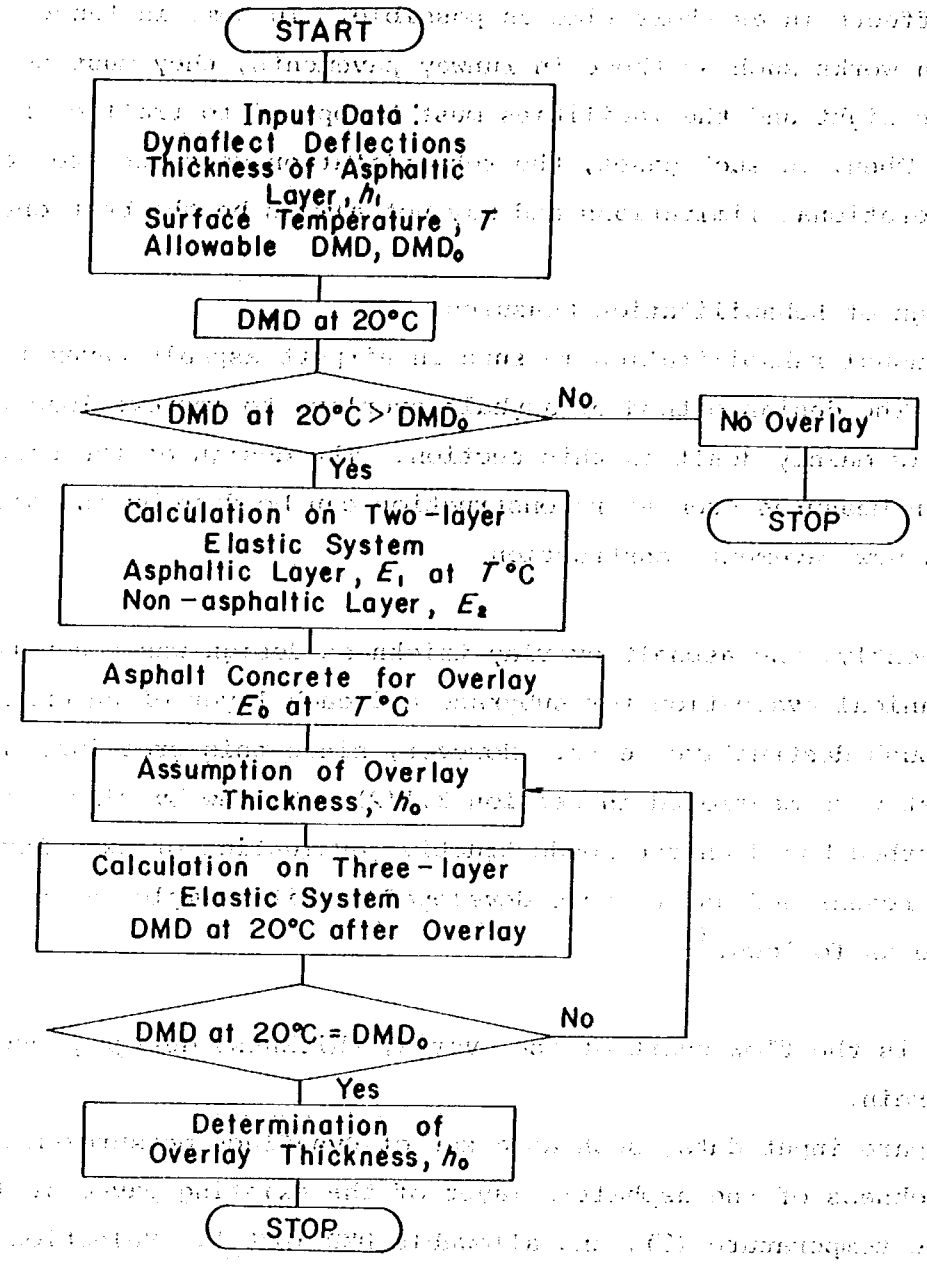

Figure 6 Flow Chart of Proposed Scheme for Overlay Thickness Design

crete for overlay at T°C.

$$\log E = 5.237 - 0.02688T \quad (5)$$

where, E: modulus of deformation of asphalt concrete (kgf/cm^2)
T: surface temperature (°C)

⑥ Assume overlay thickness, h_0.

⑦ Considering the overlaid pavement structure as a three-layer elastic system as shown in Figure 7, estimate DMD after overlay using the

three-layer elastic theory. The DMD calculated in this step corresponds to the DMD after overlay at T°C.

⑧ Adjust the DMD obtained in step ⑦ to its value at 20°C.

⑨ Repeat step ⑥ through ⑧ until calculated DMD into step ⑧ sufficiently approaches DMD_0.

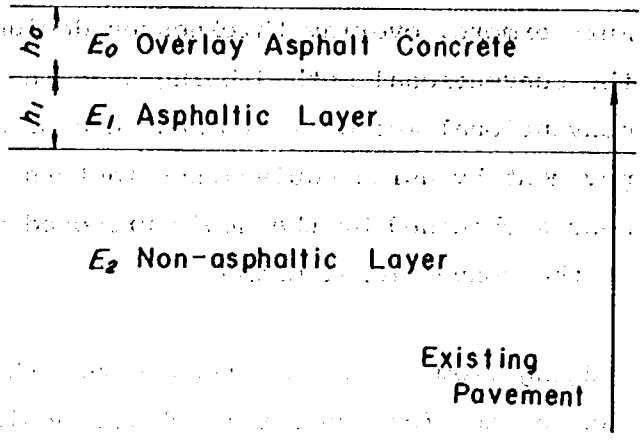

Figure 7 Pavement Structure after Overlay

One of the ways to confirm the accuracy of the overlay thickness design procedure is to compare the DMD predicted from deflections on the existing pavement with the DMD actually measured after overlay. Figure 8 shows a close agreement between predicted and measured DMDs. Overlay thicknesses employed herein are between 3.6 and 19 cm. From Figure 8, it can be seen that the three-layer elastic theory can predict the DMD after overlay with high accuracy.

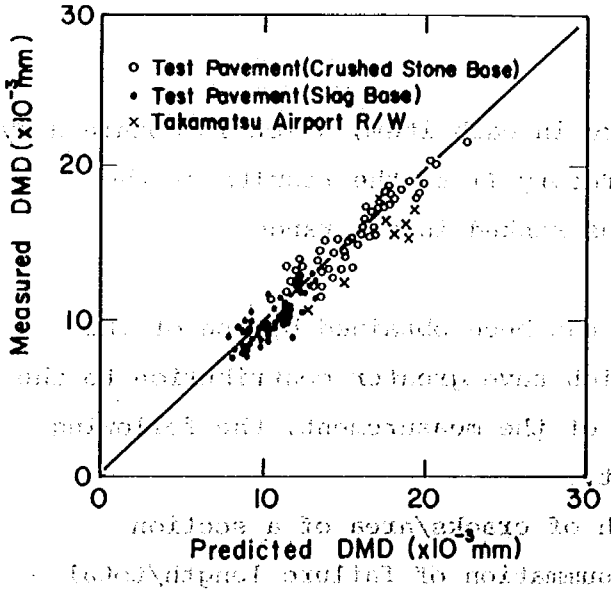

Figure 8 Relationship between Predicted and Measured DMDs

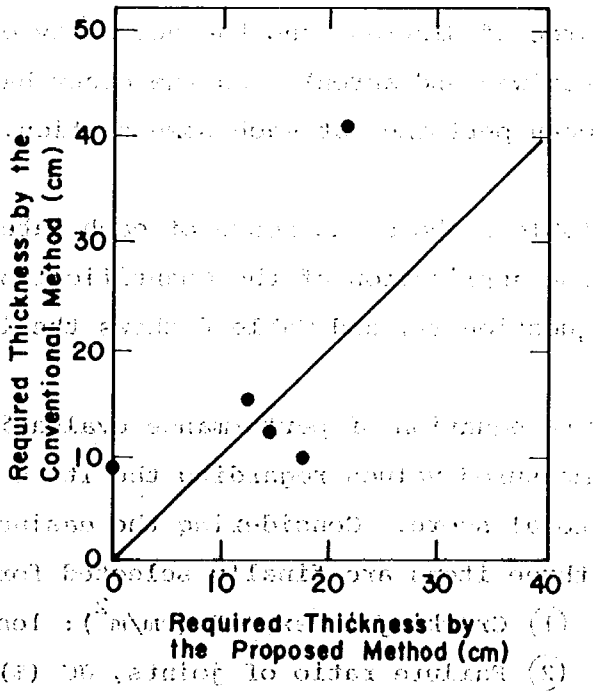

Figure 9 Comparison of Required Overlay Thickness

Furthermore, overlay thicknesses designed by this proposed method and by the conventional method being used in Japan are compared in Figure 9. The conventional method is to make use of an effective thickness of existing pavement by using equivalency factors of each component. Overlay thicknesses obtained by the newly proposed method agree approximately with those by the conventional method.

Judging from the facts shown above, the procedure porposed in this paper can be used for overlay thickness design of airport asphalt pavement.

4. Concrete Pavement

4.1 Pavement Evaluation

(1) Surface Condition Evaluation[1)]

The procedure used in evaluating surface conditions of concrete pavements is similar to that used in asphalt pavements.

In the case of concrete pavements, twenty five inventory sections (one section is 20×20 m^2), which have several types of distress, have been selected. And then the questionary to twenty four pavement engineers have been performed at each section, concerning the items shown in Table 5, the degree of distress and the necessity of rehabilitation (regarding as runway, taxiway and apron). On the other hand, mechanistic measurements also have been performed at each same section.

Table 5 shows the score of each category in each item, which is obtained by the application of the Quantification Theory II to the results of the questionary, and Table 6 shows the items ranked in the range.

The equation of performance evaluation has been obtained by use of the measured values regarding the items which have greater contribution to the total score. Considering the easiness of the measurement, the following three items are finally selected for it.

① Cracking index, CR (cm/m^2): length of cracks/area of a section
② Failure ratio of joints, JC (%): summation of failure length/total length of joints × 100
③ Difference in level, SV (mm)

Table 5 Scores Given to Each Category

Item	Category		Rating		Necessity of rehabilitation					
					Runway		Taxiway		Apron	
			Score	Range	Score	Range	Score	Range	Score	Range
(1) Longitudinal and transverse crack	no or minor influence	A	-0.373	0.769	-0.328	0.728	-0.292	0.956	-0.272	1.019
	appreciable influence	B	0.224		0.162		-0.063		-0.147	
	pronounced influence	C	0.396		0.400		0.664		0.747	
(2) Alligator crack	no or minor influence	A	-0.019	0.147						
	appreciable influence	B	0.015							
	pronounced influence	C	0.128							
(3) Diagonal crack	no or minor influence	A	0.056	0.132	-0.081	0.224	-0.072	0.391	-0.043	0.401
	appreciable influence	B	0.076		0.103		0.061		0.009	
	pronounced influence	C			0.143		0.319		0.358	
(4) Joint spall	no or minor influence	A	-1.053	1.654	-1.039	1.533	-0.814	1.597	-0.668	1.359
	appreciable influence	B	0.175		0.216		-0.007		-0.029	
	pronounced influence	C	0.601		0.494		0.783		0.691	
(5) Difference in level	no or minor influence	A	-0.047	0.138	-0.162	0.343	-0.066	0.335	-0.061	0.650
	appreciable influence	B	0.025		0.114		0.007		0.589	
	prcnounced influence	C	0.091		0.181		0.269			
(6) Scaling or pot hole	no or minor influence	A	-0.133	0.450	-0.168	0.440	-0.154	0.610	-0.139	0.612
	appreciable influence	B	0.049		0.119		0.018		-0.011	
	pronounced influence	C	0.317		0.272		0.456		0.473	
(7) Surface texture	no or minor influence	A								
	appreciable influence	B								
	pronounced influence	C								
(8) Patching	no or minor influence	A	-0.046	0.211	-0.025	0.115	-0.022	0.010	-0.046	0.211
	appreciable influence	B	0.165		0.090		0.079		0.165	
	pronounced influence	C								

The resulting equation is,

$$y = 10 - 0.290CR - 0.296JC - 0.535SV \quad (6)$$

The value obtained from this equation has been named PRI (Pavement Rehabilitation Index). Table 7 shows the correlation between the criteria to the necessity of rehabilitation and PRI.

Table 6 Items Having a Greater Range

Total evaluation		Order	Item
Degree of distress		1	joint spall
		2	longitudinal and transverse crack
		3	scaling or pot hole
Necessity of rehabilitation	Runway	1	joint spall
		2	longitudinal and transverse crack
		3	scaling or pot hole
	Taxiway	1	joint spall
		2	longitudinal and transverse crack
		3	scaling or pot hole
	Apron	1	joint spall
		2	longitudinal and transverse crack
		3	roughness

However, it is difficult to evaluate the distress caused by one particular item by use of Equation (6) and Table 7. In this case, Table 8, which shows the criteria concerning each item, could be used.

(2) Structural Evaluation

As well as asphalt pavements, the necessity of rehabilitation of concrete pavements should be decided according to the basis of structural evaluation. However, nondestructive evaluation methods for concrete pavements have not been established yet, so they must be evaluated by destructive one.

Its principle is as follows; using the modulus of reaction (K value) at the top of base course and the flexural strength of the concrete which are obtained by destructive test, the safety factor of concrete slab for design aircraft load is calculated. And then, based on the criteria shown in

Table 7 Criteria of PRI

Section	A, B	B, C
Runway	7.0	3.7
Taxiway	6.4	2.3
Apron	5.7	0

A: Rehabilitation is not required

B: Rehabilitation is desirable in the near future

C: Rehabilitation is required immediately

Table 8 Criteria for Each Category

Item	Section	Criteria A, B	Criteria B, C
Cracking index CR (%)	Runway	0.2	5.6
	Taxiway	0.6	7.6
	Apron	1.1	11.1
Faliure ratio of joints JC (%)	Runway	0.1	1.3
	Taxiway	0.1	3.2
	Apron	0.1	5.7
Difference in level SV (mm)	Runway	5	10
	Taxiway	5	12
	Apron	6	14

(A, B, C are the same meaning in Table 7)

Table 9, the load carrying capacity of the pavement can be evaluated and its necessity of rehabilitation can be determined.[3)]

Table 9 Evaluation of Existing Concrete Pavement

Safety factor	Evaluation of existing concrete pavement
greater than 2.0	It can sustain coverages between 10,000 and 20,000.
1.7 to 2.0	It can sustain coverages between 3,000 and 10,000. If an expected traffic volume is greater, some rehabilitation works should be carried out.
less than 1.7	It can sustain coverages of 3,000 or less, so some rehabilitation works must be carried out.

4.2 Selection of Rehabilitation Measure

There are two kinds of rehabilitation for concrete pavements; one is to improve the surface conditions, and another is to strengthen the structures. When aging of materials or traffic deteriorates only pavement surface without structural problem, only the surface characteristics should be improved. At that time, asphalt overlays are generally applied. However, in case that severe rutting would be expected in the overlay layer (e.g. apron area), concrete overlays should be considered as the rehabilitation measure. Besides, at a junction with other structures, some parts of existing slabs should be planed and then overlay might be carried out.

When pavements have insufficient load carrying capacity, the rehabilitation

which can strengthen the structures must be conducted. In consideration of the degree of the distress in the pavements, the expected aircraft and their converages, and some constraints related with aircraft operation and the geometry of the pavements, the best measure should be selected from the followings.

① asphalt overlay
② concrete overlay
③ reconstruction

In the order of the severity of distress, the foregoing item should be adopted to the rehabilitation measure in order.

An asphalt overlay is suitable for the situation where the facilities can not be closed. In this case, after the rehabilitation work is conducted during the night, the facilities can be opened the next morning, because an asphalt overlay needs little curing time.

In concrete overlays there are three types; bonded, partially bonded and unbonded. Although an unbonded overlay can be applied whether distress in existing slabs is severe or not, the other two methods can only be applied to the pavements whose distress is not so severe. Besides, at present the construction technique for a bonded overlay is not necessarily established yet, so its application might be considered carefully.

A reconstruction is generally considered as the rehabilitation measure in severely distressed slabs. However, since placing concrete in-situ requires a long curing time, it is not so widely applied. In order to overcome this difficulty, prestressed concrete (PC) precast slab pavement has been developed recently.[4] By using this new method the applicability of a reconstruction has become larger for the rehabilitation measure of airport concrete pavements.

4.3 Design of Rehabilitation Measures

The design of structural rehabilitation measures for existing concrete pavements is generally based on the structural evaluation. Since non-destructive test procedure has not been developed yet, the design of structural rehabilitation measures should be based on the destructive evaluation at present.

(1) Asphalt Overlay[5)]

Until now, a design method which has been applied to asphalt overlays on asphalt pavements has also been used for asphalt overlays on concrete pavements. Using the design CBR value of existing subgrade and the new design load, the required thickness of asphalt pavement of which base is composed of granular materials is determined. The existing concrete slab is assumed as the base course and the thickness is converted to the equivalent thickness of a granular base using equivalency factor (E.F.) shown in Table 10 (equivalent thickness = E.F. × concrete slab thickness). And then the other existing base courses are also evaluated by a way similar to concrete slab's case. Finally the required asphalt overlay thickness can be calculated by subtracting the total equivalent thickness of the existing pavement from the required pavement thickness.

Table 10 Equivalency Factor of Existing Concrete Slab

Condition of existing slab	Equivalency factor
Reinforced slab without pumpings and cracks	2.5
Plain slab without pumpings and cracks	2.0
Initially cracked slab without pumpings (crack spacing is greater than 1 m)	1.5
Others	1.0
Mechanically stable granular base course (used as calculation basis)	1.0

However, it has been recognized that this method gives too thick overlay when the existing slab can act as a satisfactory slab and the improvement of design method has been desired.

When reflection cracks are not expected to be appeared in the asphalt overlay layer, it has become clear that the three-layer elastic theory, where the pavement is assumed to be composed of an asphalt overlay, a concrete slab and subgrade, can describe the behavior of an asphalt overlay on an existing concrete pavement. In this case, it has been found that the critical design factor is a strain at the bottom of the existing concrete slab. Therefore, the effect of the asphalt overlay to strengthen the existing pavement can be estimated in terms of the strain.

The elastic properties of the pavement components can be estimated as follows.

① existing subgrade

Modulus of elasticity (E modulus) is estimated using Equation (6).

$$K_{75} = \frac{1}{h}\sqrt[3]{\frac{E_2^4}{E_1}} \qquad (6)$$

where, K_{75}: modulus of reaction (kgf/cm^3)

h : existing concrete slab thickness (cm)

E_1 : E modulus of existing concrete slab which can be estimated in ② (kgf/cm^2)

E_2 : E modulus of existing subgrade (kgf/cm^2)

Poisson's ratio is assumed to be 0.3.

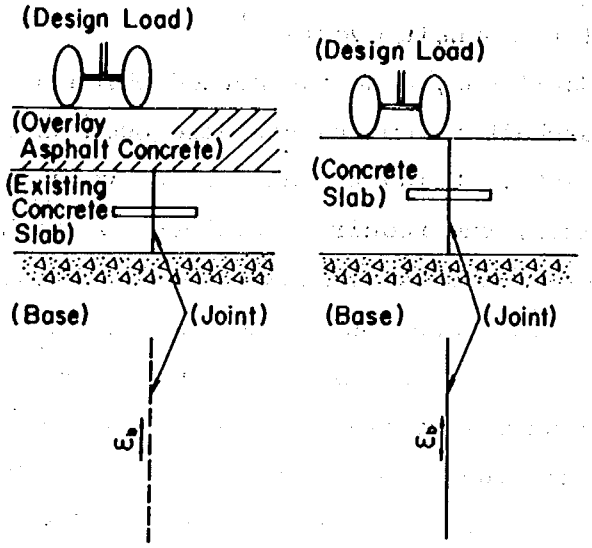

Figure 10 Schematic Representation of Structural Analysis of Asphalt Overlay

② existing concrete slab

Structural performance of a concrete slab is varied with an efficiency of load transfer at an existing joint (E_{ff}). Then, as an E modulus of concrete slab, apparent E modulus (E^*), which is shown Figure 11, must be used. E_{ff} is defined in Equation (7), which can be obtained by a loading test on the concrete slab.

$$E_{ff} = \frac{d_2}{(d_1 + d_2)/2} \times 100 \qquad (7)$$

where, d_1: deflection of a loaded slab

d_2: deflection of an unloaded slab

Posisson's ratio is 0.15.

③ asphalt overlay layer

E modulus and poisson's ratio is 7,000 kgf/cm^2 and 0.3, respectively.

For example, Figure 12 shows the required thickness of asphalt overlay for

the design aircraft of Boeing-747-200B and the efficiency of load transfer at joints of 80%.

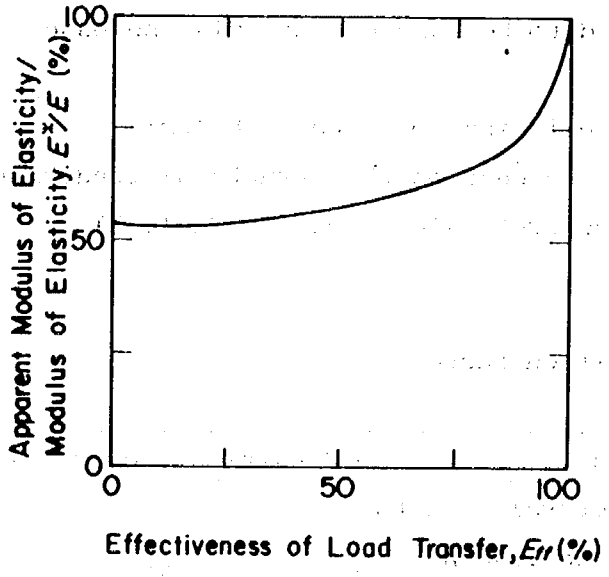

Figure 11 Relationship between E_{ff} and E^*

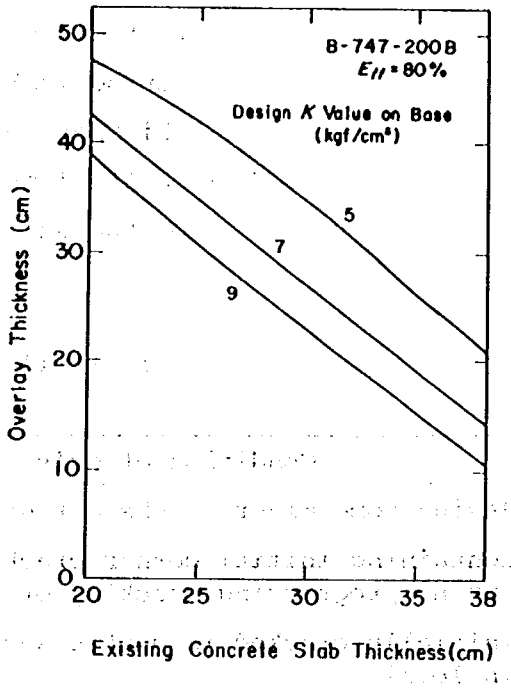

Figure 12 Overlay Thickness Design Curve for B-747-200B, E_{ff} = 80%

(2) Concrete Overlay

It is considered that the following equations, which many agencies adopt,[6] can also be used.

① bonded overlay

$$h_0 = h_d - h_e \tag{8}$$

② partially bonded overlay

$$h_0 = \sqrt[1.4]{h_d^{1.4} - C[(\frac{h_d}{h_{db}})h_e]^{1.4}} \tag{9}$$

③ unbonded overlay

$$h_0 = \sqrt{h_d^2 - C[(\frac{h_d}{h_{db}})h_e]^2} \tag{10}$$

where, h_0 : required thickness of concrete overlay (cm)

h_d : required thickness of concrete slab of which materials are expected to be used for concrete overlay, in case that it is assumed to be laid on the exist-

ing base course (cm)

h_{db}: required thickness of concrete slab by using the same materials as the existing concrete slab's, in case that it is assumed to be laid on the existing base course (cm)

h_e : thickness of the existing concrete slab (cm)

C : coefficient which depends on the structural condition of the existing concrete slab shown in Table 11

Table 11 Condition Factors

Condition of existing slab	Condition factor (C)
Having some minor cracks but no structural defects	1.0
Containing initial corner cracks due to loading but no progressing cracking or joint faulting	0.75
Initially cracked in most slabs but badly cracked in local	0.5
Badly cracked or crashed and faulted joints	0.35

In these three types of concrete overlays, at present an applicability of bonded overlay as rehabilitation measure must be checked carefully, for there still remain some problems such that bonding between an overlay and an existing slab is not perfect.

(3) Reconstruction

In general, the design procedure is almost similar to that for new construction.

In case of reconstruction with prestressed concrete (PC) precast slab, the following jointing systems between PC slabs and between a PC slab and an existing slab have been developed. Figure 13 shows the jointing system, so-called horn-joint, between PC slabs and Figure 14 shows the jointing system between a PC slab and an existing slab. It is verified that these jointing systems have been applied successfully to the rehabilitation of airport concrete pavements.[7)]

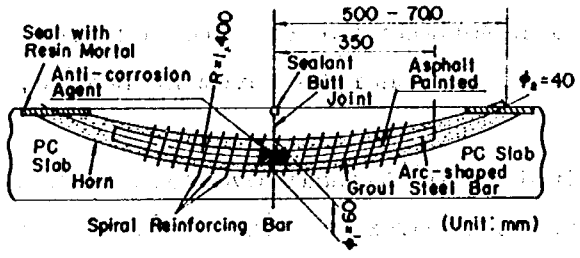

Figure 13 Horn Joint

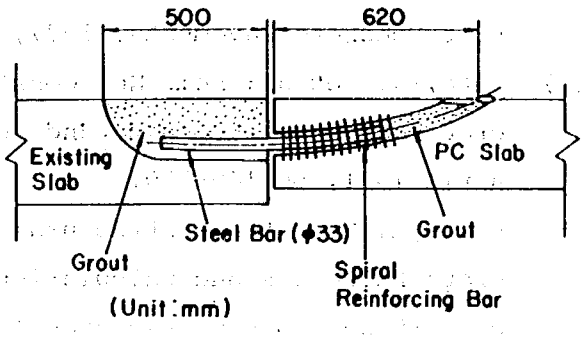

Figure 14 Jointing System between PC and Existing Slabs

5. Concluding Remark

According to the new method described in this report, it has become possible to objectively judge the necessity of rehabilitation of airport pavements on the basis of quantitative values and to predict the necessity of rehabilitation in future to some extent. In the designing of rehabilitation measures, more convenient and rational design methods of overlays have been developed and proposed.

Even though a lot of such advances have been achieved in the design of rehabilitation in airport pavements, the researches on the pavement management system in airport pavements in Japan is just started. For instance, concerning predicting the distress and performance of airport pavements as the most important item for the determination of rehabilitation policy, we have just developed the data bank for airport pavements and are collecting the actual data. We would make efforts to study many concerned subjects for developing the pavement management system.

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Water Damage in Cold Areas: Pavement Distresses, Consequences and Maintenance Alternatives

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ABSTRACT

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It has long been known that excess water in pavement structures causes a reduction in load carrying capacity. To preserve the King's Highways in Ontario, the Ministry of Transportation and Communications has extensively studied water damage to pavement structures. The problem is complicated because Ontario is a seasonal frost area in which late winter and early spring freeze-thaw cycles affect pavement.

Asphalt pavement distress directly attributed to water manifests itself in many forms. Some obvious examples are progressive pavement ravelling, cracking, and alligator cracking. Water also accelerates deterioration of pavement distresses in other ways, for example multiple cracking and potholing of cracks.

This paper identifies what water sources exist, how the problem is compounded in seasonal frost areas and the causes of pavement distress directly and indirectly related to water, such as stripping, softening of the granular bases, and construction problems.

Discussion of water damage prevention centres on design with anti-stripping agents, subsurface drainage for the granular base, preventive maintenance such as the use of prime and surface-treated shoulders, and waterproofing measures and corrective maintenance such as routing and sealing of cracks and spray patching.

Presented 1983

WATER DAMAGE IN COLD AREAS: PAVEMENT DISTRESSES, CONSEQUENCES, AND MAINTENANCE ALTERNATIVES

by

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1/ INTRODUCTION

Water damage of asphalt pavement is a constant problem for the Ontario Ministry of Transportation and Communications, particularly because Ontario is situated in the "Cold Areas" where seasonal frost action is the norm.

Most pavement damage is caused by free water in the pavement structure and inadequate drainage of that water [1, 2, 3]. The combination of free water and cold temperature subjects the pavements in seasonal frost areas to additional stresses during winter and early spring [2, 3]. Therefore, the solution to water damage is the elimination of water.

But water is a natural phenomenon, found in the form of ground water, summer rain, and winter ice and snow. There is no possibility of eliminating water or its sources.

Thus, there remains only one alternative. That is to keep water from entering and remaining in the pavement structure where damage can occur.

Free water usually infiltrates the pavement structure through the granular shoulders. It will also find its way into the pavement structure if the asphalt pavement is porous or cracked due to shortcomings in design and/or construction, and because of a high water table. If the drainage system provided is inadequate, this water will remain there, weaken the pavement structure, and act destructively in conjunction with traffic forces.

The real culprit is a combination of conditions which permit the presence of free water inside the pavement structure. These conditions are the result of inappropriate design, improper construction and inadequate drainage systems. Water is merely the destructive agent which induces pavement distresses and accelerates the deterioration of distresses.

The consequences of these water-induced damages are poor pavement performance, increased maintenance cost, and loss of pavement service life.

2/ WATER AND ITS SOURCES

Water comes in the form of rain, ice and snow, and ground water.

If the asphalt pavement is densely compacted into a designed impermeable layer, the only way summer rain can enter the pavement structure is through the granular shoulders. However, if the asphalt pavement is porous, or if there are cracks in the pavement, water can and will enter the pavement structure. It has been reported that 70% of surface run-off can enter a 0.9 mm crack, yet such a crack width is considered very slight [1].

In winter, things are a bit more complicated. Ice and snow generally remain frozen above ground. Very little free water is available unless there is a warming trend. But winter maintenance demands the removal of ice and snow from the pavement and shoulders for safe passage of the travelling public. Snowbanks are created on both sides of the roadway by plowing which alter the intended drainage paths and act as a trap for the free water created by de-icing salts on pavement surfaces (Figure 1). Some free water is also produced by direct sunlight. Blocked by the snowbanks, this free water stands on the frozen granular shoulders adjacent to the pavement edge. On warmer days during the winter, this part of the shoulder is within the influence of the heat-absorbing black asphalt pavement surface and is likely to thaw, thus permitting entry of the melt water.

However, the granular base under the pavement thaws before the adjacent shoulder, so that standing water which cannot drain down and out through the frozen shoulders enters the granular base. This snow melt is a major source of water filtering into the pavement structure [2, 3].

Another major source of water is de-icing operations using salt during winter ice or snow storms. The road salt turns the ice and snow into brine which will enter the pavement along the pavement shoulder edges, through a porous pavement surface, or through pavement cracks (Figure 2) [4]. This melt water will enter the granular base and remain trapped because of the frozen subgrade and the surrounding shoulders.

Ground water can also be a major source of water in the granular base if the water table is high and the pavement structure is built at or very close to it. During winter, cold temperatures at the surface create a thermal gradient which draws water upwards to the freezing front.

3/ PAVEMENT DISTRESS: CAUSES AND CONSEQUENCES

Pavement distress manifestations of asphalt pavement have been well documented in Ontario [5, 6]. Many of these distresses are directly related to the presence of water. Others occur for different reasons, but water accelerates their deterioration.

3.1/ Direct Water Damage

3.1.1/ Stripping of Asphalt

Stripping of asphalt is the physical separation of the asphalt cement and the aggregate. This is produced by the loss of adhesion between their surfaces, primarily due to the action of water or water vapour.

3.1.1.1/ Distress

There are two forms of stripping distress:

i) Surface Ravelling:

This results from water stripping the asphalt films from the aggregates in the upper surface of the pavements. Water is able to penetrate the surface asphalt course because of high voids in the pavement. The deterioration is generally from the upper surface downward (Figure 3).

ii) Random Cracking:

Water or water vapour trapped in the granular base acts as a stripping agent of the asphalt films from the aggregates. Stripping occurs from the bottom upward because the bottom of asphalt pavement is more extensively in contact with the granular base. Coarse aggregates strip more readily than fine aggregate particles. It is very common to see free water in the asphalt mix in extremely stripped pavement [7, 8, 9]. Random cracking and more frequent occurrences of cracking of all types are associated with stripping in the lower layers [7].

3.1.1.2/ Causes

The physical cause of stripping is water, but stripping only occurs because of features which allow the ingress of water to the asphalt pavement structure. These features are many and complex:

i) Design:

- unsuitable and/or inappropriate use of asphalt mix;
- inappropriate asphalt characteristics;
- unfavourable aggregate characteristics;
- insufficient asphalt cement;
- selection of inappropriate anti-stripping additives.

ii) Construction:

- wet or clay-coated aggregate particles;
- inadequate compaction resulting in porous pavement with high air voids;
- poor work scheduling such as paving in wet conditions or cold temperatures, resulting in high air voids in pavement.

iii) Maintenance Practice:

- snow clearing procedures such as snowbanking resulting in standing surface melt water;
- use of de-icing salt for ice and snow removal resulting in standing salt-water mixture where salt is an active agent for stripping damage.

iv) Traffic and Environment:

- water trapped under the tires is forced into pavement voids under pressure, particularly in pavements with high surface air voids;
- dense graded, well compacted granular bases are often slow to drain and can trap water under the bituminous pavement layers. Free water on the surface of the granular base under pressure from traffic will pump fines from the granular base upward into the bituminous pavement layers;
- water pressure and surface tension forces at air/asphalt/water interfaces in void air spaces initiate asphalt stripping from aggregate faces.

3.1.1.3/ Consequences

The consequences of pavement stripping are progressive ravelling and severe structure failures such as cracking and potholing (Figure 4). Ravelling of the pavement surface is visible and thus can be remedied in time. Stripping from the bottom upward is a much more serious pavement distress because its presence is hidden and is not evident until it is too late to remedy the situation.

3.1.2/ Structural Deformation

Many pavements which are structurally adequate exhibit structural distress after a few years of light traffic service [2, 3]. These distresses are generally in both wheel tracks as a result of water softening the granular base.

3.1.2.1/ Distress

There are two distresses which are interrelated:

1/ Pavement Wheel Track Rutting:
Excessive deformation of the granular base which brings on permanent deformation of the asphalt pavement because of reduction in structural support.

2/ Longitudinal Wheel Track Cracking:
Excessive stress of asphalt pavement from pavement rutting which results in cracking longitudinally along the wheel tracks.

3.1.2.2/ Causes

The direct cause of structural damage in the form of pavement rutting is water (Figure 5). But the real cause is infiltration of water into the granular base and the seasonal frost action on that water [2, 3].

During summer, water enters the pavement structure through the granular shoulders or the pavement surface via high air voids or cracks. However, base drainage is generally adequate in the summer months [2].

However, during the winter months, free water from snow melt infiltrates the pavement structure and is trapped in the granular base. Because the granular base is contained beneath by a frozen subgrade and at both sides by frozen, impermeable shoulders, a trough or bath is formed in which trapped water collects (Figure 6). The free water is usually saline, produced by snow clearing operations with salt. Salt depresses the freezing point of the water, so that freeze-thaw cycles take place at lower temperatures. Thus, during winter and early spring there is a fluctuating freeze-thaw zone which is directly below the pavement surface in the granular base and perhaps extends even into the subbase. Because of this fluctuating freeze-thaw zone, support for the still cold and brittle asphalt pavement by the granular base is drastically reduced by the development in the thaw period of pore pressure under dynamic wheel load forces. This results in excessive deformation in the

granular base which in turn leads to permanent deformation of the pavement asphalt surface [2, 3].

3.1.2.2/ Consequences

The consequences of water infiltration are permanent pavement rutting, and subsequent cracking of the pavement surface longitudinally along the wheel tracks. This distress is commonly referred to as "longitudinal wheel track cracking" (Figure 7).

If water continues to be available through the granular shoulders, the porous pavement surface, and now the longitudinal wheel track cracks themselves, the damage will accelerate into multiple cracking and alligator cracking with a drastic increase in maintenance cost and a shortened service life (Figure 8).

3.1.3/ Progressive Pavement Cracking

This type of distress is found exclusively on thin asphalt pavement (30-45 mm); usually a single lift pavement over a granular base.

3.1.3.1/ Distress

There are two pavement distresses in this form:

1/ Progressive Pavement Edge Cracking
A single crack develops approximately 15-30 cm parallel to the pavement edge, with parallel multiple cracks developing progressively towards the centre of the pavement lane (Figure 9). Water under the pavement edge causes it to heave upward. When the ice melts, traffic breaks off the unsupported edge.

2/ Elephant Footprint Cracking
A circular crack with a slightly depressed centre appears generally along the centreline of the pavement (Figure 10).

3.1.3.2/ Causes

Although water is the chief problem, the real causes are:

i) Design

- insufficient asphalt cement;
- inappropriate selection and/or gradation of aggregates;
- insufficient asphalt pavement thickness for the environment.

ii) Construction

- inadequate compaction resulting in porous pavement, particularly due to thin courses and improper centreline joint compaction;
- poor work scheduling such as paving in wet and cold weather resulting in high air voids for the asphalt pavement.

iii) Maintenance Practice

- 'centre-bare' standard de-icing and snow removal operations leaving salt water mixture standing on the pavement surface and the granular shoulders along the pavement edge.

The mechanism for water-induced damage is similar in the two distresses and similar to that explained earlier for pavement rutting and longitudinal wheel track cracking.

Salt water mixture is allowed to stand and infiltrate into the granular base along the pavement edges and through the porous pavement surface. This water is trapped in the granular base directly under the asphalt pavement. The same fluctuating freeze-thaw zone causes the pavement edges to heave (Figure 11). The thin asphalt pavement, cold and brittle, fractures under this stress. As more cycles of this freeze-thaw occur, cracking will progress inward (see Figure 9).

The elephant footprint crack occurs under similar conditions except its form and location depend on the porosity of the pavement surface.

An example showing the combined effects of inappropriate design, improper construction and current maintenance practice on the asphalt pavement resulting in the elephant footprint cracking is provided by Contract 80-48 located in the Eastern Region of Ontario.

Laboratory test results shown in Table 1 indicate that the pavement has:

- Low Asphalt Content -- 0.4% below mix design requirement.
- High Sand Content -- 9% more passing through a 4.75 mm sieve than design mix specifies.
- Poor Compaction -- average only 92.5%; construction requirement is 95.3%.
- High Permeability -- asphalt pavement has high permeability which is evident from the high pavement voids of 11.3%.

Additionally, construction records indicated that a haul of approximately 50 km was made from asphalt plant to job site. Therefore, the possibility of cold leftover lumps of asphalt in the paver's hopper exists because of a wait for new shipment from the mixing plant. This may result in cold but workable asphalt mixing with new asphalt causing low compaction results.

'Centre-bare' snow removal operations with salt usually leave a salt water mixture standing on the centreline of the pavement. For this porous pavement, the snow melt filters through the pavement surface and is trapped in a round pool between the thin asphalt pavement and the granular base. The brine freezes first at the edges of the pool and draws water towards the freezing front. The front causes heaving at the pool edge which becomes cracked (Figure 12).

3.1.3.2/ Consequences

Once the initial crack has developed and there is a continuous supply of water, the distress will progress into multiple cracking, alligator cracking and potholing.

3.2/ Water Accelerated Damage

There are pavement distresses which are not originated by water. The most prominent example is transverse cracking. However, water is the most damaging factor and its mechanism is common to all other cracking.

3.2.1/ Transverse Cracking

Transverse cracking generally begins as a hairline crack and slowly extends and widens with time. Ontario has carried out a study which extensively covers the phenomenon of crack development and movement due to seasonal changes throughout the year (Figure 13) [10, 11].

Ontario historical data indicate that transverse cracks open from 7 to 10 mm during the cold months and do not entirely close the following summer, leaving a residual crack opening which grows by 1 mm per year (Table 2). In many older pavements, say 20 years old, this residual movement produces a permanent crack width of 25 mm and over. Transverse crack widths of more than 50 mm were observed during this study [10].

3.2.2/ Water Damage

In the summer, surface runoff from rain water can enter the pavement through the transverse crack, even if it has closed to 0.9 mm [1]. This free water is now available to cause or to accelerate stripping of the asphalt. It is also available to carry fines from the granular base to the surface under the pumping action of traffic [4, 11].

The worst possible situation is winter. The same cracks open up and water is practically drawn into them; water which has a high content of salt from de-icing operations (see Figure 2). This salt water subsequently freezes and thaws more times in lower temperatures than pure water would and this can create cupping or lipping at the crack (see Figure 14).

3.2.3/ Consequences

Cupping or lipping significantly contributes to pavement surface roughness. There is residual roughness at the cracks even after the spring thaw. The result is crack spalling, multiple cracking, alligator cracking, potholing and subsequent shortening of service life (Figure 15).

4/ PREVENTIVE MEASURES

The possibility of eliminating free water from the pavement structure is zero. Therefore preventive measures must concentrate on preventing water from entering the pavement structure and providing sufficient means to remove infiltrated water.

These measures form four distinct groups:

1/ Prior measures incorporated into the design and construction stage.
2/ Drainage.
3/ Waterproofing of the pavement during and after construction.
4/ Corrective waterproofing of the pavement surface.

4.1/ Design and Construction

4.1.1/ Use of Anti-Stripping Additives

Recognition of a potential stripping problem during the mix design procedure is an important first step in avoiding future water damage. This is most readily accomplished during the vacuum procedure for measuring voids in the mineral aggregate (VMA). At this time, the coated aggregate is immersed in water at 60°C (140°F) and if significant amounts of asphalt cement are observed floating on the water surface after vacuuming, and if coating loss is observed on aggregate surfaces, then stripping is likely to be a problem.

The effectiveness of anti-stripping additives such as hydrated lime and proprietary chemical compounds can then be assessed for mixes for potential stripping problems by a series of Marshall Immersion tests, a

procedure developed by Field and Phang [8]. This test has been used in Ontario for about 18 years. While there is some criticism that the test is too severe and does not represent field behaviour, nevertheless it is sufficiently discriminating to identify successful anti-stripping additives, and it will establish the heat stability of the additive.

While the most consistently successful additive so far is hydrated lime added at a level of about 2% of the aggregate, it has not been used to any extent because of difficulty in mix design and in handling during construction. Chemical anti-stripping additives are not widely used because of the added costs involved and because of their rather mixed success. At a few locations, examination after 5 years service revealed unexpectedly high stripping where an anti-stripping additive had been used, however many projects were very successful.

4.1.2/ Sand-Asphalt Layer

A technique which has also met with mixed success is the use of a 25 mm layer of sand asphalt laid on the granular base to isolate the lower part of the asphalt mixes from water in the granular base. It was believed that because stripping of asphalt from sand size particles is more difficult than stripping from coarse aggregate particles, that the sand asphalt layer would at least delay the upward progression of stripping.

Examination of many projects constructed this way revealed that the majority of projects were successful, but the level of success has not yet been sufficient to offset the added costs of about 2% more asphalt cement in the sand-asphalt mixes as compared with normal gradation mixes. This method of construction has been cancelled because of the increasing price of asphalt cement.

4.1.3/ Minimum Thickness

Pavement lifts which are spread too thin, cool off very rapidly and are prone to being poorly compacted. This permits the moisture to pass through the mix, causing failure in bond at the interface with

underlying layers, and allowing traffic to lift out pieces of the pavement. This does not usually occur when the thickness of lift is 40 mm or more.

4.1.4/ Asphalt Cement Content

In recent years, mix designers have tended to design mixes containing less asphalt cement than formally because of the elevated prices. Ten years ago mixes were, on the average, designed with 5.5% asphalt cement. Currently, asphalt contents tend to be about 5.0%. Not only are these mixes stonier and thus harder to compact, but thinner films tend to harden quicker so that durability may be lost.

4.1.5/ Compaction

Examination of ravelled pavements has shown that water penetrates from the top when the air voids in the pavement exceeds about 9%. Ravelling becomes extensive when voids are about 12%.

Mixes are generally designed with from 3 to 5% of air voids in a fully compacted mix (75 blows Marshall per side). Mixes compacted to the 97% requirement will generally contain up to 7% air voids. Mixes which are 95% compacted or less can have air voids of 9% or more.

Compaction becomes more difficult principally due to high viscosity of the asphalt and high percentage of crushed faces, particularly in the finer fractions. Viscosity is controlled by temperature, so that paving in cold, wet weather and using thin lifts of asphalt mix which cool off rapidly, are both inconsistent with good paving practice. Unfortunately, many projects are poorly scheduled, and too much paving is carried out in late fall. Keeping the breakdown roller very close behind the paver, and raising mix temperatures by a few degrees are measures which will aid compaction during fall months. Rubber tire rollers used as intermediate rollers will create a seal on the surface. Steel finish rolling will remove tire undulations, but care must be taken not to finish roll while the mat is still hot, as 'hair checking' will occur which will open up the surface. Finish rolling is best done at a

surface temperature of about 50°C (120°F), when a person's hand can be held against it without too much discomfort.

During rolling procedures when paving an adjacent lane it is very important to avoid building-in an uncompacted zone which will result in early opening of the centreline joint due to cold temperature shrinkage. This is particularly important with single lift pavements.

4.1.6/ Partial Paved or Surface Treated Shoulders

In part because of pavement edge weakness caused by moisture infiltration through the adjacent granular shoulder, and in part to overcome pavement edge drop-off vehicle safety problems, the use of partially paved shoulders and of shoulder waterproofing with prime and/or surface treatment is becoming more acceptable in spite of increased costs. Partial paving extends the paved surface by 0.6 m, and this width is constructed at a 6% crossfall. Early cracking in one-lift partially-paved shoulders has raised controversy over whether this should be continued or if two lifts should be specified.

In lighter traffic roads where pavement edge problems are apparent, waterproofing of the shoulder with prime or with a single surface treatment has been effective. However, these treatments do not last for more than a few years and must be repeated.

4.1.7/ Pavement Edge Drains

On important routes which carry heavy traffic, additional preventive measures can be justified. Pavement edge drains are installed to intercept water infiltrating from the shoulder and quickly remove water entering the pavement structure through cracks and porous pavement. Because of low cost and ease of installation, the pavement edge drains favoured for this purpose are 10 cm filter-wrapped flexible plastic pipe. This is plowed in under the shoulder using a special plow (Figure 16) [3]. The drain outlets to the ditch are at intervals of about 150 m, and at low points.

4.2/ Preventive Maintenance

Preventive maintenance, like the proverbial stitch-in-time-saves-nine, prevents and/or reduces the amount of water infiltrating the pavement structure, thereby minimizing the effects of water damage and making pavements more serviceable over a longer period of time [12].

4.2.1/ Prime or Surface-Treated Shoulders

There are numerous methods of shoulder sealing for preventive maintenance but the cheapest and easiest is priming or surface-treating.

The Ministry constructed test sections of primed shoulders at several sites in Southern Ontario. One of the treatments used was RC-30 prime over a 0.9 m width of shoulder adjacent to the pavement. After 3 years of observation, much of this treatment was still in place [3].

Surface-treated shoulders test sections using a high-float RS-1 K emulsion and 16 mm crushed gravel were constructed in the Ottawa District. Observations indicate that the treatment is still successful and in good condition after 2 years of service [3].

4.2.2/ Fog Coating of Pavement Surface

Design and construction practices sometimes result in an inadequately compacted asphalt pavement with a high percentage of air voids. A good preventive maintenance measure to correct the porous pavement surface is an emulsion fog-coat which fills the air voids in the pavement as well as enriching the asphalt cement content of the top of the mix.

The Ministry has not used this maintenance alternative recently because bleeding or flushing may occur.

4.2.3/ Routing and Sealing Cracks

Dense-graded asphalt pavements are more or less impermeable for a period of time after construction. Eventually, cracks of various types will develop either as water damage or for other reasons. These cracks represent a significant means of entry for free water.

The Ministry has been very active in evaluating new products, equipment, and methodology for crack sealing [4, 13]. Province-wide guidelines have now been established for routing and sealing cracks based on initial trial guidelines established for our Southwestern Region [4].

The provincial guidelines recognise the environmental and climatic differences between the northern and southern parts of our province [14].

In brief, the guidelines state that high-spec sealants on the Ministry's Designated Sources List are used only for the north and the routed channel for the sealant is to be wider and deeper (19 x 19 mm) so that more sealant material is provided against the colder climate [14].

The Ministry is currently experimenting with a new routed shape, a V-shape channel of 40 mm wide by 8-10 mm (maximum 10 mm) deep [4]. This channel shape has gained considerable acceptance in 1983 by both our own maintenance field staff and by contractors in the private sector because of ease of work and superior performance (Figure 17). Specifically, it allows:

- easier router operation since cracks are simpler to follow because of the width of the cut;
- less demands on the mechanical router, so there are cost saving on energy and possibly maintenance;
- longer life for router bits; and
- faster cutting time.

Observations of the field performance of V-shaped channels shows that the shape is superior to the standard square routed and sealed shape because it presents:

- neat appearance;
- more bonding surface;
- a shape which induces less bonding stress; and
- recessing against possible snowplow damage.

4.3/ Corrective Maintenance

Corrective maintenance measures are usually done to maintain a tolerable level of serviceability and to retard further damage.

4.3.1/ Routing and Sealing Cracks

Although this treatment is generally viewed as preventive maintenance, it is useful as corrective treatment to retard further water-induced pavement damage.

Cracks, such as transverse cracks, which have large winter movement will permit significant amounts of water into the pavement. If the crack is not sealed, cupping or lipping can occur with resultant roughness. This damage is permanent and the residual distortion will increase pavement roughness, bringing about permanent serviceability loss.

Therefore, even if slight cupping or lipping has occurred, sealing the crack can delay the results of increased roughness, crack spalling and/or potholing.

4.3.2/ Spray Patching

This corrective maintenance treatment is generally viewed as a temporary measure. However, it can be very effective as a sealing treatment to retard further water damage such as pavement surface ravelling, early stages of multiple cracking and alligator cracking, slight to moderate pavement rutting and early stages of multiple longitudinal wheel track cracking.

On the severely ravelled Highway 417 in Ottawa, this treatment was applied using a RS-1K emulsion and 5 mm washed stone screening. The emulsion was sprayed to the standard 1.8 L per square meter, stone screening was applied, and it was levelled and rolled (Figure 18). The sealed surface has successfully passed three winters and is still in very good condition (Figure 19).

The Ministry has recently developed small portable tools for this treatment. The equipment is a hand-propelled mini spray-bar and a modified roofing gravel spreader (Figure 20).

A number of test sections covering various pavement distresses will be constructed in 1984 using this equipment and standardized work procedures.

5/ SUMMARY

Water damage of asphalt pavement will always be a problem for highway engineers. However, it can be minimized by consideration of all the potential causes and choice of appropriate solutions:

- The potential stripping of asphalt cement resulting from exposure to water should be considered in the design of pavement and asphalt mix.
- Dense-graded asphalt pavement should be used wherever possible to assure thorough compaction of the asphalt pavement.
- Work should be scheduled to avoid late fall paving.
- Positive drainage, both surface and subsurface should be provided for rapid water removal.
- For new construction, designs with positive internal drainage (such as open graded granular base course) and external drainage (such as full width paved shoulders) should be considered,
- Partially paved and sealed granular shoulders should be considered.
- A corrective maintenance schedule aimed at minimizing infiltration of surface water should be maintained.
- Appropriate maintenance methods, materials and equipment which can effectively waterproof the pavement over an extended period should be maintained; for example, crack rout and seal should have at least five years of effective life.

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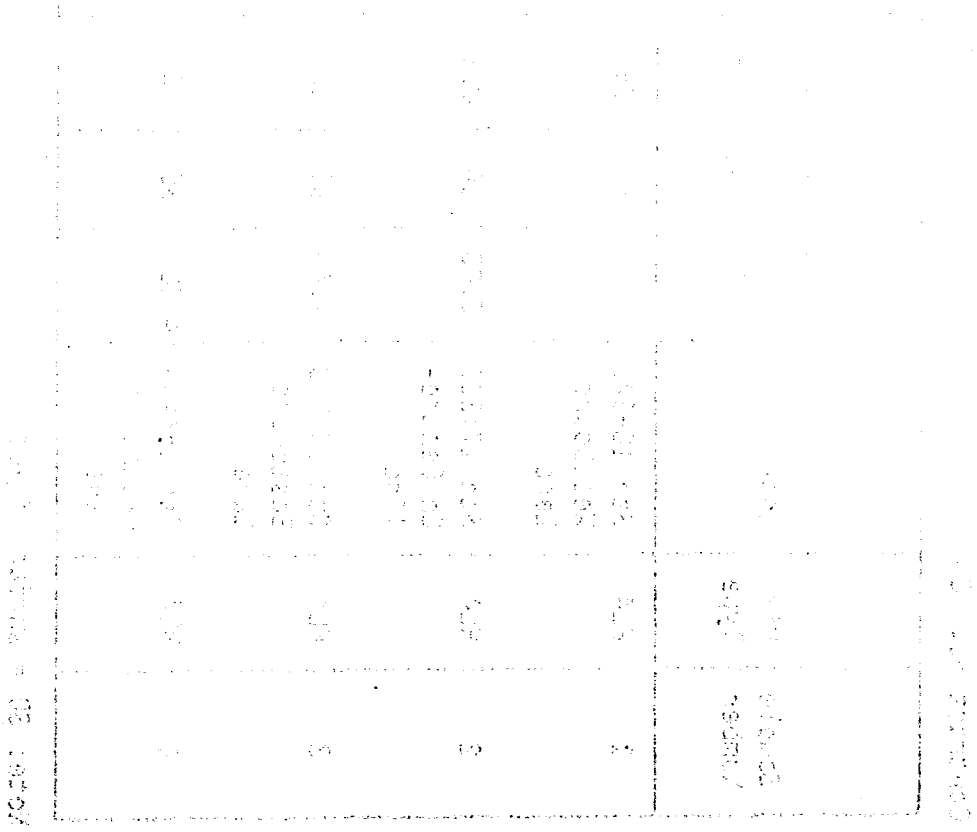


Table 1/ Laboratory Test Results of Asphalt Pavement Samples

Contract No. 80-48 Location: Hwy 500, Denbigh Westerly 10 km Date Sampled: Jan 21/82 Date Tested: Mar 5-6/82

Sample Number	Mix Type	Location	Absorption			Marshall Re-compaction			Pavement Sample			
			% AC	Pen	Visc	% Voids (SD)	Flow	Stab N	% Voids (SD)	% Comp (Expect)	Depth mm	VMA (Corr)
1	HL4	Stn 18+523 Westbound Lane	4.59	80	357	4.1 (3.5)	10.1	17 498	12.7 (12.1)	91.0 (95)	35	14.9 (14.3)
2	HL4	Stn 17+800 On Centre-line	5.03	84	338	4.8 (4.2)	12.0	17 183	11.6 (11.0)	92.9 (96)	41	16.7 (16.1)
3	HL4	Stn 13+950 Eastbound Lane	4.48	96	321	3.9 (3.3)	12.6	21 776	12.1 (11.5)	91.5 (94)	32	14.6 (14.0)
4	HL4	Stn 18+495 Eastbound Lane	4.80	86	334	4.5 (3.9)	11.1	19 422	9.4 (8.8)	94.6 (96)	51	15.8 (15.2)

Note: SD = surface dryed

Table 2/ Average Crack Movements of Transverse Cracks

Year	Crack Opening		Residual Movement	
	Inches	mm	Inches	mm
1961-2	.35	8.9	0.07	1.8
1962-3	.28	7.1	0.04	1.0
1963-4	.20	5.1	0.00	0.00

From Reference 10

Figure 1/ Snowmelt Trapped by Plowed Snowbanks

Figure 2/ Salt Water Mixture Entering Pavement Structure through the Crack

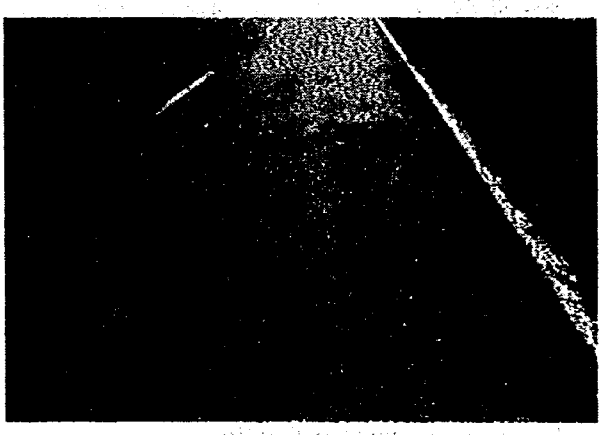


Figure 3/ Progressive Pavement Ravelling

Figure 4/ Potholing Resulting from Stripping

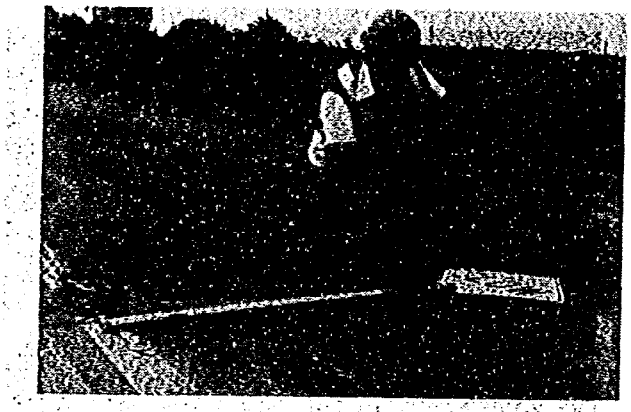


Figure 5/ Pavement Rutting

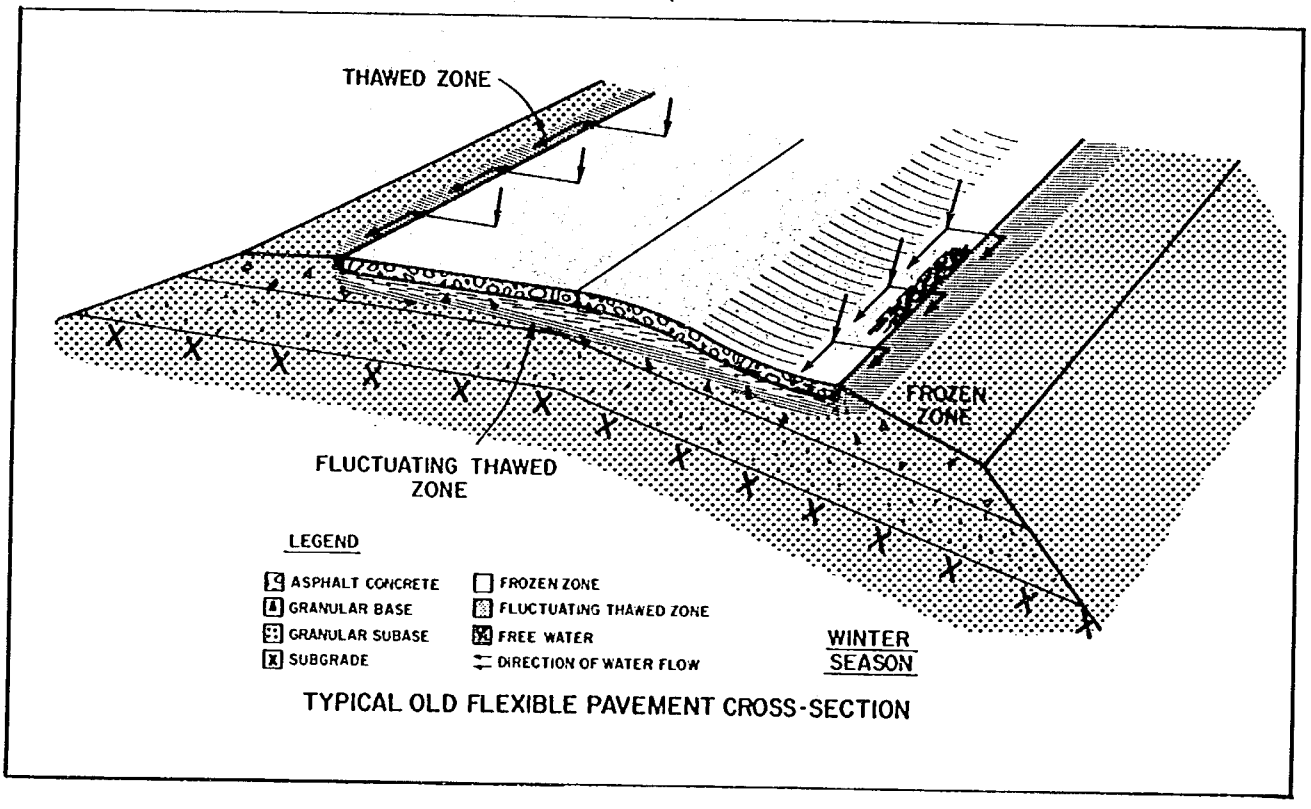


Figure 6/ Water Trapped under Pavement by Frozen Shoulders
Note: rutting in both wheel paths is due to displacement in the granular base.

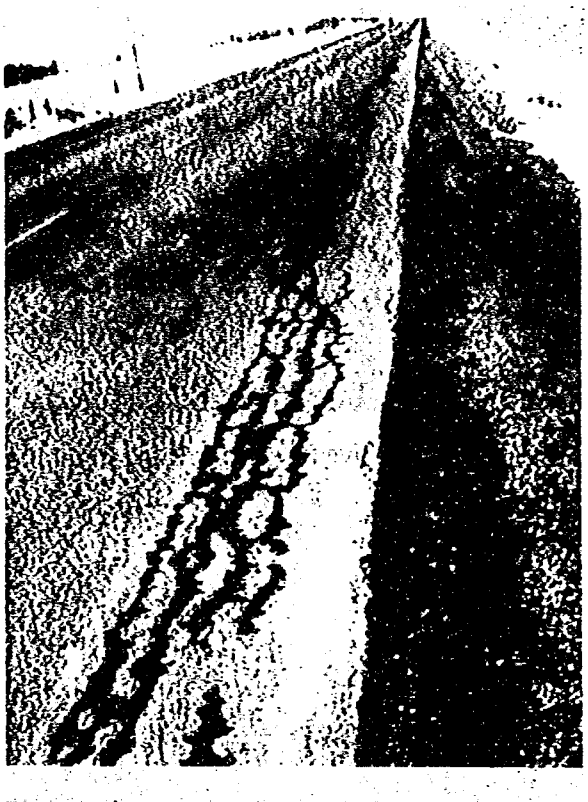


Figure 7/ Moderate Multiple Longitudinal Cracking (Outer Wheel Path)

Figure 8/ Alligator Cracking of Wheel Track

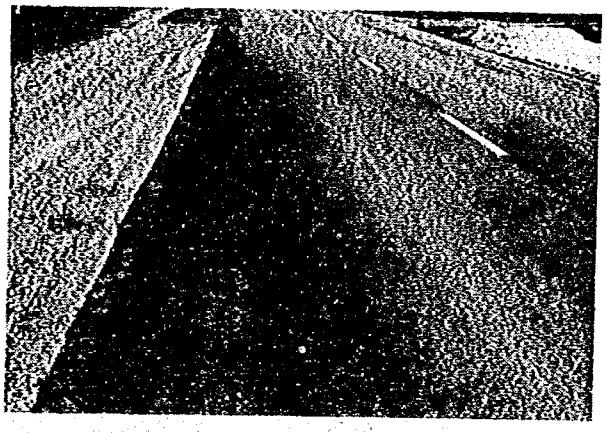


Figure 9/ Progressive Pavement Edge Cracking

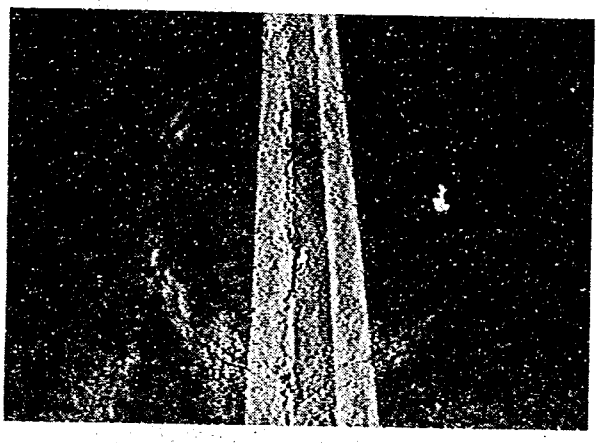


Figure 10/ Elephant Footprint Cracking

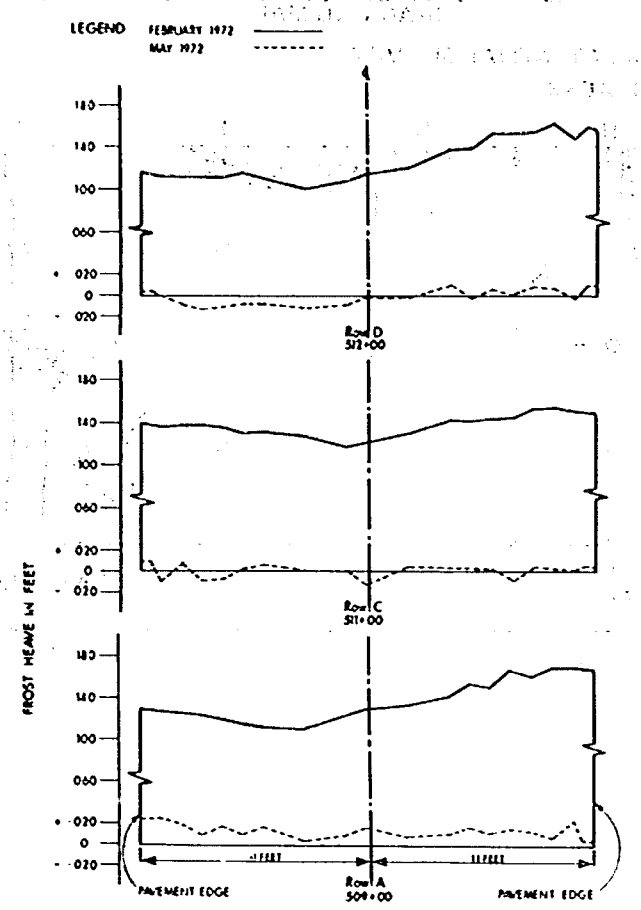


Figure 11/ Differential Frost Heaving of Pavement Edges (Hwy 89 East of Conn)

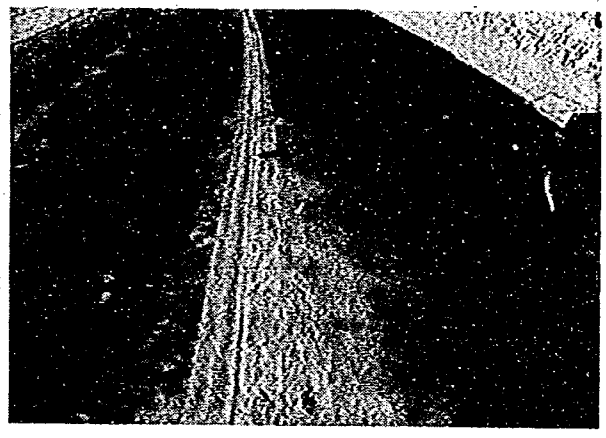


Figure 12/ Localised Heaving from Salt Water Mixture Filtering into the Porous Pavement Surface

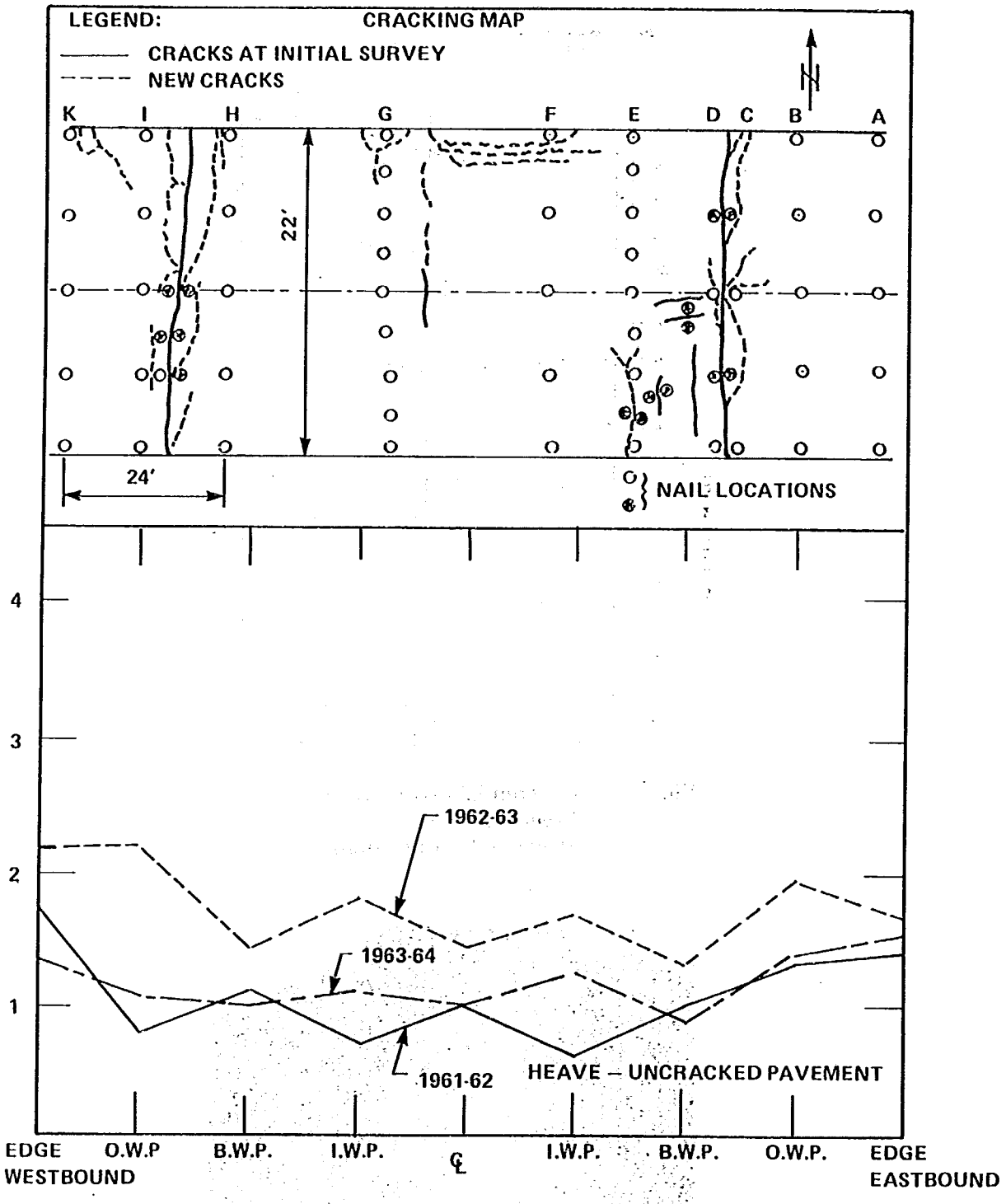
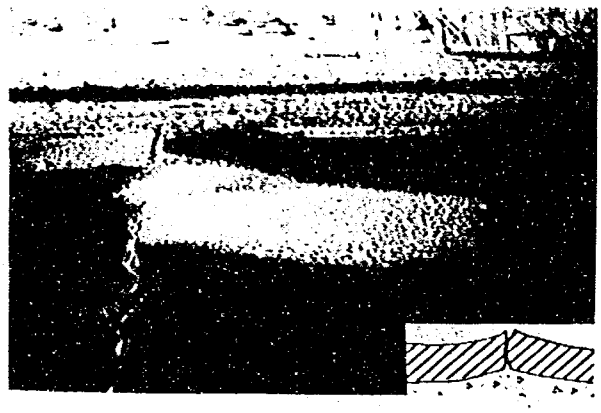


Figure 13/ Heave and Cracking Data



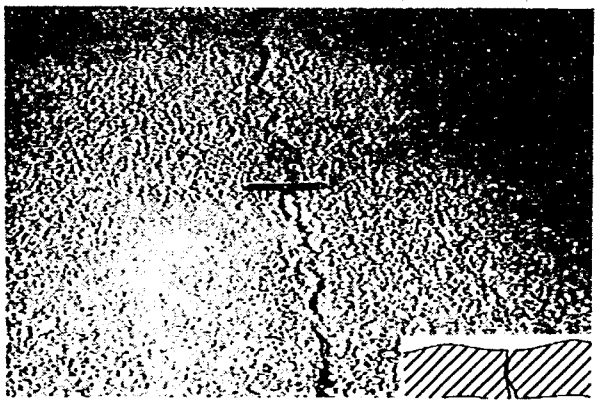


Figure 14/ Cupping or Lipping of Transverse Cracks

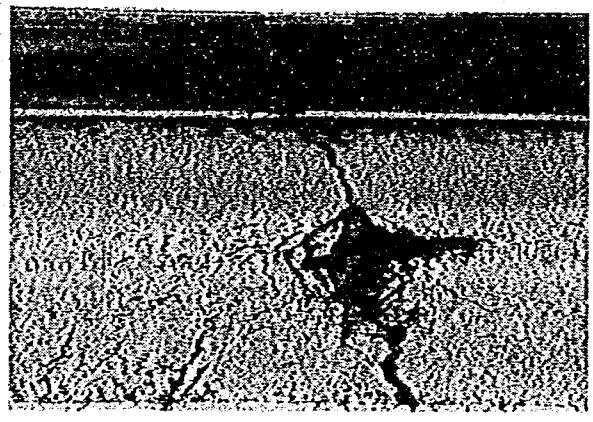


Figure 15/ Potholing of Transverse Crack

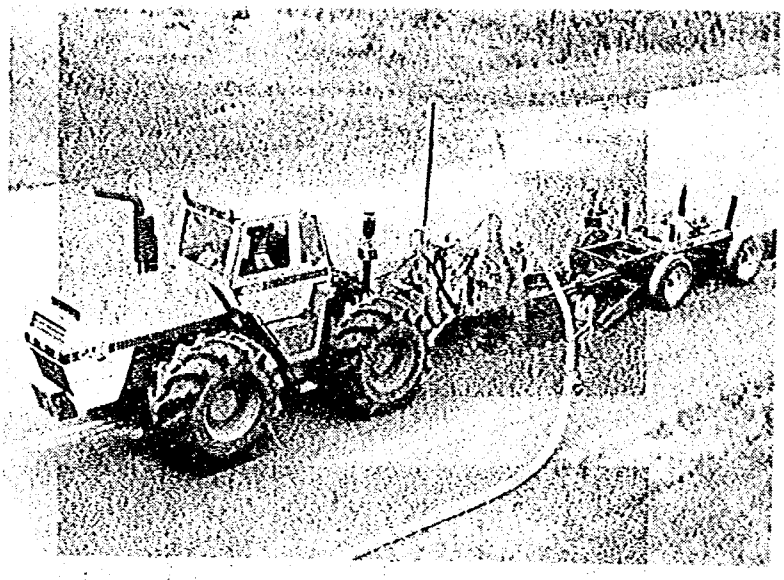


Figure 16/ Trenchless Plow Mounted on a Rubber-Tired Tractor

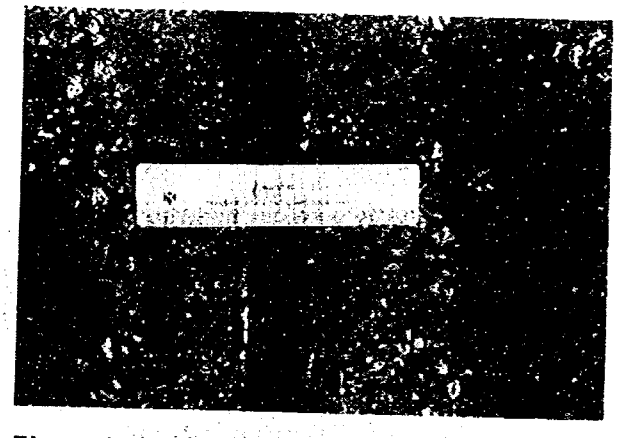


Figure 17/ V-Shape Rout 40 mm by 10 mm (maximum)

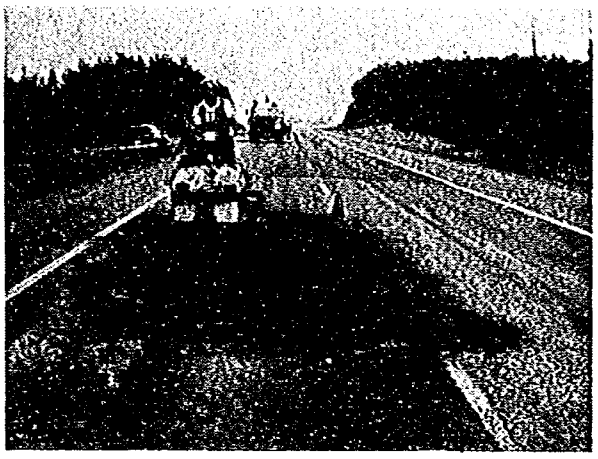


Figure 18/ Emulsion RS-1K sprayed (1.8 L/m^2) Stone Chips Applied, Leveled and Rolled

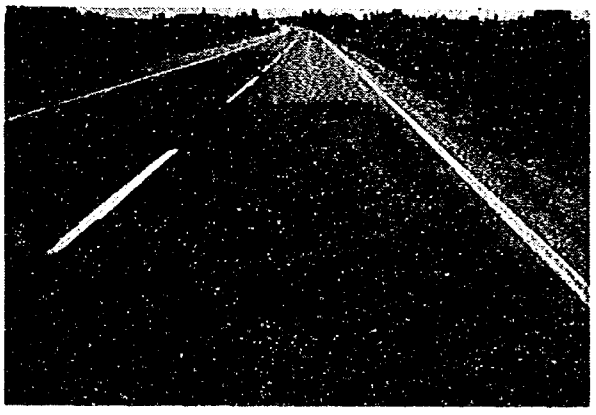


Figure 19/ Spray Patch Still Effective after Three Winters

Figure 20/ Mini Spray-bar and Roofing Gravel Spreader for Spray Patching Treatment

Performance and Reparation of Thermal Crackings in Asphalt Pavements

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ABSTRACT

Since the thermal crackings of asphalt pavements are seen prevalent in about 17% of the national roads in Hokkaido, preventive measures as well as the reparation of cracks are of a major importance.

Cracks were investigated and observed in terms of their extent of occurrence, characteristics, and yearly changes in lengths and widths, both on the national and the prefectural roads. The behaviors of crack openings were measured with self-registers, and were evaluated with the atmospheric temperature and gradients.
As a result, it was confirmed in detail that cracks could be expanded suddenly and that the residual widths of cracks are changed with the stress-relaxation of asphalt pavements.

For the repairing methods of crackings, test paving constructions were conducted using overlays of different thickness and iron mesh buried in the pavements. The performance of the repaired pavements was evaluated. Cracks were also repaired by filling materials. Two methods were used for this: 1) Blow-in fill through narrow nozzles; 2) Filling in after reshaping cracks into V shapes. Several types of fill materials were used.
It is suggested that further studies should be directed to improve the filling methods and primer, rather than filling materials.

PERFORMANCE AND REPARATION OF THERMAL CRACKINGS IN ASPHALT PAVEMENTS

by

H. Kubo, S. Kumagai and M. Oguri

Civil Engineering Research Institute, Hokkaido Development Bureau

1. Preface

Thermal crackings in asphalt pavements are one of the most serious problems in the cold regions. In Hokkaido, the 1978 survey revealed that about 17% of the national roads and 10% of the prefectural roads were affected by such cracks.[1)]

In order to have basic data on the prevention of the cracks, performance of thermal crackings were observed in test sections to determine their characteristics, relations among the causes, occurrences and the temperatures, and changes in the distributions as well as in widths. Particularly, the behaviors of the cracks were closely investigated by self-registers, thus confirming the movements in the crack openings in detail.

As for the repairing methods of cracks, test paving sections were set up to observe the repairing effects, using different thickness of overlays with iron mesh, where cracks were concentrated.

Also, where there were comparatively less cracks, cracks were filled in by blow-in nozzles, or were reshaped into the V shape before filling in. Several kinds of fill materials were used for the tests.

This paper describes the thermal crackings and the results of their repairing experiments in Hokkaido, at the same time presenting the progress results on the test paving sections constructed to prevent the thermal cracks.[2),3)]

2. Behavior of Thermal Cracks

In order to prevent thermal cracks, it is necessary to observe the movements of cracks. In view of this, the cracks being appeared in the pavements at the test paving sections were measured in the movements of their widths by self-registers and calipers.

2-1. Measurement Sites and Pavement Structure

The behavior of cracks were measured in pavements at the following two sites.

(1) The 1971 construction: National Road Route 242, Nishi-Issen, Ashoro-cho

(2) The 1977 construction: National Road Route 275, Uenbetsu, Horokanai-cho

The pavement structures for both sites are shown in Table 1. The thicknesses of the mixture layers were 12 cm for both sites.

2-2. Self-registers

In order to measure the movements of cracks, jointmeters as shown in Photo 1 were buried vertically in cracks in the surface course. The strains of the guage generated by the contraction and expansion of the pavements were measured with the conversion into electrical resistance.

Measurements by self-registers at the Ashoro Site were recorded with the temperatures and the widths of the cracks separately. However, at the Horokanai Site, the temperatures and the widths of cracks were recorded on the same paper, using multipoint registers with battery charge type instrument driven by solar battery.

The temperatures of asphalt surface course and the atmospheric temperatures were measured using thermocouple with copper-constantan at the Ashoro Site and platina resistors at the Horokanai Site.

The self-registers at both Sites were operated for one year, and the measurements with calipers were continued for several years.

2-3. Movement of Crack Openings

The movements of crack openings may be mainly caused by the expansion and contraction of pavements. The distributions of cracks in the pavements at the two measuring sites are shown in Figure 1.

Figure 2 shows crack movement and temperature fluctuation of one day in fall season at the Ashoro Site. When the temperature in 2 cm deep asphalt mixture below the road surface is related to the movements of cracks, it is seen that the widths in the low temperature range move according to the changes in temperatures, whereas the movements in the higher temperature range are small. However, the amount of these movements seems to be different from season to season.

Based on changes in the widths of cracks and temperatures of 2 cm deep asphalt mixture in Figure 2, the linear expansion coefficient is assumed to be around $1.5 \sim 2 \times 10^{-5}$/°C.

Figure 3 indicates the relationship among average daily temperature, maximum daily temperature differentials and the changes in crack widths of asphalt pavements. The diameters of the circles denote the quantities of the changes.

As seen from the Figure, changes in the crack widths are negligibly small in the high temperature range of 20 °C or more, suggesting virtually little contraction and expansion in the pavements. On the contrary, in the low temperature range of more than 10 °C differentials in the temperature, it shows prominent movements. The maximum contraction during a low temperature period with high temperature differentials during the observations reached as great as 2 mm a day.

Significant contraction and expansion in the pavements were observed for a period of 30 days during the severe cold period in January through February as shown in Figure 4. This phenomenon appeared several times a day. The reason for this phenomenon could be that asphalt pavement has little stress-relaxation in the low temperature during the severe cold period so that asphalt pavement fails to move on subbase with the friction and depressing temperature.[4] Thus, the stress is built up great enough to break up its critical range and is to make sudden movement.

2-4. Yearly Change in Crack Width

Figure 5 shows annual movement of crack width in the average monthly temperatures of the pavements both at Ashoro and Horokanai. Figure 6 shows the daily movement with the average values plotted by 10 days for one year period.[5]

It is seen from two Figures that the crack openings at the two sites reach the maximum values during mid to late February and that the crack openings show the minimum in late September at the Ashoro Site and late July at Horokanai Site. It was also found at the Ashoro Site that the crack openings did not return to the initial sizes, leaving 2 mm residual openings. At the Horokanai Site changes in crack width with the caliper-measured data indicate the existence of residual opening.

According to these Figures the movements of the cracks at the pavement temperatures 20 °C or more are almost none. This may cause residual openings with the annual repititions of contraction and expansion. In other words, the main cause of this phenomenon is stress-relaxation of asphalt pavements. The amount of changes in the crack widths shows as great as 4 ~ 10 mm at the Ashoro Site. Photo 2 shows the changes in crack widths at the Ashoro Site observed from October 1979 to October 1980, with the maximum openings of 20 mm in February. The changes observed at the Horokanai site were 1 ~ 2.5 mm.

It was confirmed that all cracks continued the yearly expansions of their openings. The amount of the expansion was as great as 1 - 2 mm at one year at the Ashoro Site, while that was almost 0.2 - 0.5 mm in Horokanai Site. The use of the jointmeters greatly helps to clarify the relations between the crack widths and temperatures of asphalt pavements.
The yearly fluctuation in the crack widths is significant, and yearly crack openings make more difficult to the crack sealings.

3. Test Pavements on New Roads[3)]

Based on the results of the 1978 investigations, test pavements were installed to find preventive measures of thermal cracks, using mix design and quality of specifications with various kinds of materials. The observations for effectiveness of the tests have been continued.

3-1. Test Location

Three newly constructed roads were selected for the testing. The 12 cm thicknesses of the asphalt mixtures were comparatively thin and the test sections are located in the regions with severe cold climate and low traffic volume in Hokkaido.

(1) National Road Route 275: Uenbetsu, Horokanai-cho (1978 construction)
(2) National Road Route 275: Moshiri, Horokanai-cho (1979 construction)
(3) National Road Route 273: Nukabira Sannosawa, Kamishihoro-cho (1981 construction)

3-2. Outline of Test Pavements

The used materials, mix design and the properties of mixtures adopted for the test pavements are shown in Table 2, Table 3 and Table 4.
Each site had special specifications in the field of:

Site (1): Rubberized asphalt mixtures, quality of coarse aggregates and materials for subbase.
Site (2): Hardness of asphalt, quantities of fillers and type of asphalt mixtures
Site (3): Different penetration with modified asphalt. Although the same grain sizes were used for the surface layer mixtures, the quantities of asphalt were changed with the straight and modified asphalt.

3-3. The Development of Cracks

The progress of cracks at Uenbetsu Site, Horokanai, is indicated in Table 2. Construction Section A is of a flat filled section and B is of a cutting section with a slight longitudial gradient, on the outskirts of a mountain. The distance between the two sections is only 1 km. For all this, cracks were developed only in Section A. The cause might be for a topographic reason or differences in the used materials.

As for the construction site A, although this area has heavy snowfall, cracks in the pavements appeared in 1980 in three winter from the construction, when the frost index marked the highest of all. Since then, numbers of cracks are increasing through the frost indices being comparatively small. The majority of the cracks were observed developing in curved portions of the road where the asphaltic mixtures of road shoulders were thinner.

It is considered that the types of subbase materials play the major role in the development of cracks. All of the three blocks in Section A with pit-run gravel used, developed cracks, whereas no cracks were observed in the blocks where crushed rock were used. This is considered dependent on the differentials in the restaining force. Namely, the stress in the asphalt mixture layer seemed to be dispersed through the greater friction force on crushed rock.

The amount of water absorption in crushed rock has, considerable influence over the development of cracks. More cracks were observed in Block 4 whose materials had more water absorption.

Table 3 shows the test description at the Moshiri Site, Horokanai. Although this area is located in an severe cold region, only a few cracks were developed in the 4th winter.

Complete removal of snow from the pavement was never done with the protection of the pavement surface from cooling.

Asphalt with low penetration seems to be a primary factor of causing cracks. The development of cracks is also considered to have a relation with overlapping curves in the section of low penetration.

Due to the fact that the overall development of cracks was low, analyses on the kinds of asphalt mixtures and the amount of filler cannot be made at this occasion.

The test progress of Nukabira Site is shown in Table 4. No thermal cracks have been observed to this date after two winters. Hairline type cracks have frequently been observed on the surfaces of the white painted lines.

Based on the test pavements on the new roads, the use of crushed rock is very effective to prevent thermal cracks. Also, high penetration asphalt seems to be effective.

4. Experiments for Repairing Cracks[3)]

Cracks in pavements must be repaired as soon as possible, because cracks introduce the penetrations of rain or thawing water assisting to weaken the bearing capacities of the subbase as well as subgrade leading to a total failure of the road structure. The extent of reparation and the methods are varied with the distribution and progressing degrees of cracks. Presented here are the results of repairing experiments carried out with the methods of overlay and filling in with asphalt materials.

4-1. Prevention of Reflection Cracks by Overlay

Overlay work is required when the surrounding pavement of cracks are caved in or when the distance between the cracks comes closer than 10 m to progress the breaking to form map crackings.

According to the maintenance practice at the sites, patching work or overlay work with 4 cm depth often invited cracks in the pavement next winter.

In order to prevent the reflection cracks, overlay method was carried out changing the thicknesses of asphalt mixtures or burying iron mesh to resist the movement of the pavement.

4-1-1. Experimental Work

The following two sites, where there were many cracks with the distance less than 10 m, were selected for the experimental work in the two national roads with heavy traffic volumes.

(1) National Road Route 38: Karikachi, Minami-Furano (1978 construction)

(2) National Road Route 39: Aibetsu, Toyosato (1979 construction)

Before the experimental work the crack distributions of the existing pavements were investigated as shown in Photo 4. These areas are also in the severely cold regions.

4-1-2. Outline of Overlay Repair

Materials used for the mixtures, work specifications including the standards of the buried wire or iron mesh, etc. are shown in Table 5. Plan views and cross sections of the overlays with the reinforcements are shown in Figure 8.

The depth of the reinforcing iron mesh were determined by the number of existing cracks and economical costs. Photo 5 shows the overlay work with the installation of reinforcing iron mesh at the Aibetsu Site.

4-1-3. Preventive Effects for Reflective Crackings

At the Karikachi Site, Minami-Furano, the first appearance of cracks was in 2 years in a thin overlay without wire mesh, the second appearance in the same type overlay with wire mesh in 3 years, the last appearance was in 4 years in a thick overlay with and without wire mesh.

When two overlays of 8 cm thick were compared, the one with wire mesh showed a dominant effectiveness. But with two overlays of 13 cm thick, no differences were shown. In Block No. 4, the wire mesh was buried at the depth of 4 cm. But due to the unstability of the wire mesh, it caused, in part, map cracks in the pavement to dislodge the mixtures leading to the exposure of the wire mesh. The result of this experiment showed that the overlays with greater thickness were effective.

On the other hand, at Toyosato Site, Aibetsu, the conventional overlay of 8 cm thick without mesh showed cracks in 3 years, but overlay of 13 cm thick with and without mesh has shown no cracks. This means the thickness dominated the prevention of cracks.

The appearances of reflection cracks are largely dependent on the local weather conditions and distribution of cracks on the existing pavements. Hence, the situations are different from region to region. In view of this, there seems to be no all-round prevention method available at present.

However, based on the results of the two experimental projects, it can be summarized as follows:

(1) Cracks may be appeared in 2 or 3 years when the thickness of overlay mixtures is 8 cm.

(2) With crimp wire mesh, no definite effectiveness can be expected because of creating many problems under overlay work.

(3) Overlay with a greater thickness can be effective.

4-2. Filling Reparation of Cracks

Even if cracks are appearing at long distances, reparation should be attempted immediately to prevent further progress. As investigations have revealed, cracks repeat movements with the contraction and expansion in the pavement to enlarge their opening with the time. Present repairing practice

on cracks are sealing cracks with heated straight asphalt, surface treatment by grinding down with hot iron, filling in with asphalt and mixtures for larger cracks. However, none of the methods seems to last more than one year. Several repairing methods specialized in coping with cracks were proposed and discussed.

4-2-1. Materials and Repairing Methods

As shown in Table 6, four test sections, each having the similar types of cracks, were selected for experimental reparations. In order to resist the movement of cracks and traffic impact with studs, various kinds of asphalt materials were used, as shown in Table 6.

The applied methods were to attach tape type material over the cracks, fill in cracks or use both combined methods

4-2-2. Results of Experiments

Cracks of Block No. 1 and No. 3 were repaired after cleaning the cracks in 1980. The tape type material shown in Table 6, were scraped off by snow-removal equipment and were ripped open as cracks moved.

Where cracks were filled in with material by a gun tool after being compressed air cleaned as shown in Photo 7, the applied materials were peeled off due to shallow penetrations of the materials owing to narrow openings. Sheets were used for combination work, but were found to be vulnerable to moisture, finally to peel off.

Based on these results, cracks of Block No. 4, were reshaped into V grooves with concrete cutter in 1982 as shown in Photo 8. Primer coating was applied on both sides prior to fill in with the material. The adhesion of primer on both sides was not satisfactory, leaving openings. Wider opening cracks may have invited greater traffic impact or the primer was not adequate in its quality for the studded tires of traffic vehicles. Sheets were used partially, but they came off easily.

The filling materials remained flexible with fair performance. Further study and effort should be made to produce more effective primer as well as better-holding groove shapes.

Conclusions

The field observations of crack openings and repairing methods to prevent the thermal cracks in asphalt pavements are summarized as follows.

(1) With the use of buried jointmeters, continuous yearly movement of crack openings were measured in detail.

(2) The movements in the crack widths are not constant but they are sensitive to the age of pavement, existing crack distributions and cold temperatures. With a pavement as old as 6 years after the construction, the annual changes in the crack widths are as much as 4 - 10 mm. During several cold seasons, the movement of the crack openings becomes as great as 2 mm a day to reach the maximum opening as large as 20 mm. Based on these variations, the linear expansion coefficient of asphalt pavements is assumed to be approximately $1.5 - 2 \times 10^{-5}$/°C.

(3) It was found that cracks were suddenly expanded during cold period.

(4) With the temperature of asphalt pavements reaching 20 °C or more, it was found that the movement of the crack would be small due to the stress-relaxation of asphalt pavements. This fact causes not to return the original width. This phenomenon is assumed to make the cracks wider with the time. The annual amount of the openings was as much as 1 - 2 mm with an old pavement and 0.2 - 0.5 mm with a new pavement.

(5) In order to prevent the occurrence of cracks, it was found that the use of crushed rock as subbase course material was effective to build up restraining force with asphalt mixtures. Also the use of high penetration asphalt is effective for the minimizing the occurrence of cracks.

(6) As for the prevention of reflection crackings by the overlay method, key point is the thickness of the overlay mixtures. With a thickness smaller than 8 cm, the overlay introduced cracks in 2 or 3 years.

(7) Reparation of cracks is a difficult work because the crack openings get enlarged with the time. When a filling-in method is considered, it is preferable to reshape the crack opening wider to insure the effective penetration of materials. It is felt necessary that more studies should be given to finding effective shapes of grooves, cleaning method and more adhesive materials.

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"On Traverse Cracks in Asphalt Pavements". The 33rd Technology and Research Conference of the Construction Ministry, October 1979.

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Table 1. Pavement Structure of Crack Measurement Sites

Construction year	1971	1977
Sites	National Road Route 242 Nishi-Issen, Asyoro	National Road Route 275 Uenbetsu, Horokanai
Surface course	Fine graded gapped asphalt concrete (3 cm)	
Binder course	Coarse graded asphalt concrete (4 cm)	
Base course	Asphalt stabilized mixtures (5 cm)	
Subbase course	Pit-run gravel Max. 40 mm (40 cm)	Pit-run gravel Max. 40 mm (25 cm)
Frost preventive layer	Volcanic ash (50 cm)	Pit-run gravel Max. 80 mm (65 cm)
Subgrade soil	Corroded soil (Fibered)	Clayey soil (Gravel mixed)

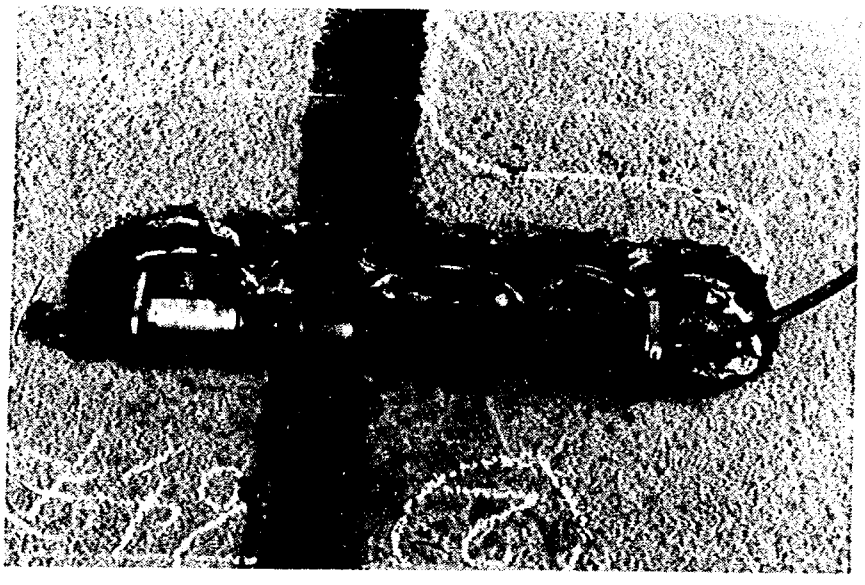


Photo 1. Jointmeters Buried in the Surface Course

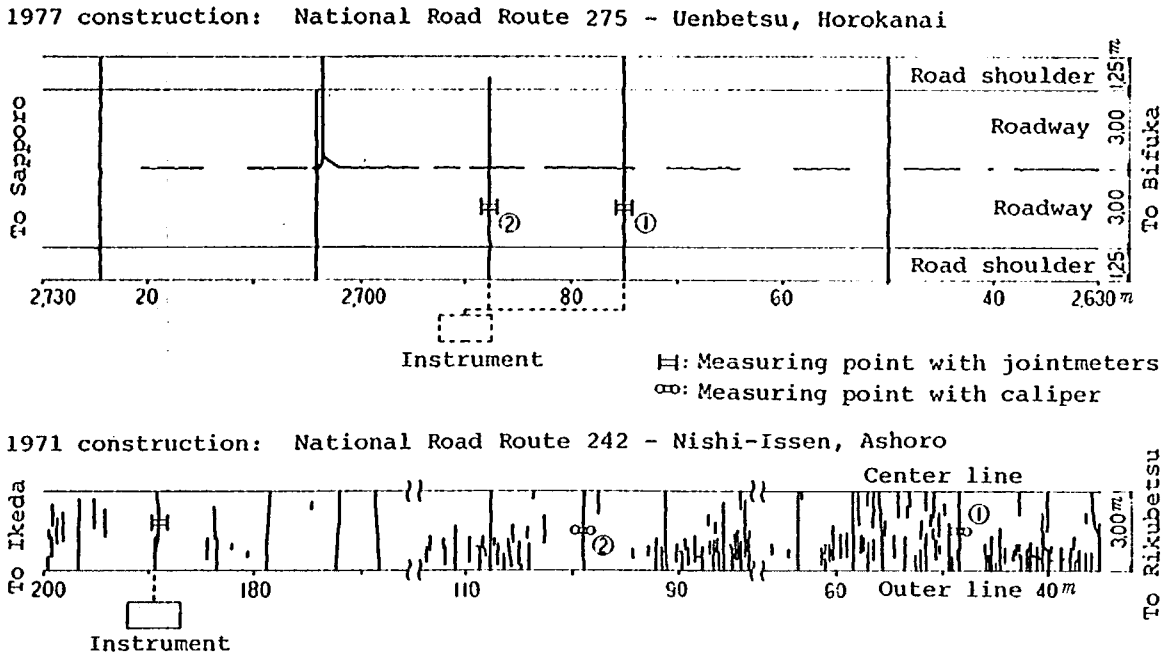


Figure 1. Measurements of Crack Widths

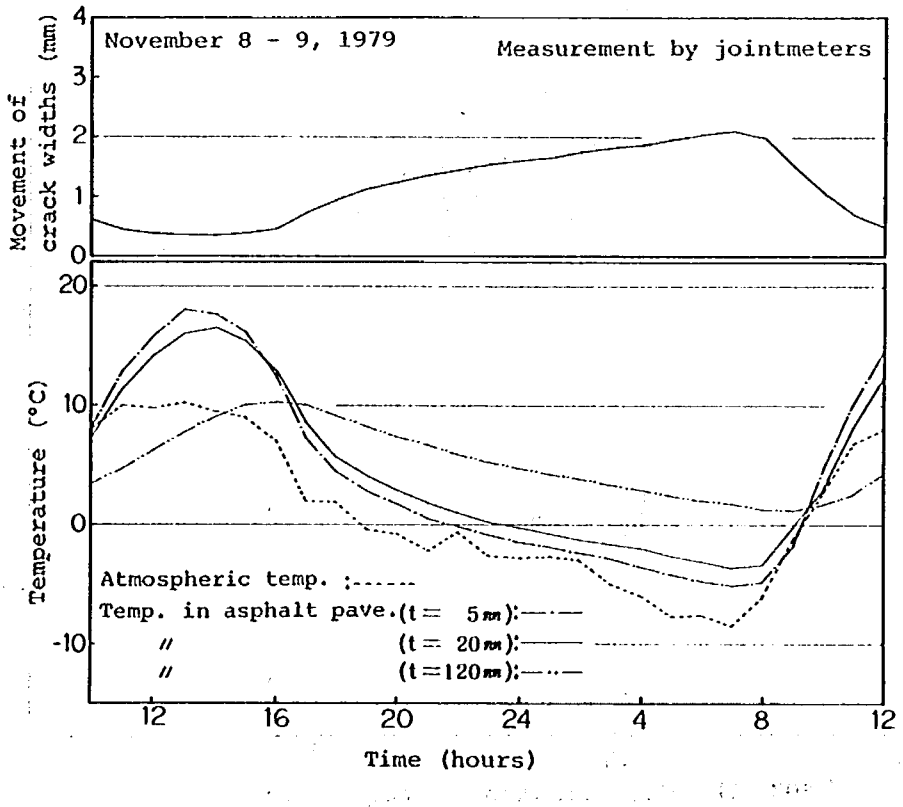


Figure 2. Changes in Crack Widths of Asphalt Pavements (1971 construction: National Road Route 242, Nishi-Issen, Ashoro)

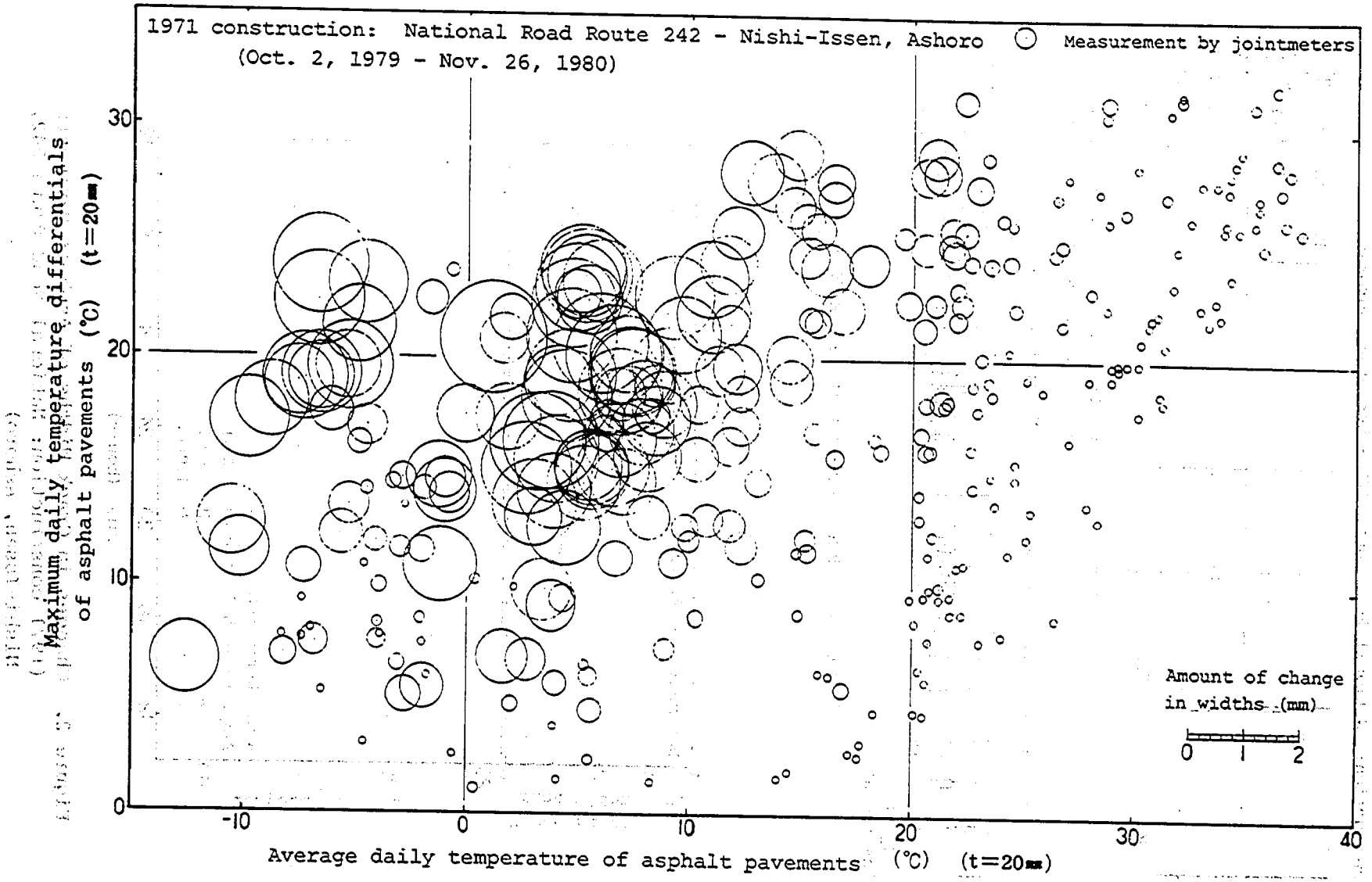


Figure 3. Relations Between Average Daily Temperature and Maximum Daily Temperature Differentials of Asphalt Pavements

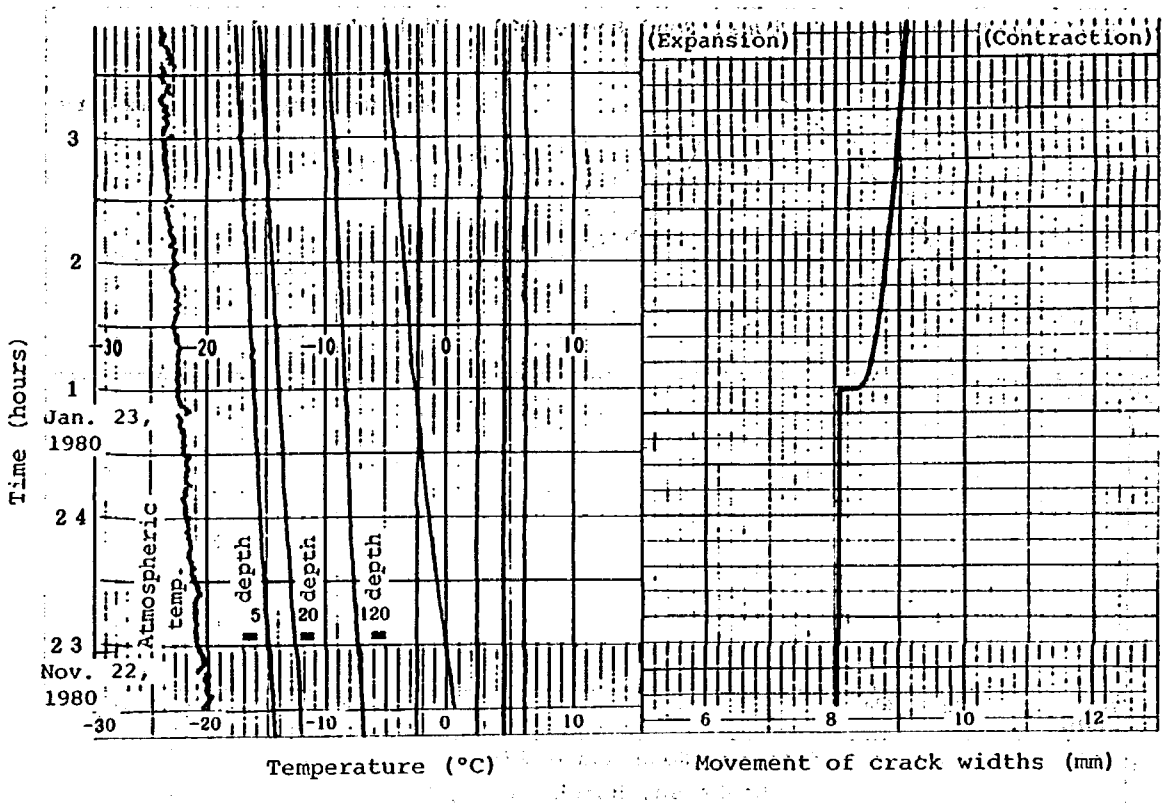


Figure 4. Sudden Expansion of Crack Widths

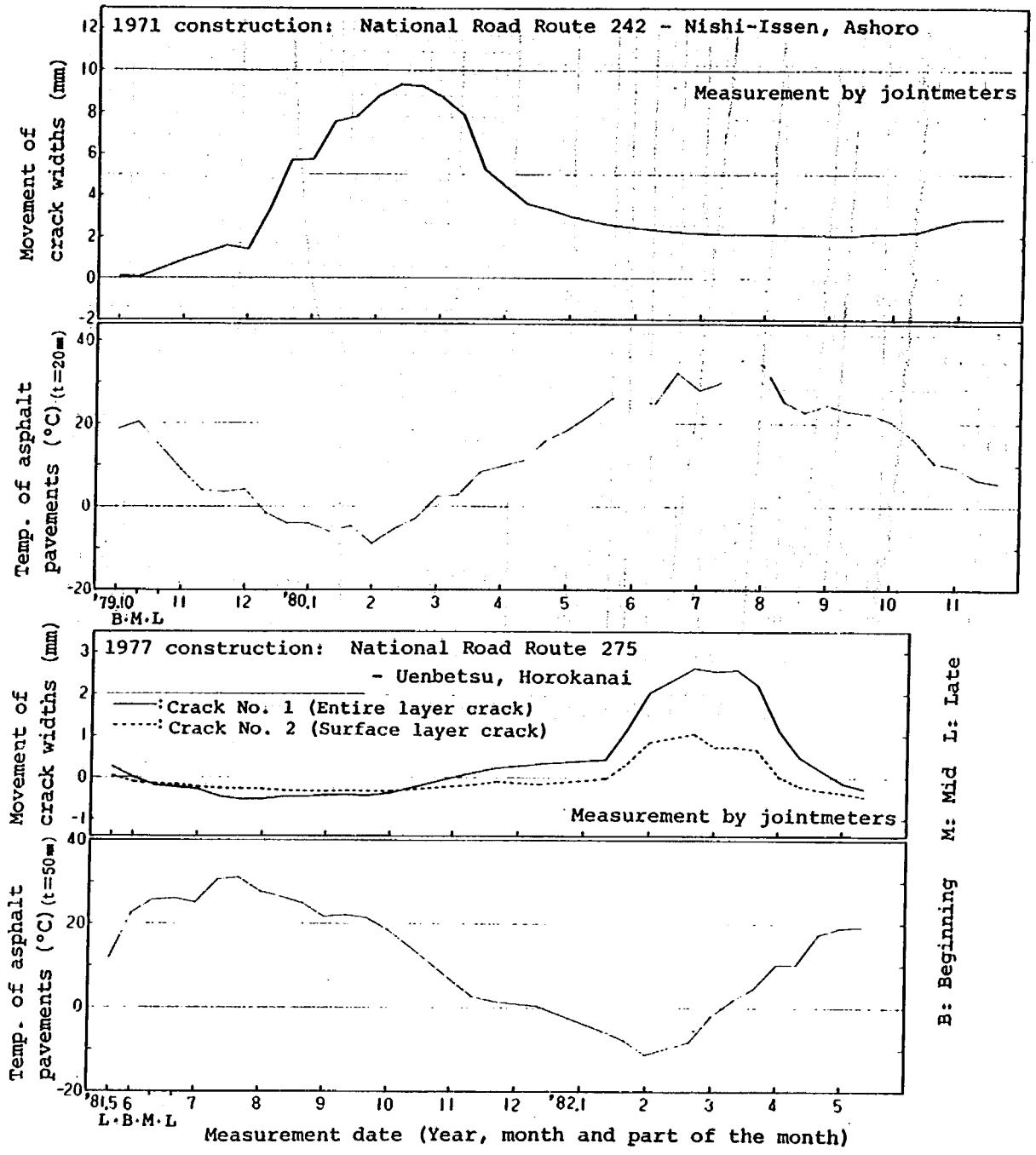
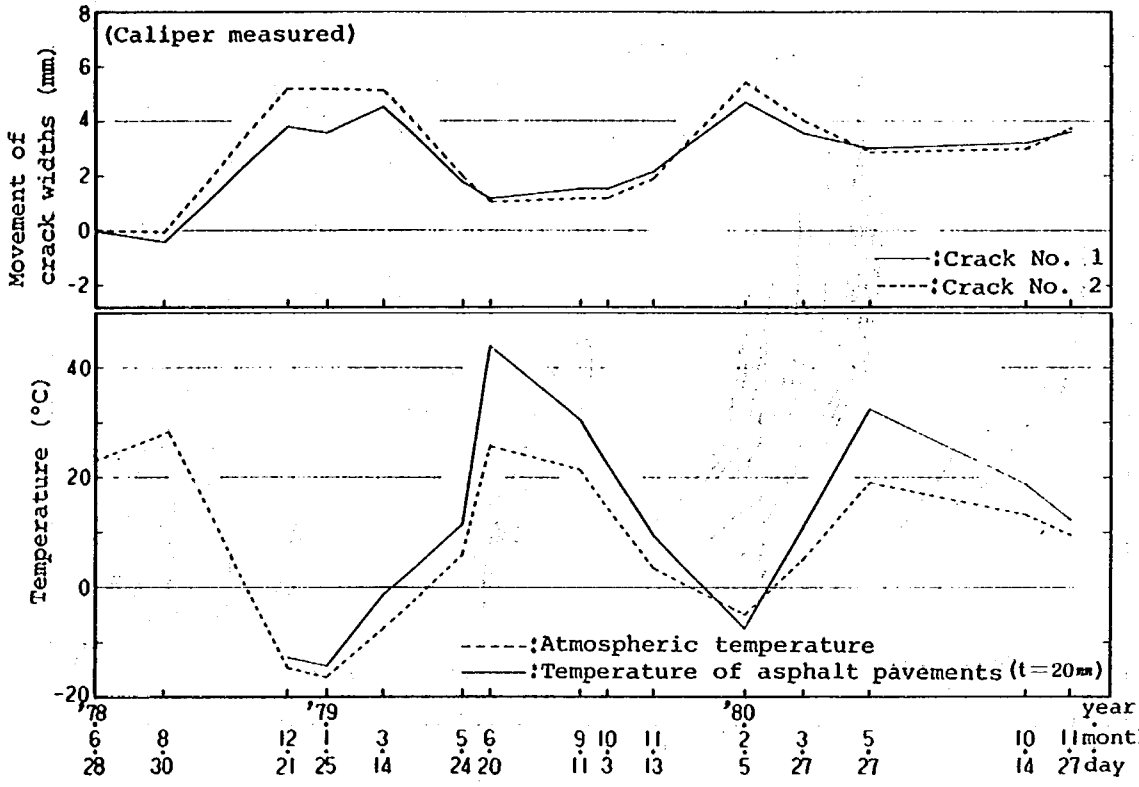
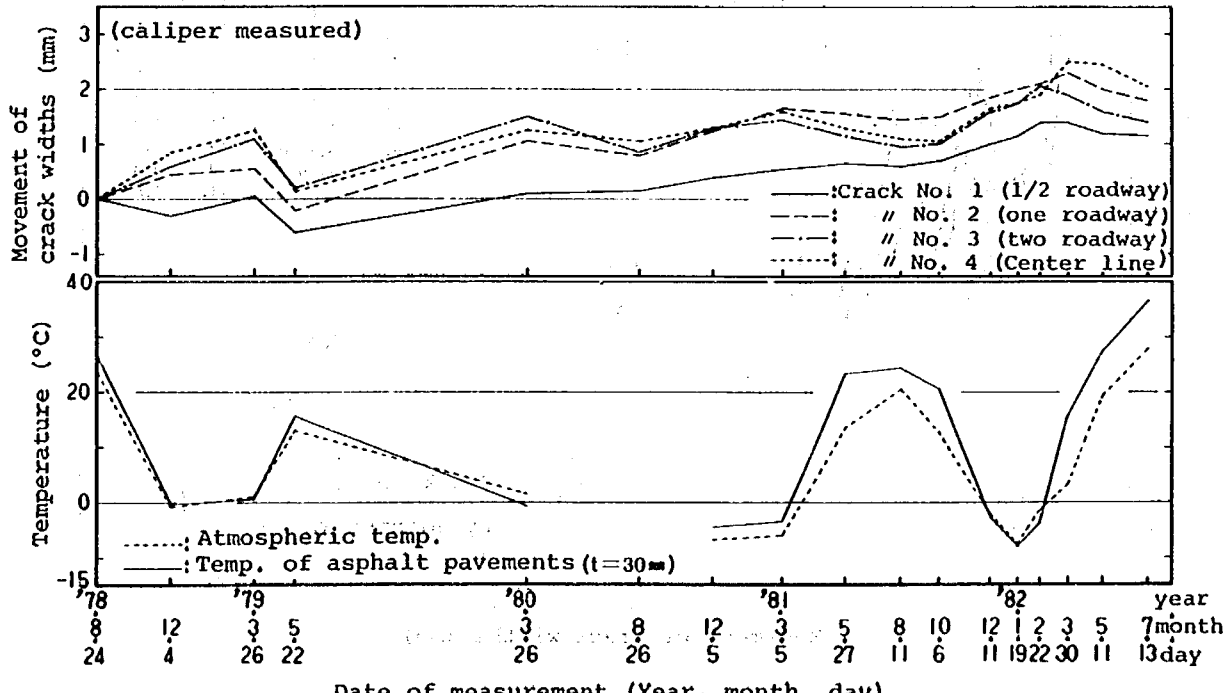


Figure 5. Annual Movement of Crack Widths (ten day average value)

1971 construction: National Road Route 242 - Nishi-Issen, Ashoro



1977 construction: National Road Route 275 - Uenbetsu, Horokanai



Date of measurement (Year, month, day)

Figure 7. Annual Changes in Crack Widths

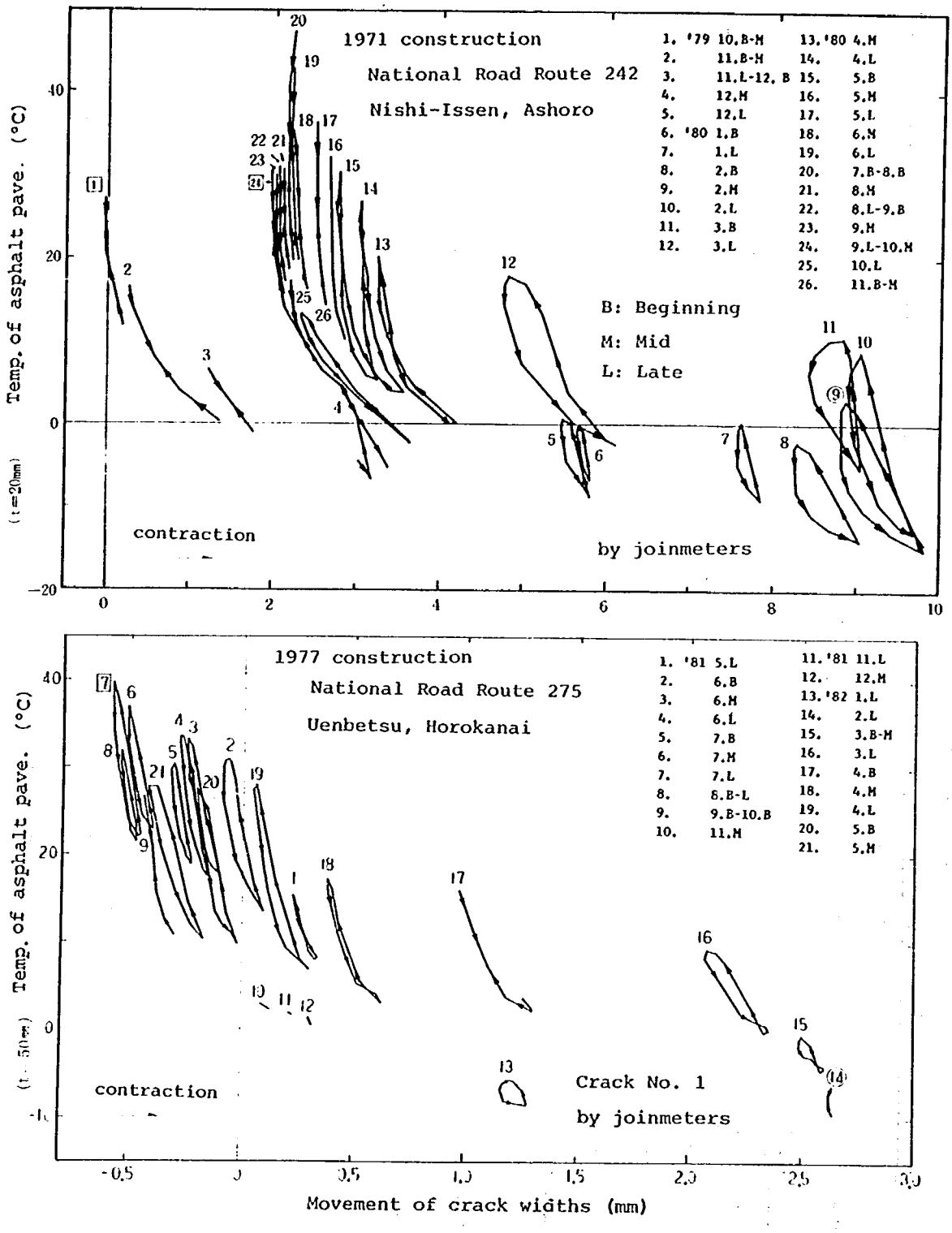


Figure 6. Annual Movement of Crack Widths
(Daily change in ten day average)

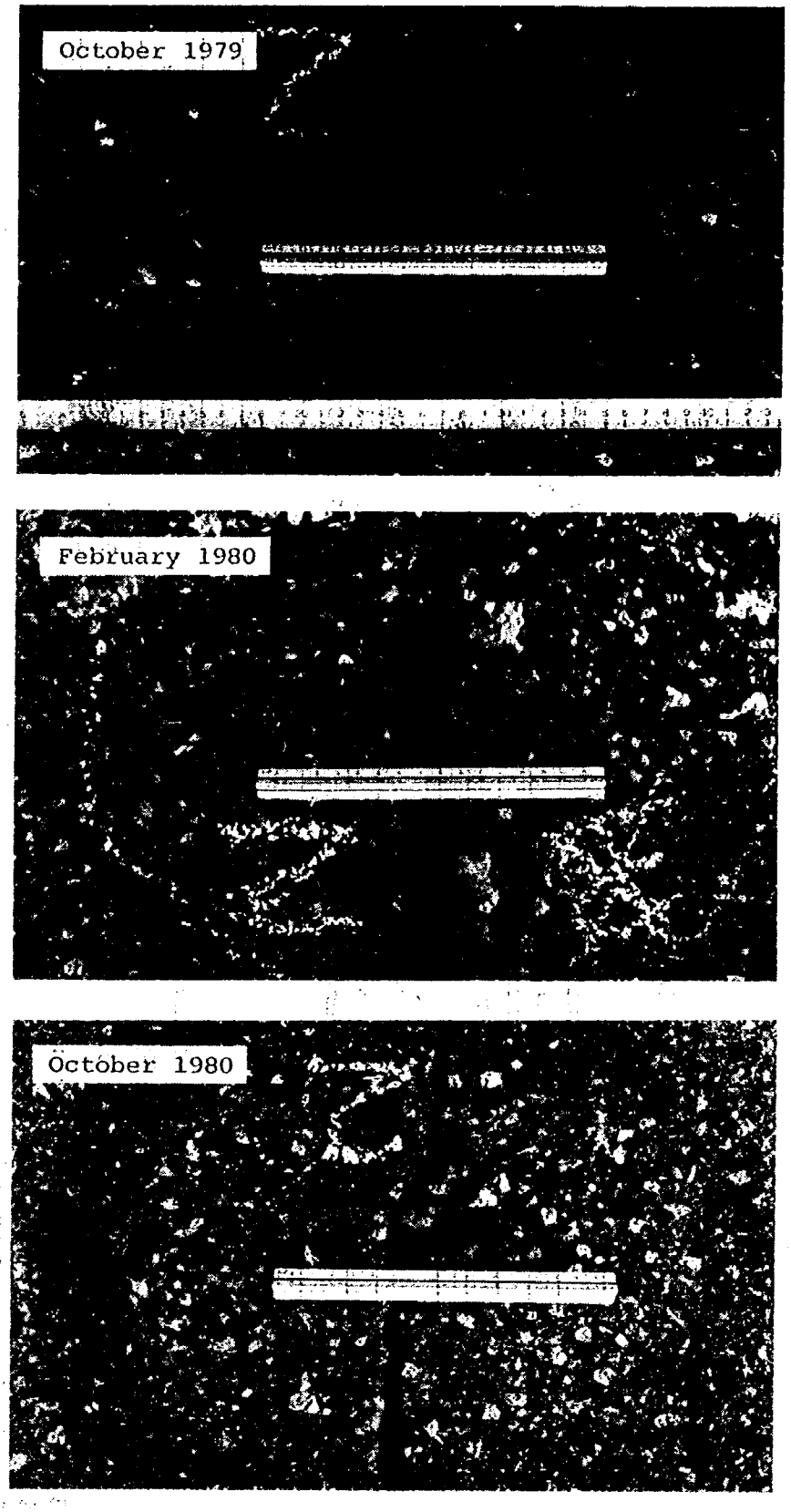


Photo 2. Changes in Crack Widths (1971 construction: National Road Route 242, Nishi-Issen, Ashoro)

Table 2. Outline of Test Pavements (1978 construction: National Road Route 275, Uenbetsu, Horokanai)

Year	Frost index (°C days)	Construction Section	Section A				Section B	
		Block No.	1	2	3	4	3	4
		(Curved portions) Extension (m)	(90) 150	(150) 150	(32) 227	(150) 997	(130) 150	(368) 722
		Properties of materials and mixtures						
		Surface layer mixture	Fine graded gapped asphalt concrete					
		Quantity of asphalt (%)	7.0 (F/A = 1.7)					
		Asphalt penetration	93		93		88	
		Asph. softening point (°C)	46		46		45	
		Asphalt P-I	-0.7		-0.7		-1.2	
		Moisture absorption by aggregates (%)	1.72 (Tooma)			2.62 (Wassamu)	1.55 (Tooma)	2.38 (Wassamu)
		Total thickness, mixture (cm)	12.9	12.9	12.9	12.8	12.7	13.0
		Surface layer compaction (%)	99.2	98.8	99.2	99.1	99.1	99.2
		Elements						
		Rubber mixing	St	Ru	St		St	
		Moisture absorption by aggregates	Little			Large	Little	Large
		Subbase material	Crushed rock	Pit-run gravel			Pit-run gravel	
'78	899	Number of cracks	0	0	0	0	0	0
'79	994	( ) value is number of	0	0	0	0	0	0
'80	1,040	curved lines.	0	1(1)	1(1)	4(3)	0	0
'81	572	As of June 1983	0	16(16)	2(1)	8(6)	0	0
'82	892		0	17(17)	2(1)	9(6)	0	0

1) Mixed amount of rubber is 4%.
2) Number of cracks is accumulated figure of cracks until the following spring.

Table 3. Outline of Test Pavements (1979 construction: National Road Route 275, Moshiri, Horokanai)

Year	Frost index (°C days)	Block No.	1	2	3	4	5	6	7	8
		(Curved portion) Extension (m)	(80) 150	(36) 179	(75) 164	208	150	150	150	289
		Properties of materials and mixtures								
		Sidewalk	None				Yes (one side)		Yes (either side)	
		Subbase course materials	Pit-run gravel Max. 40 mm							
		Surface layer mixtures	Dense graded gapped asphalt concrete		Fine graded gapped asphalt concrete		Dense graded gapped asphalt concrete		Fine graded gapped asphalt concrete	
		Moisture absorption by aggregates (%)	1.96							
		Total thickness of mixtures (cm)	12.5	12.4	13.4	12.8	12.9	12.8	13.6	13.2
		Surface layer compaction (%)	98.3	98.5	100.0	100.0	99.5	98.8	99.0	99.2
		Asph. softening point (°C)	48.0				46.5			
		Asphalt P·I	-0.8				-0.6			
		Element								
		Asphalt penetration	73				91			
		Quantity of asphalt (%)	5.5		7.0		5.5		7.0	
		Quantity of filler (F/A)	1.3	1.7	1.3	1.7	1.3	1.7	1.3	1.7
'79	1,062	Number of cracks	0	0	0	0	0	0	0	0
'80	1,107	( ) value is number of	0	0	0	0	0	0	0	0
'81	722	curved lines.	0	0	0	0	0	0	0	0
'82	861	As of June 1983	0	1(1)	1(1)	1	0	0	0	0

1) Number of cracks is accumulated figure of cracks until the following spring.
2) All cracks noted were found around the center markers.
3) Snow remained on the pavement with the depth of 15 cm during winter.

Table 4. Outline of Test Pavements (1981 construction; National Road Route 273, Nukabira Sannosawa, Kami-Shihoro)

Year	Frost index (°C days)	Block No.	1	2	3
		(Curved portion) Extension (m)	(164) 340	(130) 130	(12) 130
		Properties of materials and mixtures			
		Subbase materials	Pit-run gravel Max.40 mm		
		Surface layer mixture	Fine graded gapped asphalt conc.		
		Moisture absorption by aggregates (%)	1.12		
		Type of asphalt	St. 80-100	Modified M 150	Modified M 200
		Asph. softening point (°C)	46.0	39.8	37.1
		Asphalt P · I	-0.8	-0.9	-0.6
		Quantity of asphalt(%)	7.3	7.0	
		Quantity of filler(F/A)	1.71		
		Total thickness of mixture (cm)	13.5	12.9	13.3
		Surface layer compaction (%)	100.6	99.7	100.0
		Element			
		Asphalt penetration	91	168	236
'81	940	Number of cracks	0	0	0
'82	970	As of September 1983	0	0	0

1) Block No. 1 is the total of the starting and terminal point section.
2) Cracks appeared in the boundary of cutting and filling work were not counted.
3) Numerous hairline type cracks were found in the surface of block boundary lines.

Photo 3. Map Crackings due to Decrease in Bearing Capacity

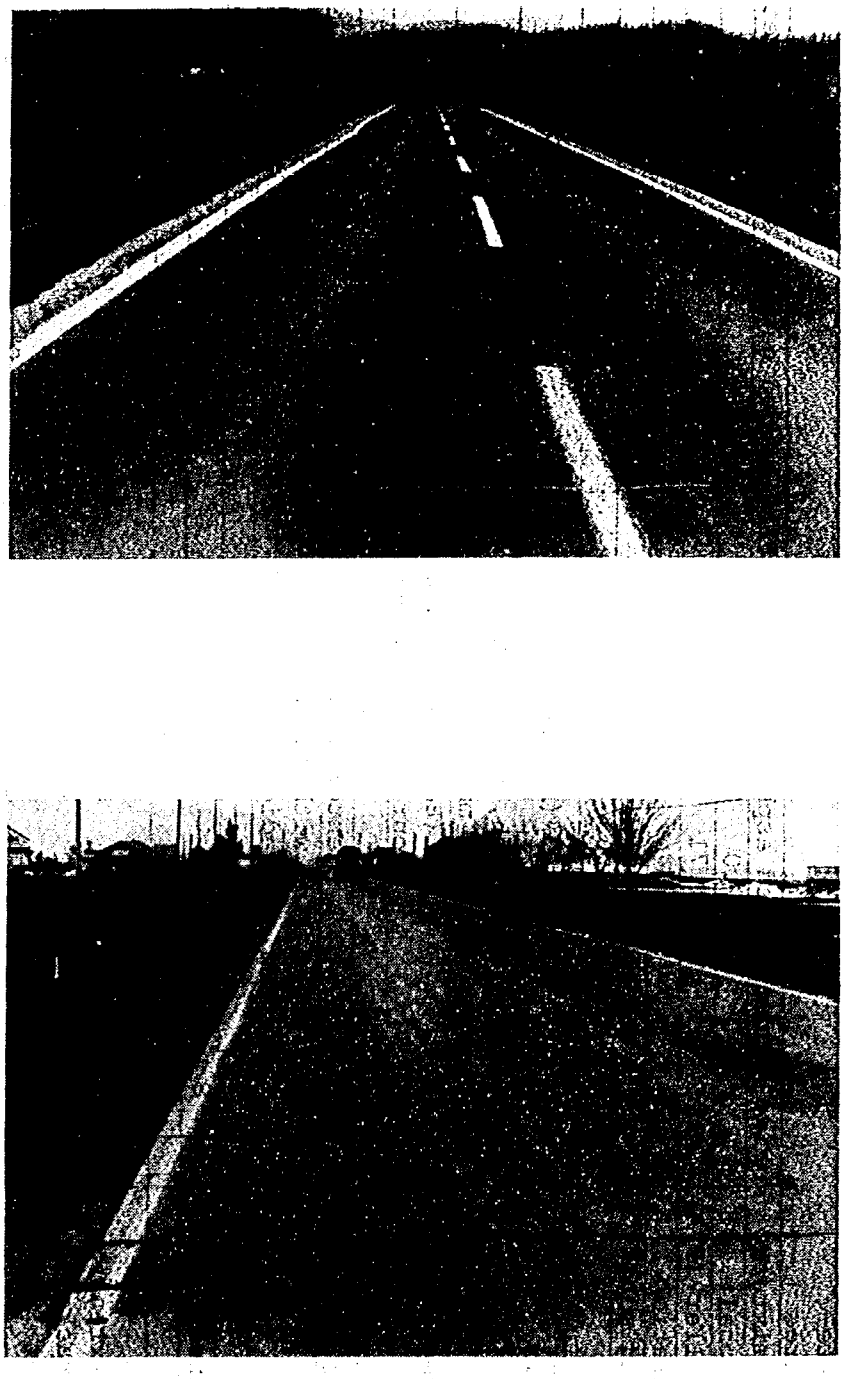


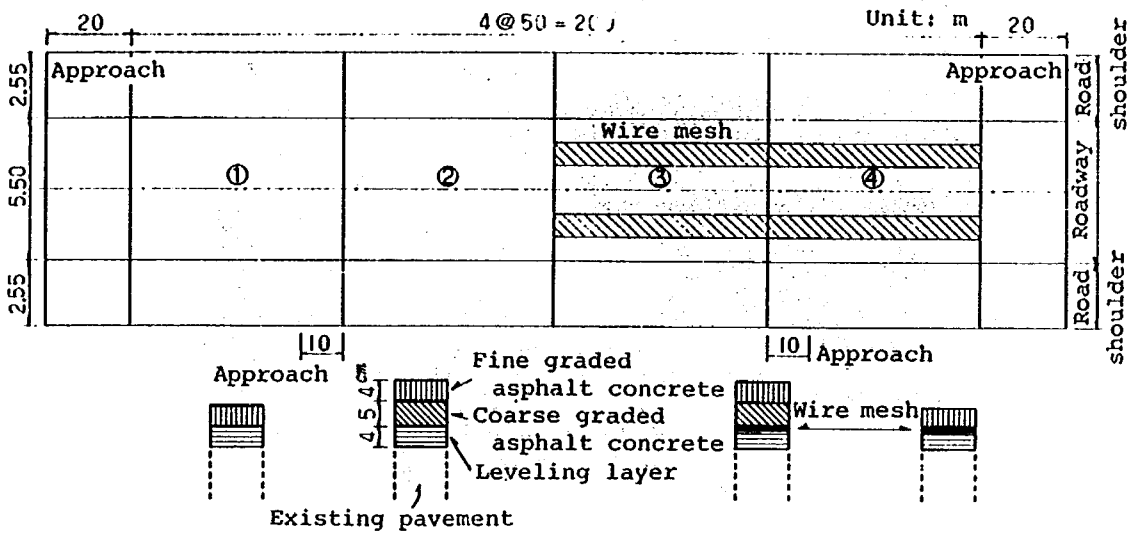
Photo 4. Crack Distribution in the Existing Pavement Before the Overlay Repair
(Above: National Road Route 38, Karikachi, Minami-Furano.
Below: National Road Route 39, Toyosato, Aibetsu.)

Table 5. Outline of Overlay Work

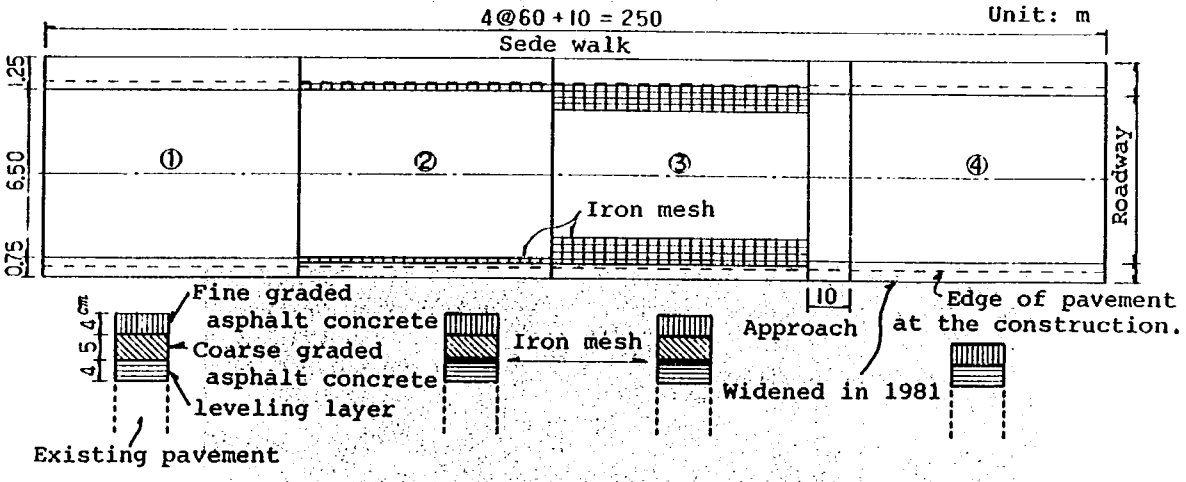
Year	Name of Site	1978 construction: NRR 38, Karikachi, Minami-Furano				1979 construction: NRR 39, Toyosato, Aibetsu			
	Block No.	1	2	3	4	1	2	3	4
	Extension (m)	50	50	50	50	60	60	60	60
	Properties of materials and mixtures								
	Surface layer mixtures	Fine graded gapped asphalt concrete				Fine graded gapped asphalt concrete			
	Quantity of Asphalt (%)	7.0				7.3			
	Quantity of filler (F/A)	1.71				1.7			
	Asphalt penetration	89				85			
	Asph. softening pont (°C)	46.5				44.5			
	Asphalt P.I	-0.7				-1.4			
	Moisture absorption by aggregates (%)	2.13 (Furano)				1.99 (Ranru)			
	Wire or iron mesh — Standard	Mesh: Crimp wire mesh, zink coated 50 mm mesh, dia. 2.5 mm				Welded iron bar mesh, mesh 25 x 50 cm, dia. 6 mm			
	Wire or iron mesh — Depth (cm)			9 (Base course)	4 (Surface course)		9 (Base course)		
	Wire or iron mesh — Width (cm)			91			25	100	
	Wire or iron mesh — Location			Center of roadway			Edge of roadway		
	Other	Straight line with slight longitudial gradient				Flat straight line, widened in 1981 one side sidewalk was added			
	Elements								
	Overlay thickness (cm)	8	13		8	13			8
	Screens	-		o		-	o		-
'78	Number of cracks	0	0	0	0	-	-	-	-
'79	( ) value is extension	1	0	0	0	0	0	0	0
'80	of existing cracks.	1(1)	0	0	2	0	0	0	0
'81	As of September 1983	1	1	3	4(2)	0	0	0	6
'82		-	-	-	-	0	0	0	6

1) Number of cracks is accumulated figure of the cracks until the following spring. Number of extended cracks is for the current year.
2) All the cracks were reflection cracks.

1978 constrution: National Road Route 38 - Karikachi, Minami-Furano



1979 construction: National Road Route 39 - Toyosato, Aibetsu



*Back and front of work section paved at the same time in 4.

Figure 8. Test Section with Overlay Reparation

Photo 5. Overlay Repairment with Iron Mesh
(Width of iron bar screen: 100 cm)

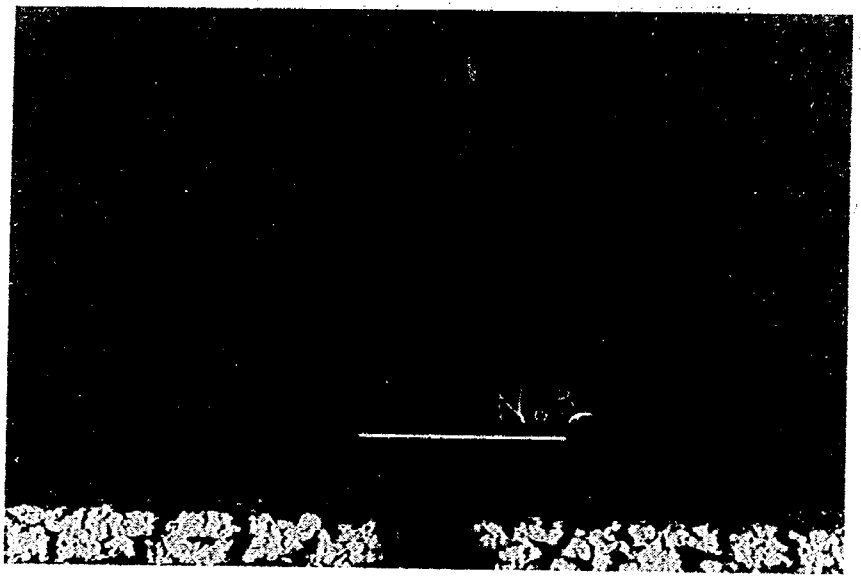


Photo 6. Repairment with Sealing Material Processed to Tape Type

Table 6. Repairing Method of Cracks

No.	1	2 and 3	4
Work year	1980	1980	1982
Name of site	National Road Route 275 Kami-Horokanai, Horokanai	National Road Route 274 Minami-Naganuma, Naganuma National Road Route 275 Uenbetsu, Horokanai	National Route Road 274 Minami-Naganuma, Naganuma
Type	2	6/3	4
Properties of materials	·Asphaltic polyurethane ·Hydraulic urethane Both mixed with main ingredient with hardener to be in a fluid state at normal temperature.	·Adhesive sealing materials under normal temperature. Composed of rubber plus asphalt master batch with bond additive. Properties: Penetration 109 Softening point 120 °C P · I +10.6	N·No. 2 and 3 sealing materials mixed with extenders (filler). T·RC bond (asphaltic) ·CAM filler (cement asphalt-mortar)
Method	*Crack at the time of repairment ·Repairing work as they were.	·Repairing work as they were.	·Reshape into V shape with the top width of 26 mm and depth of 30 mm.
	·Fill in at normal temperature.	·Sealing material was re-processed to tape, mat, or rope to be sealed down. ·Heated application with above method combined.	N·Heated fill in T·Combination of heated application at normal temperature with sheets.

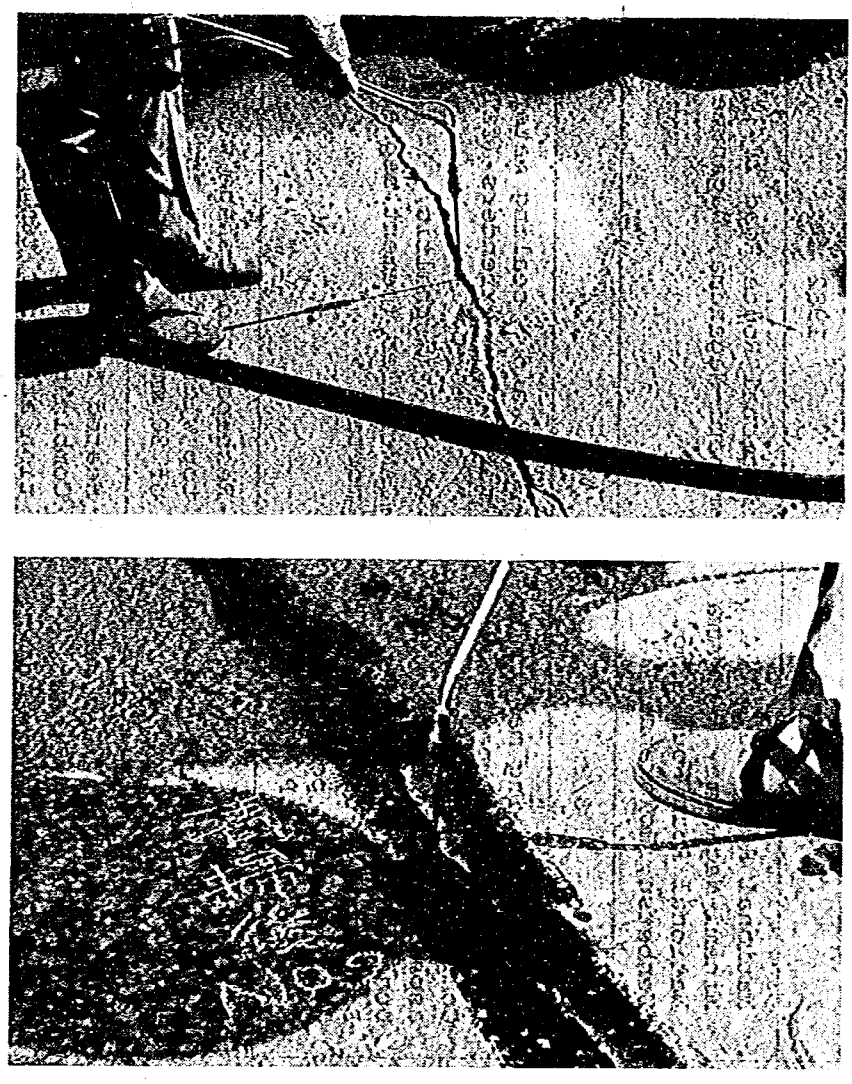


Photo 7. Compressed Air Cleaning and Sealing Work

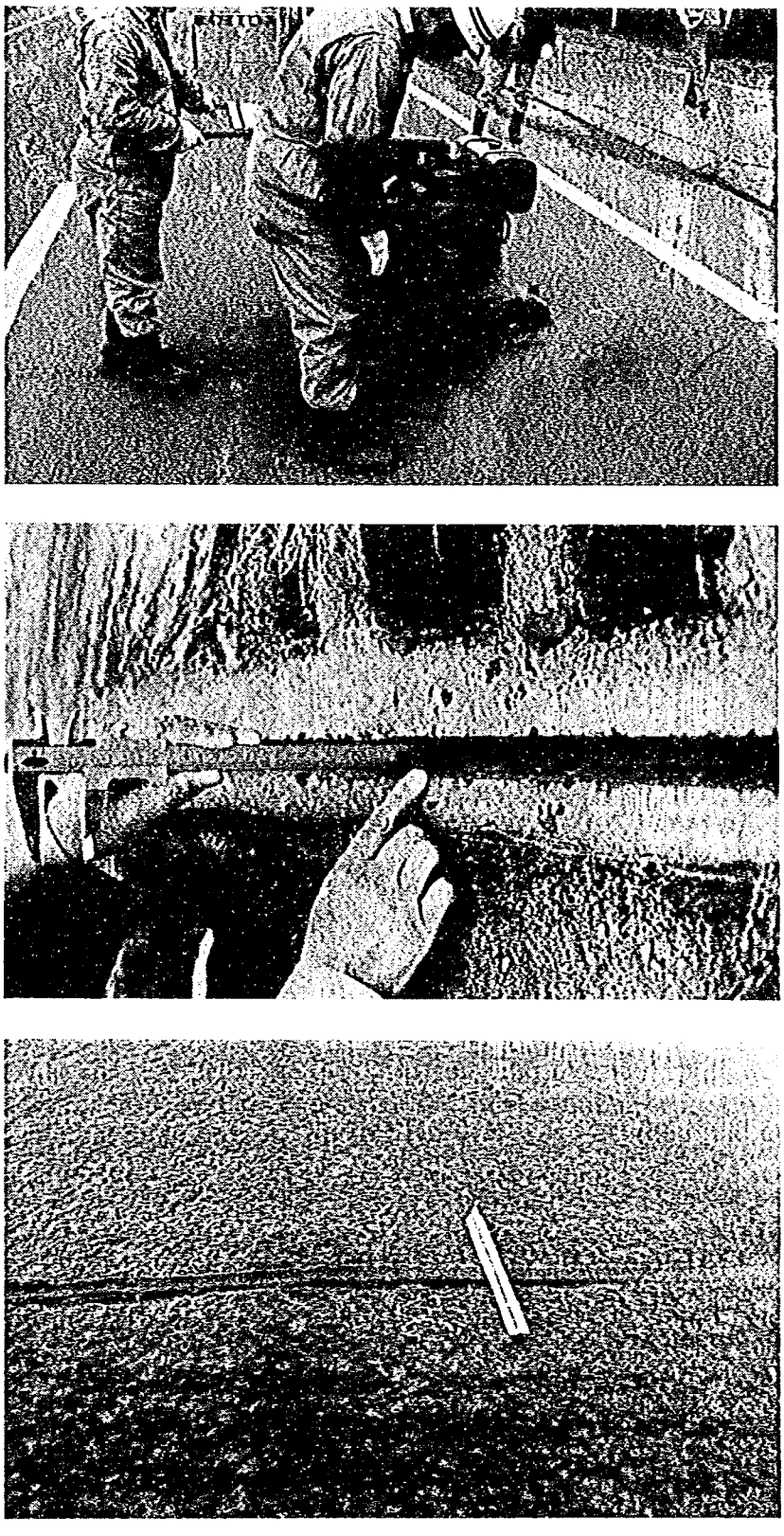


Photo 8. Sealing Work After Cracks Being Reshaped into V

Longitudinal Surface Cracking of Flexible Pavement

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ABSTRACT

Cracking in flexible pavements has been considered to occur at the bottom of asphalt concrete layer by fatigue as the result of repeated flexural stresses under the traffic loads. However, in Japan, many longitudinal cracks starting from the surface of wearing course have been observed along the wheel pass. This type of cracking is basically different from the prevailing concept of the cracking in flexible pavements,so that it is regarded as a problem in structural and mixture design of flexible pavements in Japan. Though this phenomenon is not particular to pavements in cold regions, it brings forward a problem to the rational structural design method in recent years.
This report presents the general state of this type of cracking and the result of calculations by the layered elastic theory including Finite Element Methods. The result of calculation showed large tensile strain occurred on the surface of wearing course when pavement surface temperature is high. The cracking seems to be caused by this large tensile strain and it should be pointed out that the hardening of asphalt in wearing course increases the risk of cracking.

LONGITUDINAL SURFACE CRACKING OF FLEXIBLE PAVEMENTS

by

Saburo Matsuno

Sato Road Company Ltd.

Tatsuo Nishizawa

Kanazawa University

1. Introduction

Longitudinal Surface Cracking (LSC) is longitudinal cracking developping from the top of wearing course along the wheel paths of flexible pavements. According to the current rational structural design method, cracking in flexible pavements is considered to occur at the bottom of asphalt concrete layer by fatigue as the result of repeated stresses under the traffic loads, if the settlement of base and subgrade caused by permanent deformation is not large. Therefore it must be peculiar phenomena that LSC are frequently observed in flexible pavements in Japan. Japan Road Congress already pointed out that LSC is a new problem to be thoroughly investigated.

In heavily trafficed highway rutting of asphalt mixture by plastic deformation is the most serious problem, but LSC is likely to occur in the pavements where rutting is scarcely observed. Therefore LSC and rutting are contrary terms in flexible pavements.

In this report several examples of LSC and possible mechanism for LSC are presented. If LSC is caused by structural weakness of asphalt pavement, the current rational structural design of asphalt pavement should be reviewed. From this reason authors consider that LSC is a very important phenomenon in the design of flexible pavements.

2. Explanation of Phenomenon

Photos 1,2 show LSC in a national highway and a city street. Longitudinal cracks occurred in wheel paths. Photo 3 shows a core sample taken from cracking area of Photo 1. A crack developed only in 5cm thick wearing course of 10cm thick asphlt layer.

There are several patterns in type and location of occurrence of LSC. A relatively linear type and a zigzag type as shown in Photo 2 are both observed in LSC. As for location of occurrence, two patterns,1) in the center of wheel path, and 2) approximately one tire width outside the center of wheel path, are observed, and some of both occur in outer wheel path in the lane and

others in inner wheel path. In some cases it distributes uniformly in the construction area, but in other cases partially. Generally it occures near-side lane first, but in some cases it occures only in passing lane. Such differences in pattern and location of occurrence can not be explained with our present knowledge.

Photo 4 shows LSC in the asphalt pavement on concrete bridge decks, which indicates that LSC does occur on the asphalt pavement with such sufficient bearing capacity as concrete decks.

3. Condition of Occurrence of LSC

Analysis data of wearing course mixtures shown in Photos 1 and 3 are summarized in Table 1. Asphalt existing in the mixture of cracked zones is extremely hardened and segregation of mixture materials is also recognized.

Data of mixture analysis of LSC zones in express highways also showed significant hardening of asphalt, and the limit values for hardening leading to cracking were reported to be 25-35 in penetration and 54-60° C in softening point.[1),2)] Mixtures containing such aged hard asphalt are obviously considered to show small rutting by plastic flow. As seen from these examples, it seems that occurrence of LSC are strongly affected by hardening of asphalt in mixtures.

Traffic volume is not main cause for LSC, because there are many examples of LSC which appeared within a few years after paving. Developement of cracking is in general considered to be related to low bearing capacity of pavement, however, data of deflection survey of LSC zones reported to date, deflection values are almost the same as other zones. Because LSC even occurred in pavements on concrete decks, deflection of pavement is not the cause for LSC.

4. Calculation of Strains on the Surface of Wearing Course

Suppose LSC occurred by fatigue of surface course mixture, there must be large tensile strain in transversal direction on the surface of wearing course by wheel load.

We calculated strains on the surface of weaning course by BISAR program under the following conditions.

1) Structural cross sections of pavements and input data used for calculation are shown in Figure 1 and Table 2 respectively. Asphalt Ⓐ is extremely aged and Ⓑ is ordinary aged.

2) The stiffness of asphalt mixture was obtained from Figure 2. Stiffness values in Figure 2 are slightly higher than that indicated in reference 3). Stiffnesses of asphalt corresponding to each pavement temperature were obtained from Shell Methods.

3) Pavement temperatures actualy measured in Kyushu Region were adopted.[4] Figure 3 shows pavement temperature distribution in depth in fine weather days on January, April, July and October. Average temperatures of upper 2cm and lower 3cm of 5cm wearing course were used. Temperatures at 7.5cm and 17.5cm from the surface were adopted for binder course and hot mix asphalt stabilized base respectively.

4) Standard wheel load 5ton, one tire load 2.5ton was adopted. In this case the radius of circular loaded area is 11.3cm and contact pressure is 6.2kg/cm^2.

Result of calculation are as follows.

1) Transversal surface strains of cross section A measured at 4 p.m. of a fine day in July (surface temperature 51°C) of Case1 and 2 are shown in Figure 4. Large tensile strains are observed just outside of tire edge. This strain is larger in Case 1 than in Case 2. In Case 1 stiffness of the mixture is small.

2) Figure 5 shows the relation between average stiffness values of 5cm thick wearing course and maximum surface tensile strains of cross section A at tire edge. Maximum tensile strains on the surface are well correlated with the stiffness values of surface mixtures.

3) Figure 6 shows the transversal tensile strains on the surface and bottom of each cross section. Transversal surface strains of structural cross sections B and D, both of which have concrete base course, are larger than those of cross sections A and C. Thickness of hot mix asphalt layers does not significantly affect the transversal surface strain.

4) Figure 7 shows distribution of transversal strains in depth at tire edge and at the center of tire. Transversal strain at the edge of tire suddenly changes to tension below the depth of a few millimeters from the surface. It seems that some kind of punch shear stress condition is likely to occur at tire edge at high temperature condition. Nishi already has pointed out by the calculation with FEM, that some tensile strains occured at the surface of wearing course when surface temperature was high.[5] The result of Shell Group also showed calculated and measured values similar to Figure 5.[6]

Figure 8 shows the result of calculation with FEM.[7] Though it is naturally expected with an axisymmetric FEM , Figure 8 shows almost the same result as multielastic layered theory. It is concluded as described above that large tensile strains occur on the surface of wearing course at high temperature.

5. Possible Mechanism for LSC

Photo 5 shows an example of LSC that occured in the front side of overpass bridge, but under the bridge no cracks are observed. Because there is no sunshine under the overpass bridge, it is estimated that temperature of pavement surface does not much exceed air temperature in summer season, and consequently tensile strains are not large in calculation. This example shows LSC is likely to occur at high temperature on summer season.
In Figure 5 the tensile strain is larger in Case 1 with soft asphalt than in Case 2 with hard asphalt. But as already mentioned, empirically speaking, LSC is likely to occur in Case 2 and not in Case 1. When hardening of asphalt took place such as in Case 2, LSC is considered likely to occur because stress relaxation and healing properties of the asphalt mixture are inferior to Case 1, and fatigue by repeated tensile strains is accumulated in the mixture.

6. Future Research

The study of LSC is just started. It is still necessary to investigate LSC on actual highways and to clarify the relation between occurrence of LSC and mixture properties and permanent deformations of asphalt pavements.

Acknowledgements

The authors wish to thank Pavement Section, Public Works Research Institute, Ministry of Construction, for the calculations by BISAR Program. We also wish to thank Dr. S.Nishi, Dr.U.Miura, Mr.H. Yamashita and Nippon Doro Kodan for many helpful suggestions.

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7) Matsuno,S. et al.,"Analysis of the Asphalt Pavement Surface Strains by Finite Element Method" Proceedings of the Meeting, Chubu District Branch of Japan Society of Civil Engineers, Mar.1984. (in Japanese)

Photo-1 Typical Longitudinal Surface Cracking,National Highway

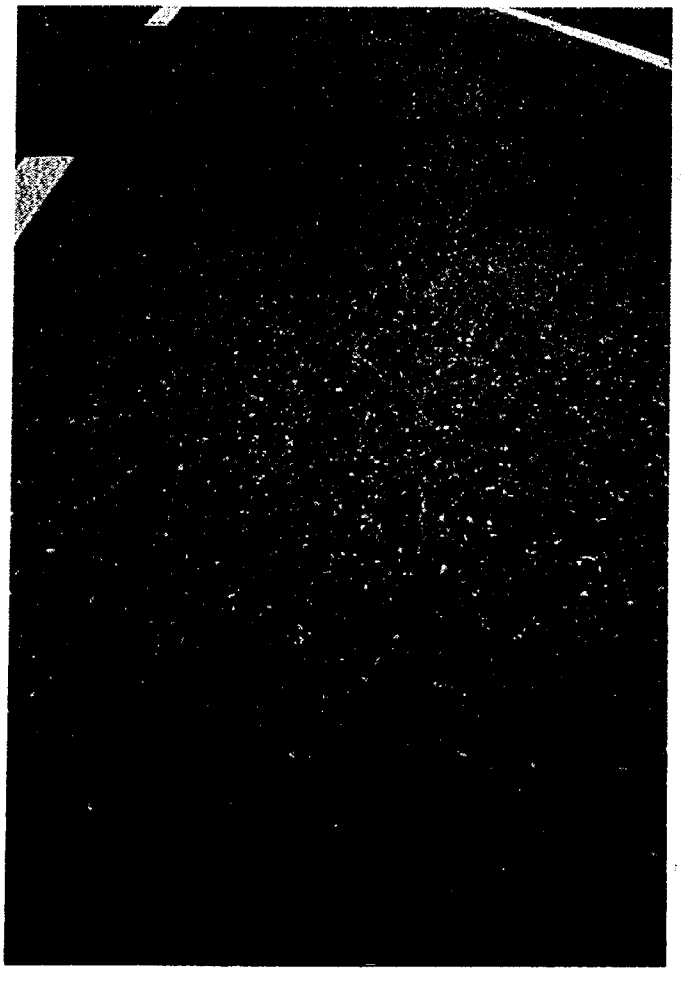


Photo-2 LSC,Street

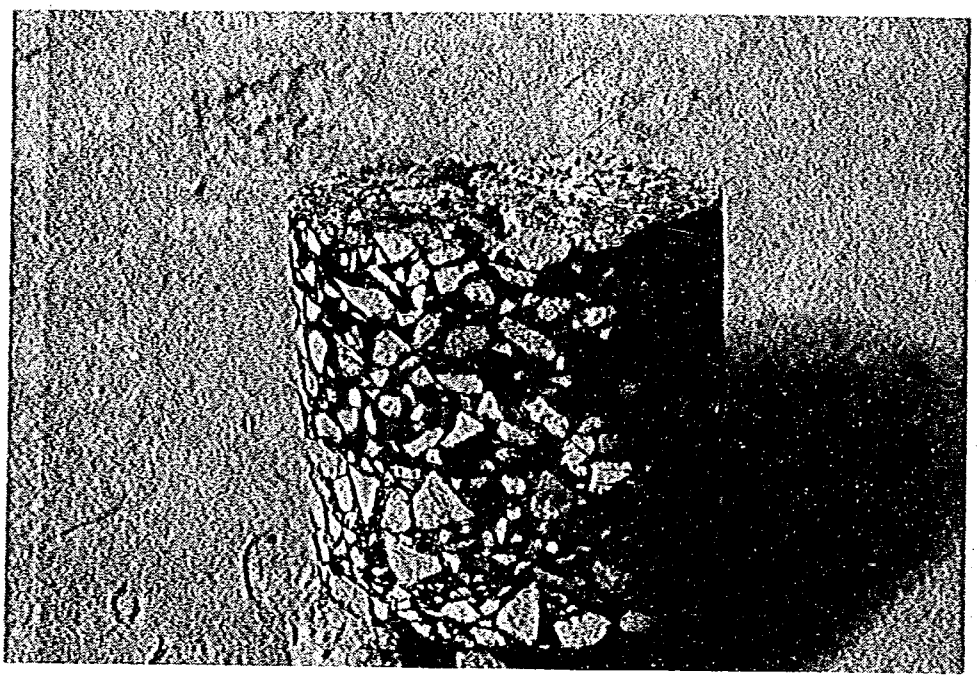


Photo-3 Core Sample of LSC in Photo-1, Crack Generated Only Wearing Course

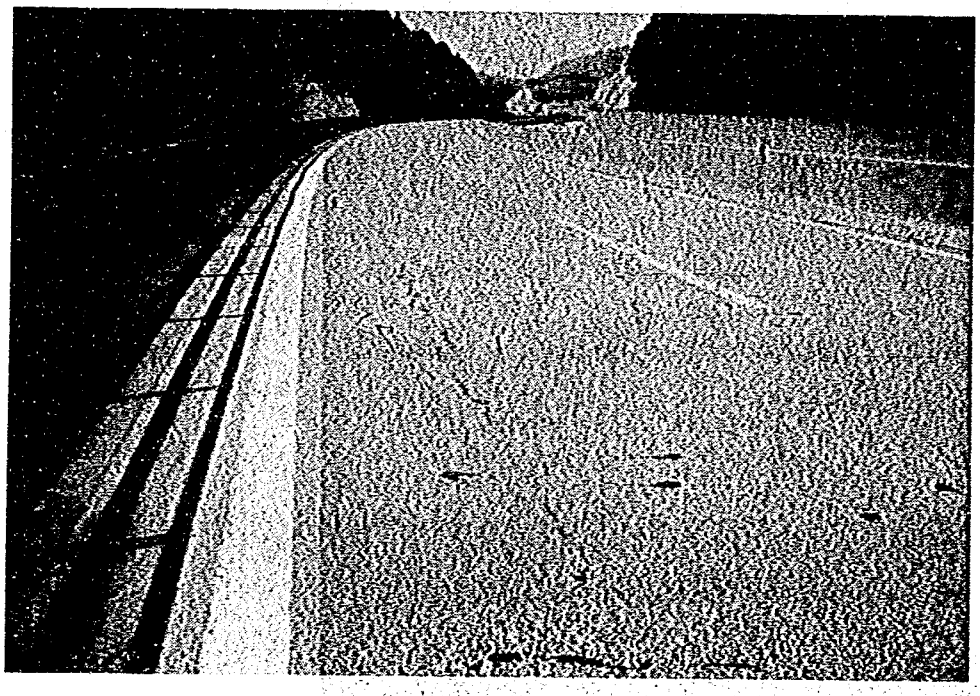


Photo-4 LSC in Asphalt Pavement on Bridge Deck

Photo-5 Cracking on This Side of the Over Pass Bridge, but no Cracking under the Bridge

Table-1
Extraction Test Result, Cracking Area of Photo-1,
Average of 2 Samples

Characteristic		Cracking	no Cracking
Penetration, 25°C, dmm		24	34
Softening Point, R&B, °C		61.5	59.5
Viscosity, 60°C, poise		24,000	13,000
Asphalt Content, %		5.7	6.4
Gradation, % passing,	13 mm	97	99
	2.5 mm	41	49
	0.074 mm	11	12

Table-2
Input Data

Asphalt Mixture	E;Depend on Temp., μ = 0.35
Granular Material	E=2,000 kg/cm^2, μ = 0.35
Concrete	E=350,000 kg/cm^2, μ = 0.25
Subgrade	E=400 kg/cm^2, μ = 0.35

Asphalt, used, Ⓐ; Penetration 25, R&B 65°C
Ⓑ; Penetration 46, R&B 55°C

Cross Section	Layer	Case 1	Case 2	Case 3
A, B	Wearing	Ⓑ	Ⓐ	Ⓐ
	Binder	Ⓑ	Ⓑ	Ⓐ
	Bituminous Base	Ⓑ	Ⓑ	Ⓑ
C, D	Wearing	Ⓑ	Ⓐ	Ⓐ
	Binder	Ⓑ	Ⓑ	Ⓐ

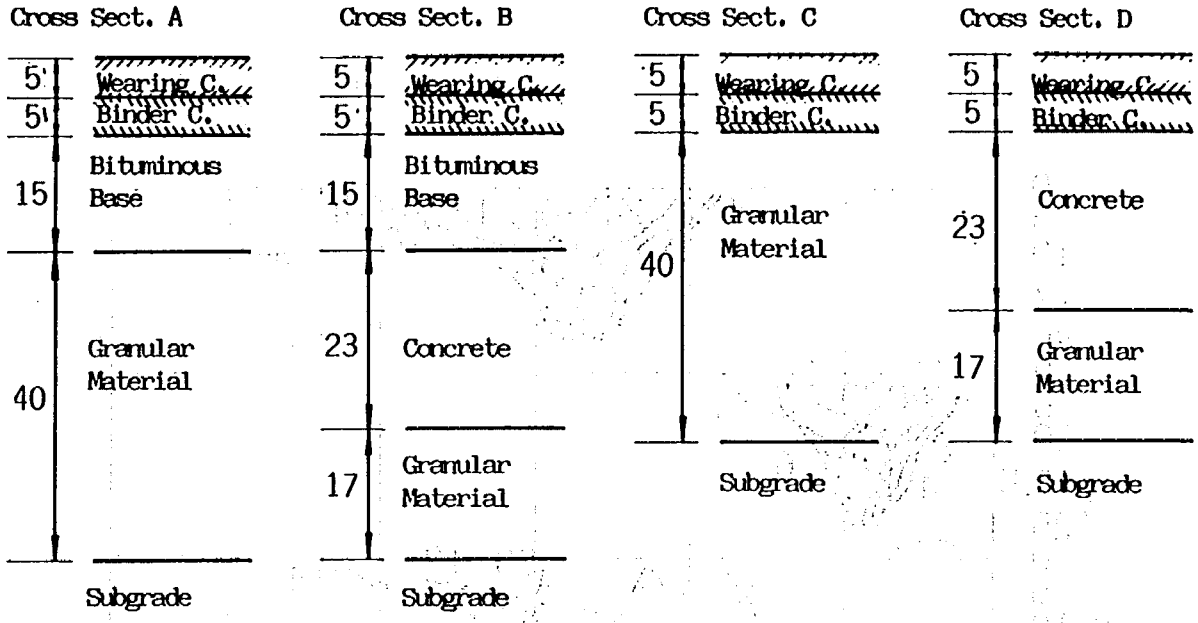


Figure-1 Cross Section (cm)

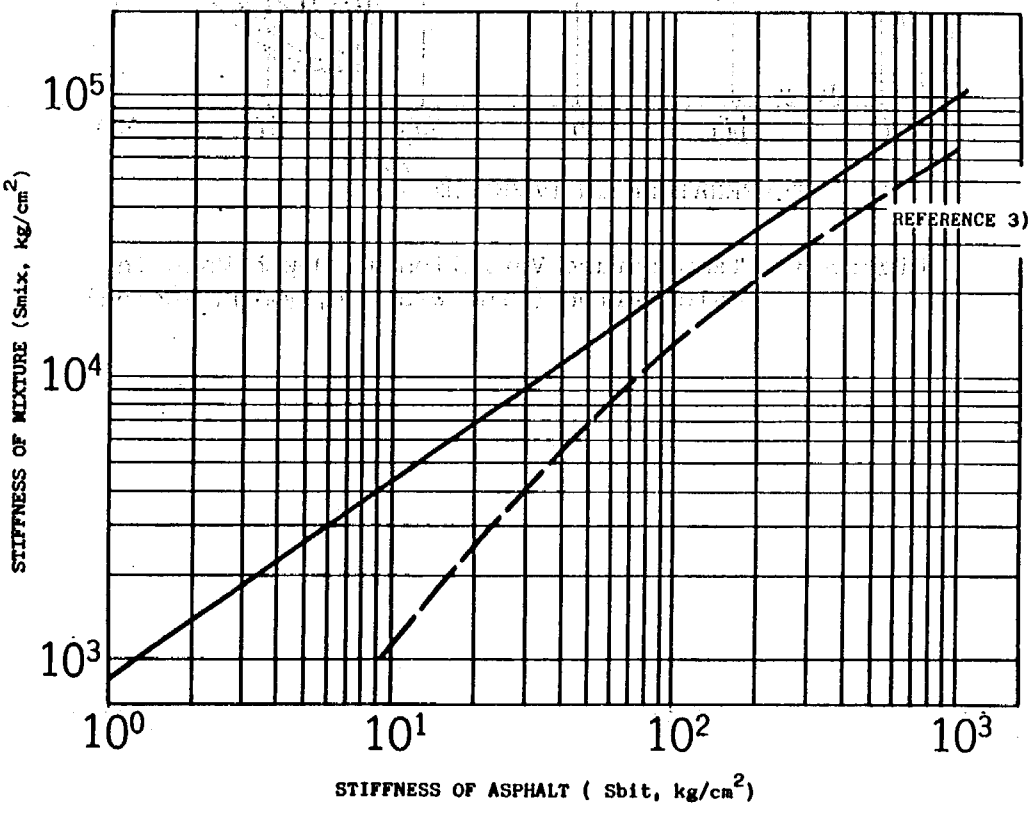


Figure-2 Relation Between Sbit and Smix

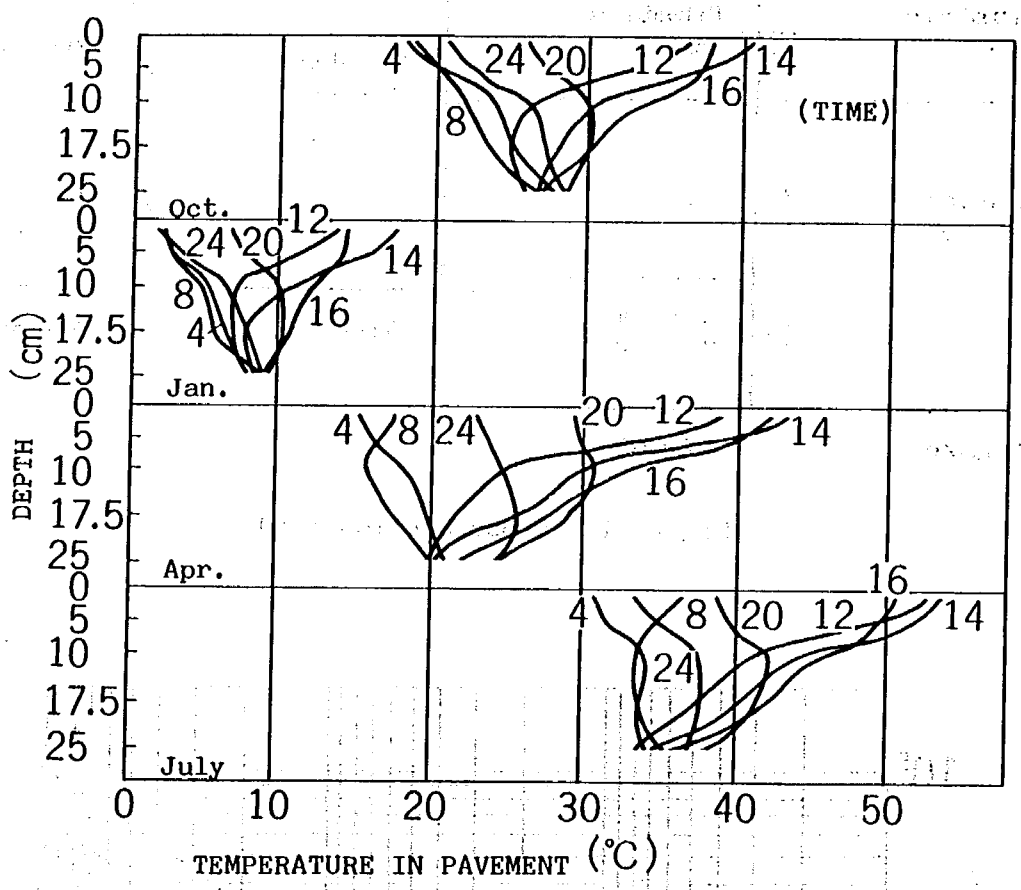


Figure-3 Temperature Variation in Depth Used in Calculation (Fine Weather, Kyushu Region)

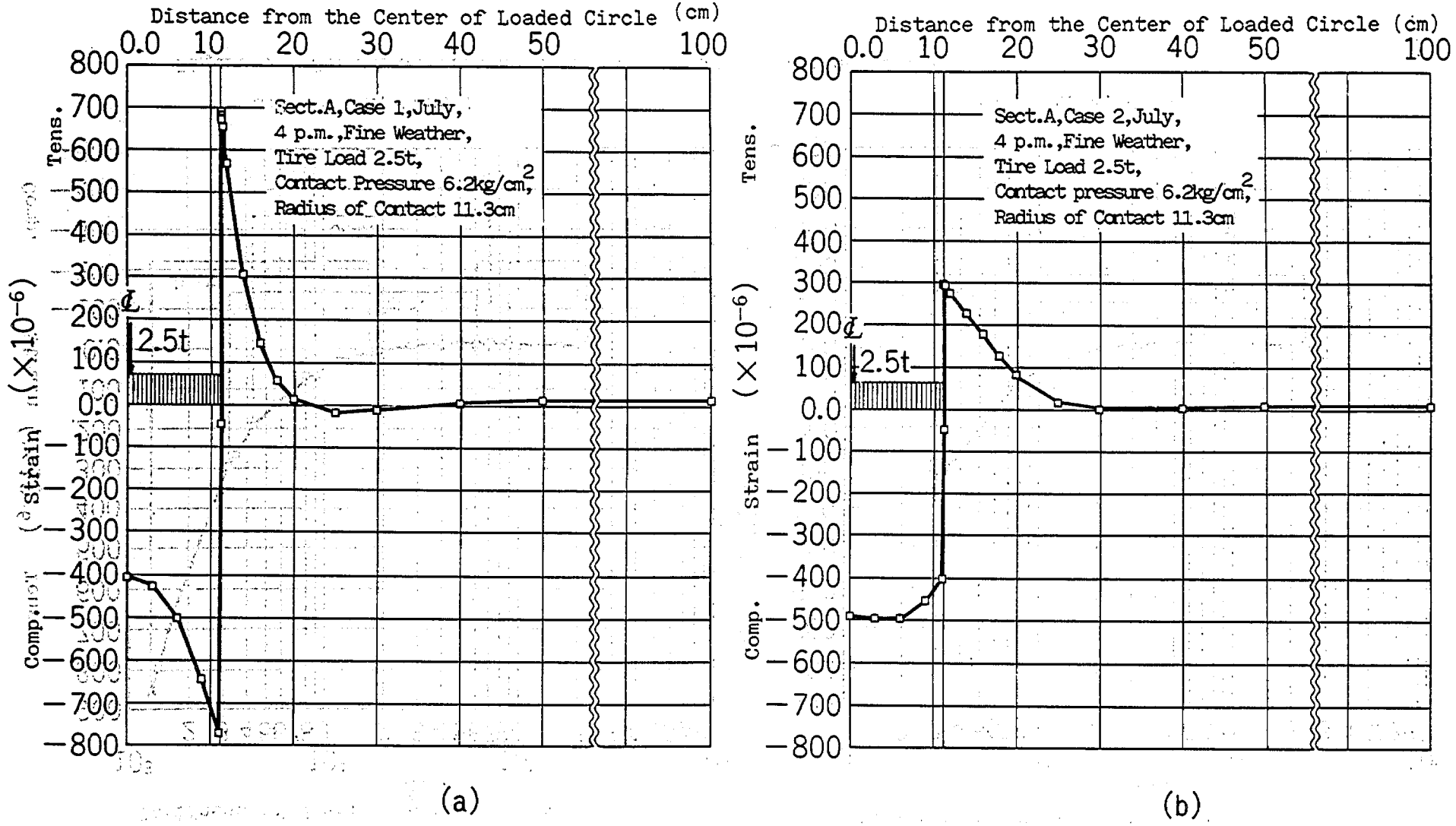
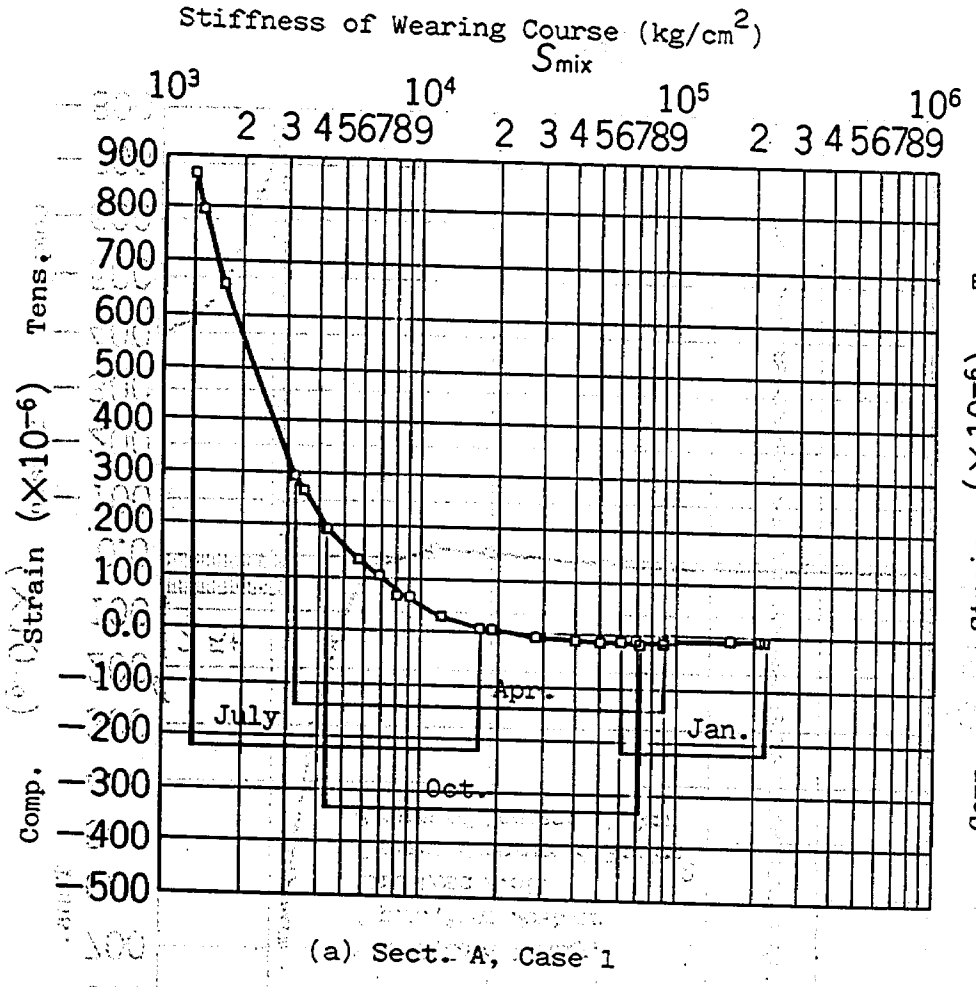
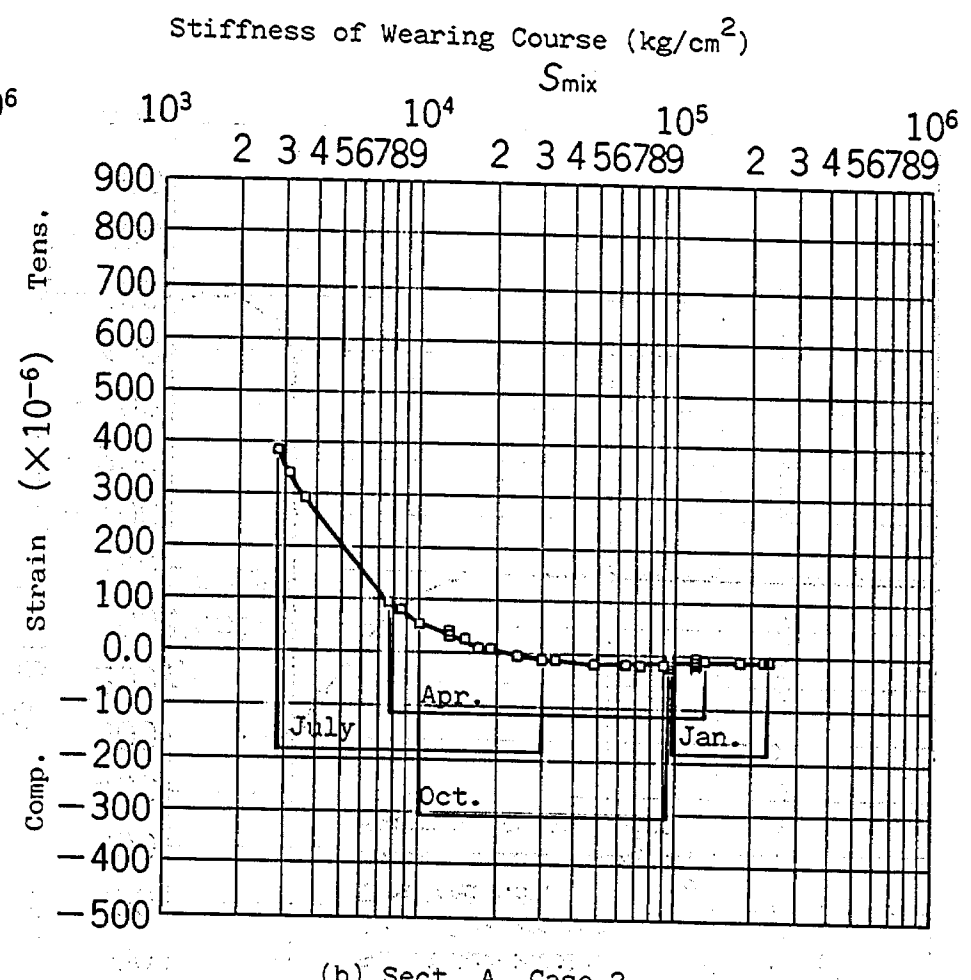


Figure-4 Transversal Surface Strain



(a) Sect. A, Case 1



(b) Sect. A, Case 2

Figure-5 Strain at Tire Edge and Stiffness of Wearing Course

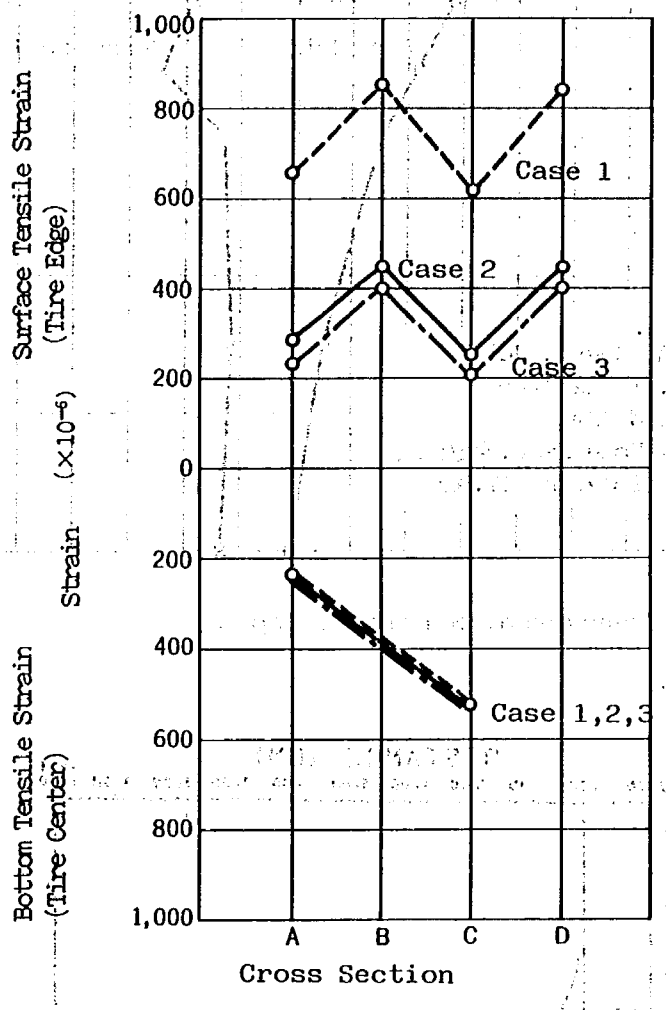


Figure-6 Comparison of Tensile Strain for Each Cross Sections (July, Fine Weather, 4 p.m.)

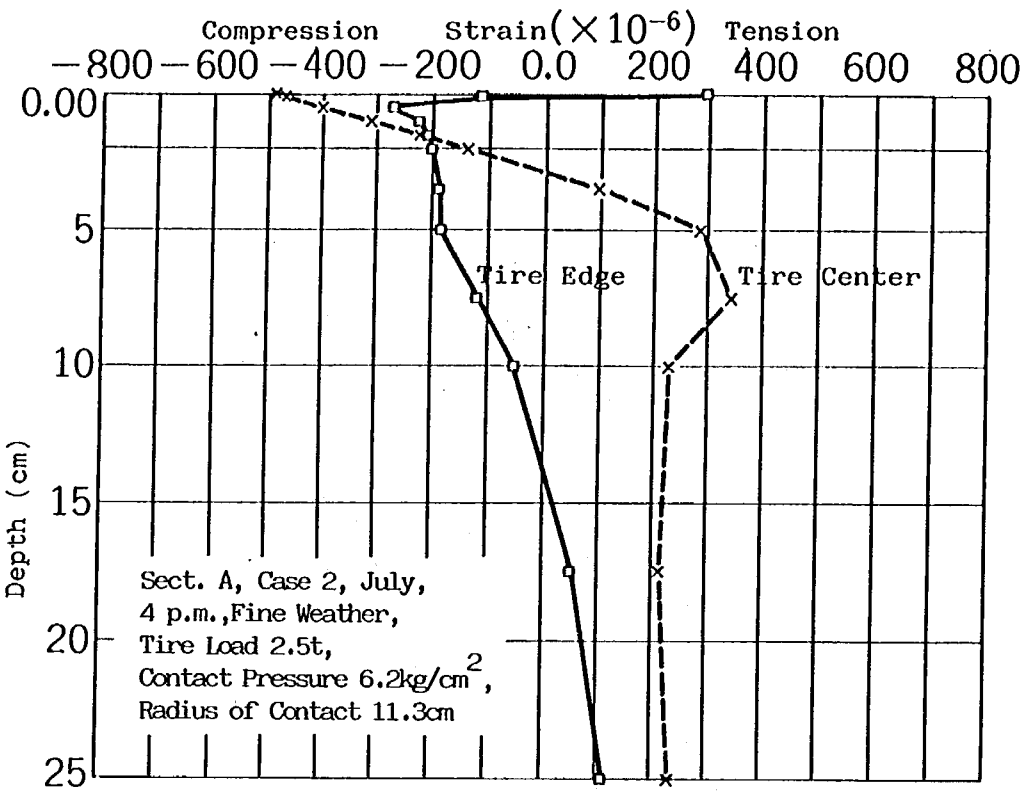


Figure-7 Transversal Strain in Depth

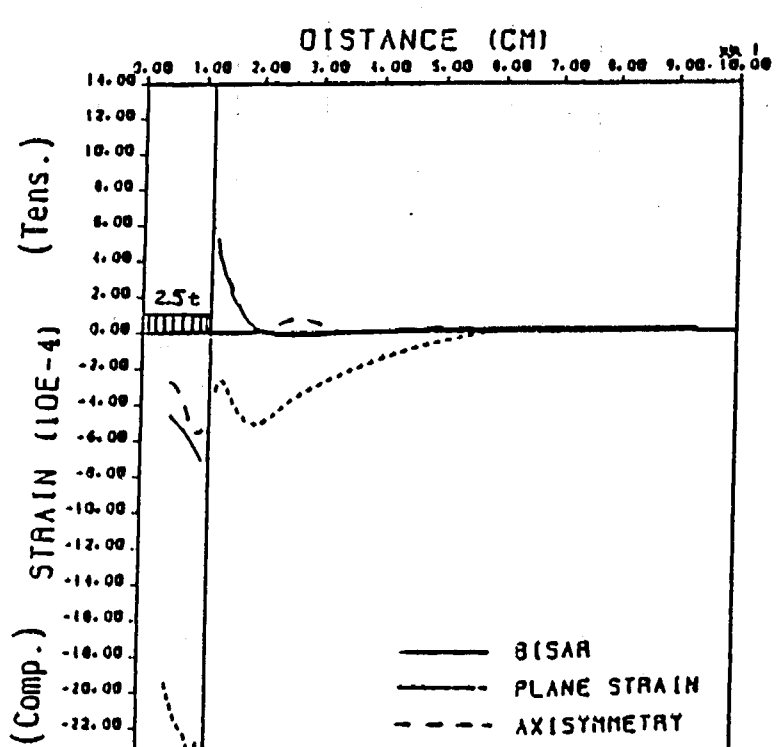


Figure-8 Transversal Surface Strain(July, Fine Weather,4 p.m.)

Sealing Cracks in Asphaltic Concrete Pavements

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ABSTRACT

In 1980 the Ministry of Transportation and Communications of Ontario embarked on a programme of field trials to identify the most suitable materials for sealing asphalt pavement cracks. The trials were subsequently expanded in 1982 to identify the most appropriate methods.

Current testing methods do not indicate the ability of a material to perform in asphalt pavements. These trials proved that existing MTC testing criteria would exclude three of the four best performing materials in that they would fail the bond test.

It was established that sealants which are installed such that they overlap on the pavement surface on each side of a routed crack have a much better winter survival rate than those poured flush. Better performance was observed with wider routing (19 mm) rather than narrow routing (13 mm) and sealing of non routed cracks of less than 8 mm did not prove successful.

It was demonstrated that it is of vital importance that cracks be clean and dry at the time of sealing. The use of hot compressed air as a conditioner immediately ahead of placing the sealant was determined to permit crack sealing to progress in damp conditions and significantly increase the long term bond of the sealant.

This work has led to three additional materials being approved for use in asphalt pavements and the extensive use of hot compressed air and overband sealing.

ACKNOWLEDGEMENTS

The authors wish to express their thanks to the sealant manufacturers, contractors, and to the maintenance staff of Huntsville and Ottawa Districts, for the interest and excellent co-operation extended in the construction of the trials, to the MTC Downsview laboratory staff for the large quantity of testing performed and to Nina DeRocchis for typing this report.

Views expressed in this paper are not necessarily those of the Ontario Ministry of Transportation and Communications.

INTRODUCTION

In 1980 the Ontario Ministry of Transportation and Communications embarked on a program to identify the most suitable materials for sealing cracks in asphalt pavements. Up to this time the products used were supplied from a list of designated sources of hot applied rubberized asphalt which conformed to ASTM D-1190 requirements, plus a more stringent MTC bond test. Opinions were expressed in the industry that products were available which would perform well in asphalt pavements but would not meet the criteria for the Designated Sources List. The appropriateness of the bond test which is performed between concrete blocks was questioned since these materials were to be used in asphalt pavements.

Manufacturers of products for sealing asphalt pavement cracks were invited to participate in a field evaluation through an advertisement in The Daily Commercial News as well as by personal contact by members of the Bituminous Section's staff. Six manufacturers responded offering a total of fourteen products along with proposed methods of installation. The Ministry accepted all proposed products for trial installation. Table 1 lists the materials used in the 1980 trials along with the generic type and method of installation.

THE 1980 TRIAL AREA

A nine kilometre portion of Hwy. 69 between Port Severn in the south and MacTier in the north was selected for the crack sealing trials. This location has a severe environment for crack sealing with January mean daily temperatures of -10°C and three expected occurences of -30°C per winter. This section of highway has two 3.65 m driving lanes with 2.74 m paved

shoulders constructed in 1976. The pavement had non continuous multiple longitudinal cracking about centreline and transverse cracking at approximately 6 m intervals extending from centreline to the edge of pavement.

The northbound lanes only were used so as to permit two way traffic on the southbound lane and shoulder during construction of the trials. The trial area was subdivided into twenty 450 m sections each containing approximately 1,000 m of cracks. Each section was marked with a bar of yellow paint and the sections were numbered from one in the south to twenty in the north.

The manufacturers of the crack sealing products were responsible for the installation of their products with the Ministry providing traffic protection. The manufacturers identified the installer of their products and a timetable was then prepared. The middle three weeks of July were selected for the trials, anticipating the most favourable weather in this period. Due to heavy summer weekend traffic on this highway, a four-day week, Monday to Thursday, was selected to minimize inconvenience to Friday afternoon traffic. To install 14 products in twelve working days and make allowance for inclement weather, the installation of two products at one time was required.

Since 20 sections were laid out and only 14 products were to be installed, sections 7 to 20 were selected for use, so as to take advantage of the straighter alignment in that portion of the trial area. One cold pour sealant (RS-1 & sand) was cancelled from the trials causing the trials to end in section 19 rather than section 20.

Table 1 lists the materials used in the 1980 trials along with the generic type and method of installation.

TABLE 1

SUMMARY OF MATERIALS & INSTALLATION METHODS (1980)

TRIAL	MANUFACTURER	PRODUCT	PRODUCT DESCRIPTION	INSTALLATION METHOD
7	Uniroyal	6160	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Filled Flush
8	Be-Ram	Supergook 21-151	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Overfilled & Levelled
9	Uniroyal	6165	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Filled Flush
10*	Flintkote	X-8032	Cold Poured Rubber Asphalt Emulsion	Routed/Air Blown/ Filled Flush
11	Chevron	Rubberized Crack Filler	Cold Poured Rubber Asphalt Emulsion	Routed/Air Blown/ Overfilled & Levelled
12	Meadows	164R	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Filled Flush
13	Flintkote	X-8006	Cold Poured Solvent Borne Rubber Asphalt	Routed/Air Blown/ Filled Flush
14	Meadows	164	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Filled Flush
15	Flintkote	X-8013	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Filled Flush
16	Meadows	Sof-Seal	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Filled Flush
17	Crafco/ Sahuaro	Overflex - MS Modified	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Overfilled & Levelled
18	Crafco	Overflex MS	Hot Applied Rubberized Asphalt	Routed/Air Blown/ Overfilled & Levelled
19	Flintkote	X-8013	Cold Applied Two-Part Rubber Asphalt Emulsion	Routed/Air Blown/ Filled Flush

* Washed out by rain and repaired with X-8013

CONSTRUCTION OF THE 1980 TRIALS

Routing the Grooves

All the manufacturers of the sealing compounds required the cracks to be routed. Tennant and Crafco routers were used for most of the work with vertical routers of various manufacture being employed for the more meandering cracks. Even though this was a demonstration project, and one would expect to see above average workmanship, there were numerous examples of cracks being missed in the routing operation. This problem was most severe where the Tennant router was employed, for it is best suited to straight line cracks. The Crafco router could follow meandering cracks far better than the Tennant. The Crafco routers could be used alone but whenever the Tennant router was used alone many meandering cracks were missed. All the routers produced grooves with semi-vertical sides and rounded bottoms. Most routing was to a width and depth of about 19 mm but some were deliberately less.

Unfortunately, the weather was not too favourable during the trials and rain caused several delays and some less than desirable installations. After a rain and subsequent drying of the pavement surface, the cracks would still be moist for up to a day. When the damp cracks were routed, the moisture caused the finer fraction of the cuttings to form a slurry in the groove which resisted removal by either brushing or blowing.

Cleaning the Grooves

The routed grooves were all cleaned with a jet of air, mostly from back pack blowers with air velocity of up to 320 km/hr. A rented compressor was at first used in Section 7 at the manufacturer's request but was given up in favour of a back pack blower after a very short time. The compressor is

bulky and must be constantly moved therefore is not nearly as convenient as the back pack blower. The back pack blowers were found to be in very poor to good condition. Only back pack blowers in top working condition were able to do a satisfactory job of cleaning out the grooves. Problems encountered with these units included:

- engine hard to start,
- engine would not run at full speed,
- engine stalling,
- hoses and nozzles in poor repair.

A Crafco wire brush/blower was used for short periods in Sections 16, 17 and 18, and this equipment did a superior job of cleaning the groove. The Crafco wire brush/blower was much more convenient than a compressor but still much slower in operation than a back pack blower.

In routed grooves that were damp neither back pack blowers nor the Crafco wire brush/blower would remove the fine slurry deposit.

It was necessary to air blow the grooves immediately preceding the filling operation or adjacent traffic blew dirt back into the grooves. Whenever the blowing of the grooves got more than about 30 metres ahead of the pouring of the sealant, the grooves were observed to be dirty.

Filling the Grooves

The sealant was placed in the routed grooves with either a hand pouring cone, or by hose and application wand from a low pressure pump on the melter. The hand pouring cones are equipped with replaceable nozzles (standard 3/4 inch iron pipe thread) and with some of the more viscous

sealants, these had to be removed to allow sufficient flow of material.

All but four of the materials were poured flush with the pavement surface. The other four sealants, three hot applied and one cold applied, were installed by overfilling the routed groove and then striking the material off. In trial Sections 17 and 18 a specially designed "v"-shaped steel strike-off which limited the width of the spread was used, and a rubber squeegee was used in Sections 8 and 11.

Hot poured sealants placed in damp grooves did not develop much of a bond within the groove. In section 8, the northerly 171 metres of the section was placed in a damp condition but since the surface was dry and the material overfilled and levelled, a good bond was achieved on the surface. Section 9 installed at the same time was poured flush in a damp groove and a poor bond was observed. Six days later an examination of Section 9 revealed some improvement in bond of the sealant.

The hot poured sealants, when dusted with sealbond or portland cement, could be exposed to traffic within 15 minutes. All the cold applied sealants were skinned over but still liquid under the skin after an hour and subject to damage and tracking if exposed to traffic.

In Section 10, the cold applied material was washed out of the grooves during an overnight rain. This section was subsequently repaired with the hot poured material used in Section 15 and removed from the evaluation.

In two of the hot applied trials the manufacturers' recommended temperature range for application was not followed; Section 14 was above and Section 16 below the recommended range.

OBSERVATIONS

- The Tennant router had the highest production rate of all the routers used on the trials but was not effective in routing meandering cracks.
- The Crafco router, although less productive than the Tennant router, could, with care, rout meandering cracks.
- The vertical routers are the least productive but most effective in treating meandering cracks.
- Many cracks were missed during routing in spite of the trials being a demonstration project.
- If the cracks showed dampness when routed, a muddy slurrywas deposited within the groove which could not be removed by back pack blowers.
- Back pack blowers had to be in first class condition in order to do a satisfactory job.
- Cleaning of the grooves too far in advance of the filling operation allowed traffic to blow debris back into the groove.
- Hot poured sealants placed in a damp groove would not bond properly.
- Cold poured sealants do not set up quickly enough to be used in typical highway crack sealing operations where the seals are exposed to traffic in less than one-half hour.
- Emulsion type cold poured sealants are prone to wash out if rain occurs within a few hours of installation.
- Cold poured sealants are subject to tracking for up to a day after installation.

1980 TRIALS REVIEWED

Following three field appraisals (Nov. 1980, Jan. 1981 and June 1981) and discussions with the MTC staff responsible for maintenance of this section of highway, it was determined that the most successful 1980 trial installation was Section 8, which employed the method of overfilling the routed grooves and striking the sealant off flush. The second, and almost equally successful trial, employed the method of filling the routed groove flush with sealant. In this trial section (No. 7) the material tracked out of the groove a little and had the final appearance of having been overfilled and struck off.

None of the twelve materials installed in the 1980 trials maintained a 100% seal throughout the first winter. Most of the failures were in bond to the side of the routed groove with some cohesive failures. Six trials were considered complete failures (11, 13, 15, 17, 18 and 19), all three cold poured and three of the nine hot poured sections. The other four hot poured trials, all employing the fill flush method, although not considered complete failures, are deemed to have performed less than satisfactorily. Two hot poured trials (17 and 18) which employed the system of overfilling the routed groove and striking off the sealant, failed due to the material becoming very hard and brittle in the cold weather. The materials in these two sections cracked down each side of the routed groove and offered little resistance to water infiltration until the following June when the warm weather softened the material and it resealed considerably. This resealing of previously failed seals in warm weather was observed in several of the trial sections. It is apparent that summer only evaluation of crack sealing can easily deceive the observer into thinking that a particular seal had been effective and still

in good condition when, in fact, it may well have failed badly when it was most needed.

Due to the variation in installation technique and the variable conditions of the installations, it was not possible to determine to what degree the individual materials influenced the rate of failure.

1981 PROGRAMME

In order to evaluate all of the materials on an equal basis, it was decided to repeat the trials in 1981, specifying a common method and condition of installation for each product as follows:

- Each supplier to have a representative on the site to supervise the installation of his product.
- Routing and sealing work to be performed only when the cracks (not only the pavement surface) are dry.
- Only routing that would be filled the same day to be performed. Width and depth of rout to be 19 mm. Routers capable of following the exact crack line to be employed.
- Immediately ahead of sealing, the routed grooves to be thoroughly cleaned by compressed air.
- The routed grooves to be overfilled with sealant and struck off such that a minimum overlap of 40 mm on each side of the groove is achieved.
- The completed seal to be dusted with an appropriate material to eliminate surface tackiness.

The manufacturers who participated in 1980 and who did not employ the overfill and strike off method were encouraged to reparticipate in 1981.

and other prospective manufacturers were advised of the trials through an advertisement in the Daily Commercial News.

Nine suppliers offered fourteen products for evaluation and thirteen of those were approved for installation. A cold pour material that failed badly in the 1980 trials was declined. Of the thirteen products accepted, two were cold poured materials which had not been installed in the previous year's trials and were only accepted with the suppliers assurance that they would set up sufficiently to permit traffic in 15 minutes. Section 8 which employed the overband method and had performed well in the 1980 trials was included in the new trials.

Each supplier advised the Ministry which contractor would install his product and a timetable for the installations was drafted for July 6-23.

LOCATION OF TRIAL AREA

The trial area continued northward from the end of the 1980 trials.

Table 2 lists the materials used in the 1981 trials with the generic type and method of strike off.

TABLE 2

SUMMARY OF MATERIALS AND STRIKE OFF METHOD (1981)

TRIAL	MANUFACTURER	PRODUCT	PRODUCT DESCRIPTION	STRIKE OFF
8	Bemac	Supergook	Hot Poured Rubberized Asphalt	Flat
20	Bakelite Inc.	Flintseal 590-13	Hot Poured Rubberized Asphalt	Flat
21	Hydrotech	Sealz 6165	Hot Poured Rubberized Asphalt	Flat
22	Crafco Inc.	Roadsaver 201	Hot Poured Rubberized Asphalt	Flat
23	Hydrotech	Sealz 6160	Hot Poured Rubberized Asphalt	Shaped
24	Paraseal	Super-Seal 2065	Hot Poured Rubberized Asphalt	Flat
25	Paraseal	Ultra-Seal 2070	Hot Poured Rubberized Asphalt	Flat
26	Meadows	164R	Hot Poured Rubberized Asphalt	Flat
27	Shell Can.	Cariphalte ELT	Hot Poured Rubberized Asphalt	Shaped
28	Meadows	Sof-Seal	Hot Poured Rubberized Asphalt	Flat
29	Meadows	Hi-Spec	Hot Poured Rubberized Asphalt	Flat
30	Chevron	C.I.M.	Two Part Asphalt Extended Urethane	Flat
31	Southwestern Petroleum	Patching Compound	Cold Applied Asphaltic Mastic	Flat
32	IBIS	Prismoseal A-1	Hot Poured Rubberized Asphalt	Flat
33*	Bemac	Supergook	Hot Poured Rubberized Asphalt	Flat

* Overband sealing trial using HCA only to treat crack.

CONSTRUCTION OF THE 1981 TRIALS

The weather during the trials was exceptionally good with no time loss due to rain. Daily temperatures ranged between 18 and 32°C with mostly clear skies. The environmental conditions for the trials, 20 through to 31, should be considered equal. Trial No. 32 was conducted more than a month later in cool but dry conditions.

In addition to the 13 material trials, a demonstration was performed in Section No. 33 using hot compressed air (HCA) to dry unrouted cracks prior to overband sealing. This work was performed intentionally in adverse weather conditions. A similar HCA lance with less air volume and heat, was used to blow out the routed cracks in Trial No. 32.

Routing the Cracks

As in the 1980 trials, the routers used by the contractors were either:

(a) A Tennant router followed by a vertical router or

(b) a Crafco router working alone.

All routing was performed to a width and depth of 19 mm. The overall quality of routing this year was improved over that observed in the 1980 trials. It was evident that even with the additional care being taken that the Tennant router was incapable of routing the meandering cracks. When a crack changed direction quickly, the Tennant router would not respond fast enough and would miss the crack, requiring the vertical router to be employed to cut the missed portion. This would result in two cuts side by side which would either create a wide groove or leave an island of pavement between the actual crack and the groove cut by the Tennant router. (See arrows in Figure No. 1). The cutting debris left by the Tennant would

often obscure the missed cracks and the operator of the vertical router would then miss cutting some.

FIGURE 1

ROUTING DEFICIENCIES

The Crafco router, with its cutting head on the same axis as the wheels supporting the machine, could respond quickly to a change in crack direction.

Although the Tennant router is much more productive on straight line cutting, the Crafco router is more productive cutting meandering cracks.

Cleaning the Grooves

The routed grooves were blown out with back pack blowers in all but one test section. The blowers used this year were in a good state of repair and were effective. As in the 1980 trials the problem with adjacent traffic blowing dirt back into the cleaned grooves occurred when the cleaning operation was more than about 30 metres ahead of the sealing operation.

In trial No. 32, a Prismo hot compressed air (HCA) lance #35 was used to blow out and heat the routed grooves prior to placing the sealant. The HCA lance system is comprised of a compressor, liquid propane tank(s), mixing unit, hoses, lance and an ignition device. In operation, the combustion chamber glows red hot. This lance (with a 13 mm orifice) did an excellent job of cleaning out and heating the routed grooves but was much slower than cleaning with a back pack blower.

Filling the Grooves

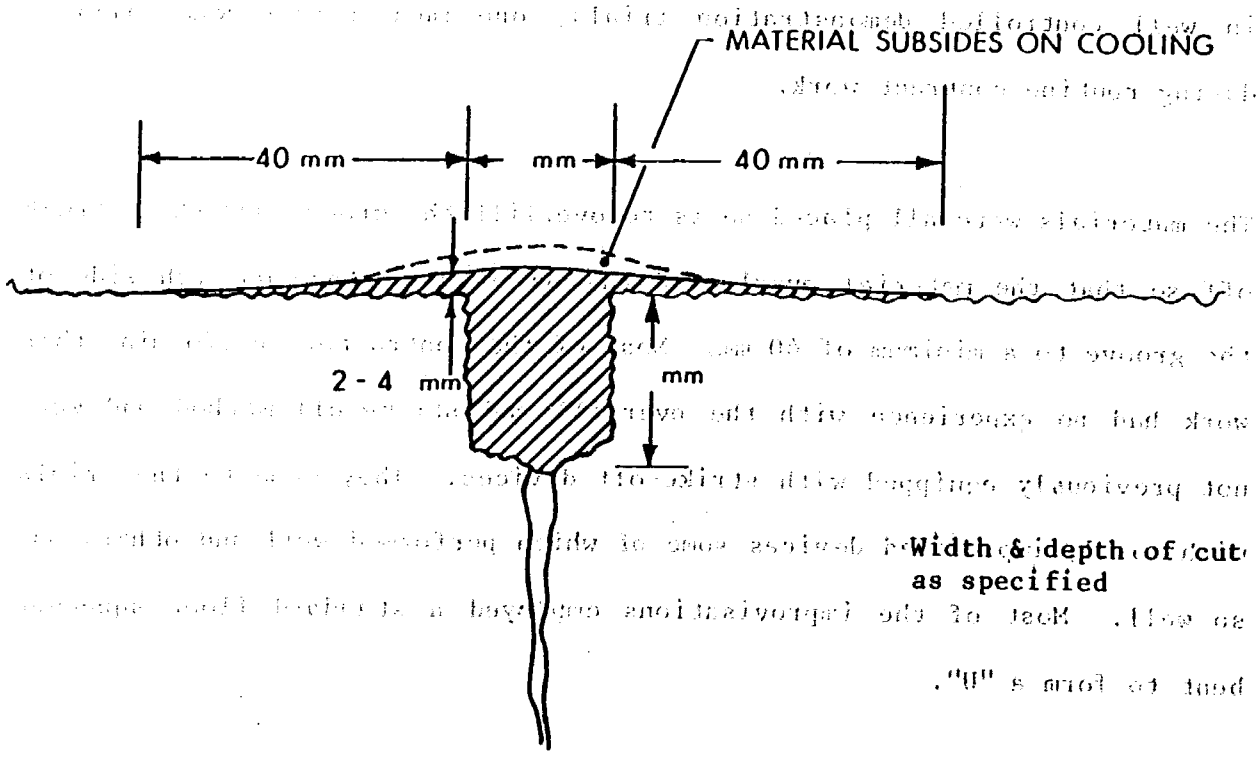
All the melters used in the trials were of the oil jacketed double boiler type except for the Prismo unit used in Section 32 which was direct fired. The Prismo melter and the Crafco melter used in Sections 22, 24, 25 and 27 had thermometric controls which automatically controlled the product temperature. These automatic controls are a definite advantage in preventing overheating of the sealant which occurred in 50% of the trials using melters without automatic controls. With this high incidence of overheating in well controlled demonstration trials, one must wonder what happens during routine contract work.

The materials were all placed so as to overfill the groove and then struck off so that the material overlapped the pavement surface on each side of the groove to a minimum of 40 mm. Most of the contractors performing this work had no experience with the overfill and strike-off method and were not previously equipped with strike-off devices. They came to the trials with mostly improvised devices some of which performed well and others not so well. Most of the improvisations employed a standard floor squeegee bent to form a "U".

The desired cross section of seal in a routed groove is as shown in Figure 2. Most of the strike off devices used just levelled the sealant but the "U" and "V" shaped squeegees with a crescent cut out of the rubber at the back gave the desired cross section to the sealant. The sealant needs to be struck off so as to leave a bead of material over the routed groove with the edges of the material feathered out. The bead of material will flatten out considerably after subsiding into the crack and through thermal shrinkage. If the sealant stands too proud of the surface, it is subject to snow plough damage. Due to either inappropriate strike off devices or lack of operator skill, lumps of sealant were left proud of the pavement surface at the ends of cracks or where a crack changed direction. This material created a noticeable bump which will likely be cut off in the winter by snow plowing operations, with the risk that additional material may be pulled out of the sealed crack.

FIGURE 2

DESIRED CROSS-SECTION OF ROUTED AND SEALED CRACK



Strike-off devices which are pushed rather than pulled allow the operator to see where he is going and thus contribute to a safer operation. Another advantage of the push type strike off is that the operator applying the sealant can see the needs of the strike off and adjust the amount of material dispensed accordingly.

One hot poured material (Section 27) was found to be subject to traffic damage 15 minutes after installation. The cold poured materials (Sections 30 and 31) were both subject to tracking after 15 minutes. In Section 30, a two component material, one batch had not set up the next day and tracked down the highway 50 metres or more. After being opened to traffic, the cold poured sealant on the pavement surface in Section 31 tracked badly, but remained fairly well in the transverse grooves. The material in the longitudinal grooves was almost all removed by traffic in this section.

Demonstration of Overband Sealing

Kardey Ltd. (UK) in co-operation with Bemac Ltd. (Toronto) performed a demonstration of their equipment at the MTC Downsview complex November 19th, 1981. The equipment demonstrated consisted of a hot compressed air (HCA) lance, barrow applicator, hand pouring pot and band forming tools. This demonstration was to show the equipment capabilities and to promote the U.K. system of overband sealing of asphalt pavement cracks.

The hot compressed air lance is supplied by a 100 lb. L.P. propane bottle and a 160 C.F.M. compressor. The propane and compressed air are metered and mixed in a special mixing unit and the product supplied to the lance via a single hose. The gas is ignited by a sparking device in the combustion chamber and the unit allowed to heat up for about 3 minutes until

cherry red. The HCA lance is a hand tool about 1.5 m long which is coursed along the pavement crack at a slow walk to blow out debris and to heat and dry the crack. The HCA lance is also very effective in removing zone paint without any apparent harm to the pavement.

The barrow applicator is a three wheeled device which can hold about 25 kg of molten sealant at application temperature and deposit it into the heated crack and strike it off. The barrow has burners supplied by a 20 lb. propane bottle which maintains the product temperature during application. This hand pushed barrow can seal significantly more crack than a pouring cone.

The hand pouring pot and band forming tools were used to fill areas of crack that had subsided after the barrow application - a touchup operation.

Staff observing the demonstration were impressed by the equipment and arrangements were made to construct a field trial in Section No. 33 of the Hwy.69 trial area on 81 10 23 using Bemac's Supergook as the sealant. The weather was very poor for this type of work with an ambient temperature of 0°C and intermittent rain and snow showers. The HCA Lance was capable of drying and blowing out the cracks faster than the barrow applicator could seal them. Work with the barrow applicator was a little slow due to the sealant not being hot enough to get a good flow. It was noted that the barrow applicator could actually raise the product temperature. The barrow was very efficient on straight cracks but was awkward to use on the meandering cracks. By the time the touchup work with hand pouring pot and band forming tool was performed, the areas had been wetted by rain and snow.

1981 TRIALS REVIEWED

December 1981

The December review revealed that an unexpected 30% of the 15 trials were already experiencing some splitting of the sealant. The splitting was mostly at the edges of the routed groove where the sealant on the pavement surface was struck off very thin.

February 1982

The February appraisal revealed that two trials were performing well; (0-10% Splitting)

1 - Section 27, Shell Cariphalte ELT

2 - Section 33, Overband Sealing Trial without routing employing HCA and Bemac Supergook

Two other trials were acceptable; (10-20% Splitting)

1 - Section 21, Hydrotech Sealz 6165

2 - Section 29, Meadows Hi-Spec.

Four materials were rated marginal in performance; (20-50% Splitting)

1 - Section 20, Bakelite 590-13

2 - Section 23, Hydrotech Sealz 6160

3 - Section 24, Paraseal 2065

4 - Section 26, Meadows 164R

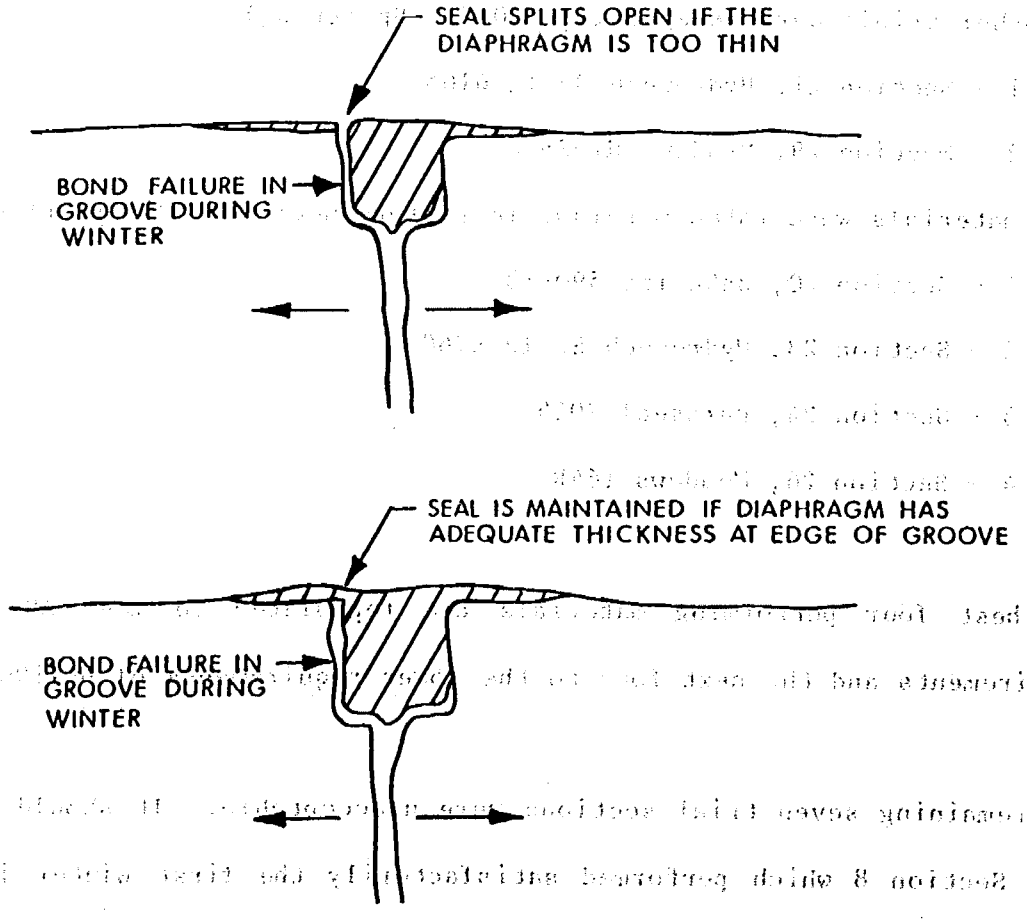
The best four performing materials are formulated to meet ASTM D-3405 requirements and the next four to the lower requirements of D-1190.

The remaining seven trial sections were unacceptable. It should be noted that Section 8 which performed satisfactorily the first winter is now in its second winter and is included in the unacceptable category.

COMMENTARY

During the two appraisals it became evident that splitting of the seals at the edge of the rout was minimized if the membrane at that point had a 2-3 mm cross-sectional thickness. Where the sealant was struck off thin, (less than 1 mm) poor performance was common. In several sections it was noted that an effective seal was maintained even though the material was no longer bonded in the routed groove as confirmed by pinching the seal between thumb and forefinger and lifting it up and down. This occurred when the diaphragm across the rout had enough cross-sectional area to maintain its integrity. Figure 3 indicates how the cross-section of the seal can affect winter performance.

FIGURE 3



Damage to the sealant by snow plows was very limited. The plows usually just sheared off sealant that stood more than 3 mm proud of the pavement surface. Damage was most frequent on longitudinal seals and at high points on transverse seals such as between the wheel track ruts. In the case of the two materials with a much lower stiffness modulus (Cariphalte ELT & Sof-Seal) the plows tended to pull the sealant from the pavement rather than shear it off.

There had been concern that where the matrix of the pavement was etched out with HCA and not covered with sealant, premature deterioration would result. In several locations where this occurred in section No. 33 no deterioration is in evidence to date.

The work in 1981 and 1982 established that seals fail primarily in bond and that if moisture was present at the time of routing and sealing, bond failure was almost assured. To clarify, separation or splitting of a seal at the edge of the routed groove has been identified as a bond failure. This is a somewhat oversimplified description because the actual failure may be a combination of bond/cohesive failure plus a partial failure of the pavement. Sealants with a good bond and high stiffness modules will pull aggregate that have been disturbed in the cutting operation out of the side of the routed groove. This was observed in many of the test sections. Figure 4 demonstrates how a "bond" failure may be a combination of failures.

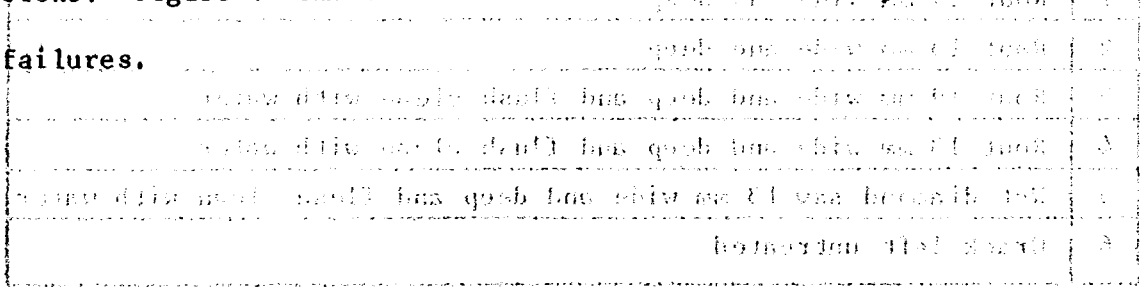
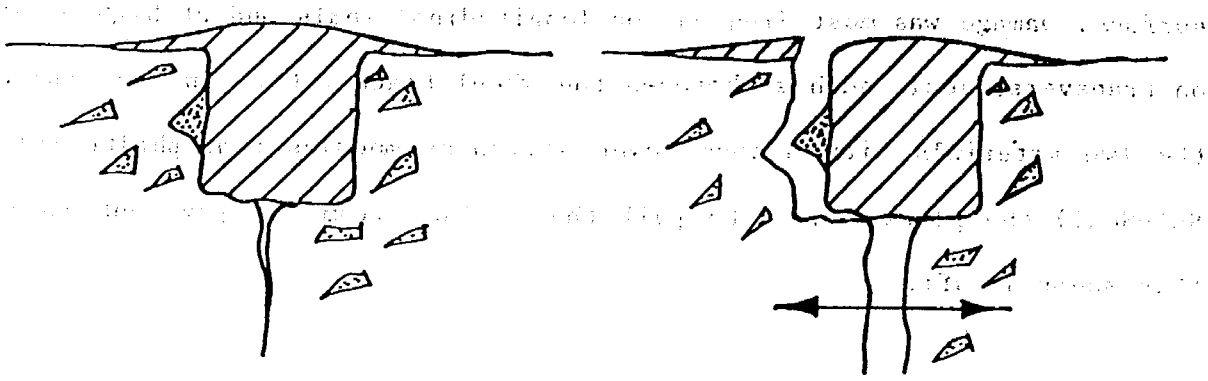


FIGURE 4

GENESIS OF A "BOND" FAILURE



1982 PROGRAMME

Since the prime requisite of dry conditions seldom exist for routing and sealing, a method of sealing cracks in damp conditions was identified as a need. A trial was organized to evaluate the use of HCA in combination with various methods of crack preparation which would permit crack sealing to proceed in damp conditions. Six methods of crack preparation were selected to be installed in triplicate using three of the most promising materials identified in the materials trials.

The method of crack preparation prior to being treated with HCA is contained in Table 3.

TABLE 3

METHOD OF CRACK PREPARATION PRIOR TO HCA

1	Rout 19 mm wide and deep
2	Rout 13 mm wide and deep
3	Rout 19 mm wide and deep and flush clean with water
4	Rout 13 mm wide and deep and flush clean with water
5	Wet diamond saw 13 mm wide and deep and flush clean with water
6	Crack left untreated

After the crack preparation and not necessarily on the same day, the crack was dried, blown clean and heated with HCA immediately ahead of placing the hot sealant according to Fig.2. The HCA slightly etched out the matrix of the pavement on the surface, for a width of 40 mm each side of the prepared crack.

The six methods were performed in triplicate using:

1. Supergook - Bemac Industries Ltd.,
2. Hi-Spec - W. R. Meadows Canada Ltd.
3. Cariphalte ELT - Shell Canada Ltd.,

LOCATION

A section of the Ottawa Queensway (Hwy. 417) a four lane divided highway, between Richmond Rd. and Eagleson Rd. was chosen for the trial area. This section of highway had been crack sealed four years previously and new cracks required sealing. Since the older sealed cracks were routed and sealed by pouring flush, the new crack sealing with the overband seal would be easily distinguishable. Each section was selected to contain approximately 500 m of crack and the beginning of each section was marked with a steel post and small number plate at the edge of shoulder in the median.

The Ottawa area provides a severe environment for crack sealing with January mean minimum daily temperatures of -15.6°C and three expected occurrences of -30°C per winter.

EQUIPMENT

The following equipment was employed in the trials:

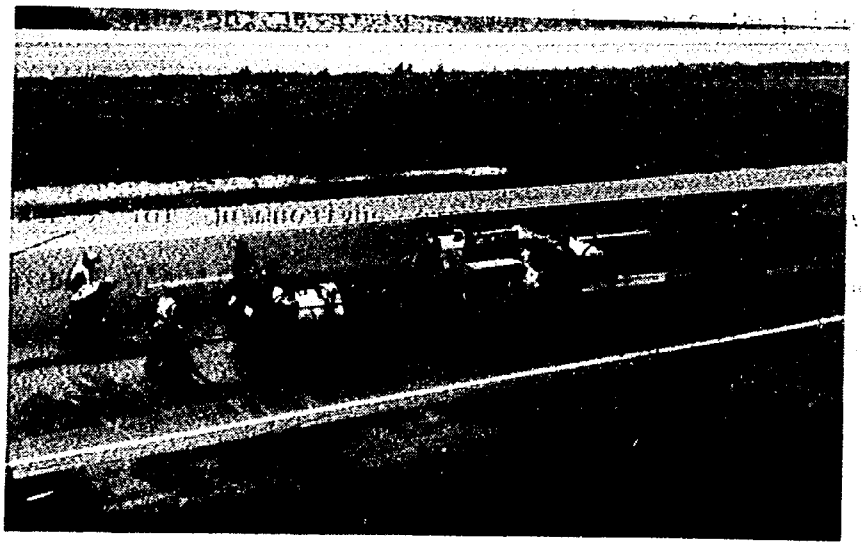
- Routers (2), Crafco Model 200
- Diamond Saw (1), Easy-Cut Runway Crack Cutter Model WCE-1200
- Melter (1), Crafco Model BC 220
- H.C.A. Lance (1), Bemac Model B-1

A two-ton truck with a stake body, was equipped with a 170 CFM compressor in the back along with a rack of five 45 kg liquid propane bottles headed together. The HCA lance mixing unit was then coupled to the compressor and propane tanks and the 42,000 volt ignition system for the lance wired into the truck electrical system. This truck was also used to haul the melter, providing a very compact working unit. See Figure 5.

A five-ton truck equipped with a 4,550 L water tank and pump was used to wet down the pavement and to flush out the specified cut cracks.

FIGURE 5

TWO TON TRUCK WITH HCA LANCE AND MELTER



CONSTRUCTION

At the beginning of each day's work, the water truck thoroughly wetted down the pavement in the working lane. The weather was good throughout the trials with average daily temperatures of 20°C drying the pavement surface quickly. Even so, the cracks would remain damp throughout the work period.

Routing

In the sections requiring routing, the two routers easily kept ahead of the sealing operation, often working two sections ahead of the sealing work. One router was set up to cut 13 mm^2 and the other 19 mm^2, and each router would work separate sections.

Sawing

The pavement sawing with the "Runway Crack Cutter" was found to be extremely slow with only 67 metres of crack cut in the first five and one-quarter hours of work as compared to 300-400 metres per hour for a router. Because of the poor production rate a decision was made to saw only one lane of each designated section and to rout the other lane. A separate traffic control zone was set up to saw the other designated sections ahead of time, so as to improve productivity. When the operator of the diamond saw tried forcing the machine to achieve better production, the blade would rise up and thus the cut depth was diminished. The cutting slurry produced by the wet diamond saw obscured the crack, making it difficult for the operator to follow. Spray painting the cracks yellow greatly improved visibility for the operator. The cut produced by the Runway Crack Cutter was very clean with smooth vertical sides and a flat bottom. This is in contrast to router cuts with ragged sides and bottom with loosened aggregates.

Flushing

Water flushing of the designated cuts, cleaned out the debris very well, removing the fine slurry on the cut faces of the sawn and routed cracks. A garden hose and nozzle from the pump on the water truck was used for this operation.

Hot Compressed Air (HCA)

It was intended that the sealant would be installed immediately behind the HCA treatment which required the lance to be operated behind the melting kettle. This was tried for a short period of time but a concern for the safety of the workers dictated a procedural change. With three men working in close proximity behind the kettle, one with the HCA lance and one with the wand from the melter discharging hot sealant and the third with the band forming tool, the chance for an accident was considered too great. A decision was made to have the HCA lance work alongside and ahead of the truck hauling the melter and for the sealing operation to work alongside and behind the melter. As the work progressed down the road, the driver of the truck had to be very alert so as to co-ordinate his forward travel to meet the need of both operations.

The HCA lance performed well, drying out, heating and etching the prepared cracks at a slow walk. The lance operator had to take care to blow the routing debris back wide enough so that it would not get caught by the band forming tool used for shaping the sealant: this became a problem if the blast from the lance was concentrated in the routed groove only, without fanning the blast around. The operators of the HCA lance were rotated to other duties about every 15-20 minutes because their feet became uncomfortably hot. The five 45 kg proprane bottles supplying the lance lasted

for the full ten working days of the trials with propane left over. It was later found that an average of one bottle per 8 hour day was consumed by the lance. The only time difficulty was encountered in drying the groove occurred on a grade where free water ran back from water flushing operations ahead. After treatment with HCA, all the cut cracks regardless of preparation method, had a very similar appearance with the matrix etched out in the groove.

Sealing

The hose and wand on the Grafco kettle became plugged several times during the trials and this usually was a result of trying to melt fresh material "on the go". In one case, subsequent testing determined that the plastic wrapping on the cakes of one brand of sealant had a melting point near the upper end of the application range for the material and that this was likely the cause of the plugging. To avoid the reoccurrence of this problem, sufficient material was melted early in the morning to last till lunch break and then fresh material was melted during the lunch hour for the afternoon needs.

Several band forming tools were experimented with and the best was found to be a floor squeegee manufactured by the Fuller Brush Company, bent to form a "U" with a crescent of rubber cut out at the back to form the bead. This squeegee is different from other floor squeegees in that the rubber is moulded symmetrically around a steel reinforcement rather than being clamped onto a steel frame. The handle assembly bolts through the centre so that when one side is worn out, the squeegee can be turned over for double life. It was found that the crescent cut out in the back of the squeegee would require recutting a little each day with a pocket knife to

compensate for wear. The crescent could be kept clear of sealant by inverting the squeegee at the end of each use so that the sealant drained away.

Successful forming of the bead over the prepared crack is influenced by the viscosity of the sealant being placed. It was found with all three materials that, at the higher end of the recommended application temperature range, the bead would slump back after being formed. This problem could be reduced by allowing the sealant in the rout to cool slightly before forming with the squeegee. It is also important that the bead be centered over the prepared crack: if the bead is not formed over the groove, the material upon cooling will subside in the groove and the bead will be left standing proud beside it. This is likely to lead to early failure of the seal in winter because there is not enough material to stretch at the edge of the groove. When the desired cross-section was attained, inspection the next day after installation usually revealed the bead had flattened out except in the sections where the cracks had not been by routed or sawn.

The quantity of sealants purchased for the trials proved to be insufficient to complete all the sections. This was because the cracks were deeper than anticipated and as much as 100 mm deep for the half-metre next to the edges of pavement. The balance of the trials were sealed at a later date using Hydrotech Sealz 6160 which the District was using for its regular maintenance crack sealing work. This allows for a close comparison of performance between a standard material formulated to meet the A.S.T.M. D-1190 Specification and the "premium" products formulated to meet D-3405, selected for the trials. The "6160" was installed in 19 mm

routs treated with HCA and formed as in Fig. 4 under "normal" conditions.

The bond of the sealants to the pavement surface was inferior where the pavement was not etched by the HCA. This was determined by rolling back the edge of the sealant and attempting to peel it off. The overall bond of the three sealants to the pavement surface was evaluated at the end of the trials resulting in the following ranking in decending order:

Hi-Spec

Supergook

Cariphalte ELT.

The pavement surface throughout the trial sections was depressed 5 to 10 mm over about 80 mm each side of the pavement cracks. The sealing of these cracks with the shaped overband seal improved the riding quality of the pavement.

1982 OTTAWA TRIALS REVIEWED

December 1982

A review of the trial sections on December 7th and 8th,1982 revealed that the method of crack preparation had not as yet influenced performance. There was no splitting with any of the six methods of crack preparation employing either the Meadows Hi-Spec or the Shell Cariphalte ELT materials. There was some splitting occurring in four of the trials sealed with the Bemac Supergook.

The splitting of the seals in the first four Supergook sections may be partially attributable to an insufficient bead of material formed over the crack or cut groove. In the first three sections, in the passing lane,

various strike off devices were employed to form the seal which did not provide the desired cross-section of sealant. The bead of material left over the crack in the passing lane was insufficient and appears to be the reason for the premature splitting. The driving lane in sections 1 & 2 which had a much better bead formed with the Fuller squeegee exhibits less splitting of the seals.

The Hydrotech 6160, which was installed in the balance of the trial sections not sealed with the test materials, was not performing well. The 6160 had a "wet" appearance and had flattened out and spread on the pavement surface, completely eliminating any bead of material. Approximately 30% of the Hydrotech 6160 seals were exhibiting splitting at the edges of the routed cracks.

February 1983

In February the method of overband sealing with preparation by HCA only, demonstrated generally poorer performance over the five other methods of preparation. Also significant was a generally higher failure rate with all methods using the Supergook material which, on subsequent material testing was found to fall below the anticipated quality level. The five other methods of crack preparation with both the Hi-Spec and Cariphalte ELT materials was comparable with generally less than 10% of the seals splitting. In the balance of the sections sealed with the Hydrotech 6160, about 45% of the transverse seals were split open. Some secondary cracking next to the originally sealed crack was noted throughout the sections sealed with both the Supergook and Hi-Spec materials but none occurred in the Cariphalte sections. This is likely due to the relatively higher stiffness modulus of these two materials as compared to the Cariphalte.

In sections 2 & 8, reference nails had been set in the pavement each side of a typical crack in order to determine the amount of crack movement between evaluations. The average distance between cracks at these locations is 24 m and it was found that the opening of the cracks since the December evaluation was 8 and 9 mm respectively. This amounts to an approximate extension of 62% for a 13 mm rout and 42% for a 19 mm rout.

June 1983

On June 14th, a very warm day (30-35°C) the trials were again evaluated. The space between the nails used to monitor crack movement was found to be back at the December '82 spacing. The trial materials in all test sections now exhibited less than 10% splitting. By picking at the seals it could be determined that the healing of previously failed areas was mostly superficial with the failure still observable down 2 to 3 mm. The Hydrotech 6160 used in the balance of the trials was now 100% resealed with sealant only in the routed grooves and only a stain of material left on the pavement surface.

The three trial materials were all observed to be blistering in the overband area on the pavement surface, but no such occurrence was visible in the Hydrotech 6160 areas as the overband no longer existed.

Table 4 summarizes the splitting values for each section in the three evaluations. The splitting is rated from 0-5 with:

0 =	No splitting	3 =	20 - 50%
1 =	0 - 10% Splitting	4 =	50 - 80%
2 =	10 - 20%	5 =	80 - 100%

TABLE 4

SUMMARY OF SPLITTING VALUES

MATERIAL / DATE / METHOD	SUPERGOOK			HI SPEC			CARIPHALTE			TOTAL
	Dec. 82	Feb. 83	June 83	Dec. 82	Feb. 83	June 83	Dec. 82	Feb. 83	June 83	
No Cut/HCA	2	5	1	0	2	1	0	1	0	12
13 mm Wet Saw/Flush/ HCA	1	1	0	0	1	0	0	1	0	4
13 mm Rout/ HCA	1	4	0	0	1	1	0	1	1	9
19 mm Rout/ HCA	1	4	1	0	1	1	0	1	0	9
13 mm Rout/ Flush/HCA	0	1	1	0*	3*	1*	0	1	1	8
19 mm Rout/ Flush/HCA	0	1	0	0	1	1	0	2	1	6
TOTAL	24			14			10			

* Atypical pavement in this section.

	6160		
19 mm Rout/ HCA	3	3	0

The totals under the materials columns indicate the overall performance of the materials whereas the totals opposite the methods indicate the overall performance of each method. It is obvious from Table 2 that the materials have a significant influence on performance regardless of installation method.

The poorest performance was observed where cracks were sealed without routing or sawing.

The most successful method employed the wet diamond saw to cut the cracks, followed by water flushing. The likely reason for the higher success rate with this method is that the diamond saw cuts clean, by abrasion and does not jar loose aggregates in the side of the cut as do routers. Even though crack preparation by sawing has a demonstrated advantage over routing, it may not be viable method due to the extremely poor productivity of this method.

The water flushing of routed cracks appears to have some benefit particularly when combined with the larger rout of 19 mm. It is estimated that this would add up to fifteen cents a metre to the cost of routing and sealing.

Since the first three sections installed with Supergook did not have the desired cross-section of seal formed it may not be correct to include this material in the analysis. Table 5 reflects the results with the Supergook deleted.

TABLE 5

SUMMARY OF SPLITTING VALUES WITH SUPERGOOK DELETED

MATERIAL / DATE / METHOD	HI SPEC Dec. 82	HI SPEC Feb. 83	HI SPEC June 83	CARIPHALTE Dec. 82	CARIPHALTE Feb. 83	CARIPHALTE June 83	TOTAL
No Cut/HCA	0	2	1	0	1	0	4
13 mm Wet Saw/Flush/ HCA	0	1	0	0	1	0	2
13 mm Rout/ HCA	0	1	1	0	1	1	4
19 mm Rout/ HCA	0	1	1	0	1	0	3
13 mm Rout/ Flush/HCA	0*	3*	1*	0	1	1	6
19 mm Rout/ Flush/HCA	0	1	1	0	2	1	5
TOTAL	14			10			

* Atypical pavement in this section.

In this analysis, the wet saw method again appears to have the best performance with the standard 19 mm rout and HCA placing second. The water flushed routs now appear to have the lowest success rate rather than the no cut method. In both the "dry" and water flushed methods, there appears to be a slight advantage to the 19 mm rout over the 13 mm.

MATERIAL SAMPLING AND TESTING

Samples of the materials delivered to the test sections were obtained and tested in the Downsview Laboratory for:

Cone Penetration	@ 25°C	C.G.S.B. 37-GP-50M
	@ 50°C	C.G.S.B. 37-GP-50M
Flow	@ 60°C	C.G.S.B. 37-GP-50M
Resilience original	@ 25°C,	A.S.T.M. D-3405
	oven aged	
Bond, 5 cycles	@ -17.8°C	A.S.T.M. D-1190 Mod.
Cold Bend	@ -25°C	C.G.S.B. 37-GP-50M Mod.
Toughness		C.G.S.B. 37-GP-50M.
Peak Force		
Relative Density		

The most interesting finding from all this testing was that of the four "best performing" materials in the Hwy.69 trials, only the Meadows Hi-Spec met the MTC bond test. The industry contention that the bond test precluded suitable materials from use by the Ministry would appear to be valid. No correlation could be found between test values and field performance. Table 6 contains the test values for the best four performing materials in the Hwy.69 trials.

TABLE 6

SUMMARY OF TEST VALUES (HWY. 69)

Product	Cone Pen. @ 25°C	Cone Pen. @ 50°C	Flow	Cold Bend	Tough.	Peak Force	Ratio	Rel. Den.	Resil Orig.	Resil Aged	Bond
Supergook	64	102	0	Pass	19.8	271.0	0.073	1.35	84	80	Fail
Caraphalte	77	177	0	Pass	11.2	166.7	0.067	1.34	73	73	Fail
6165	105	139	0	Pass	7.3	160.2	0.046	1.13	73	72	Fail
Hi-Spec	92	160	0	Pass	14.9	203.5	0.073	1.17	56	59	Pass

The test values in Table 6 are also typical of many of the materials which did not perform well in the trials.

The laboratory testing of the Supergook supplied to the methods trials in Ottawa revealed that it was not the same as supplied to the materials trials on Hwy. 69. The Ottawa material was much stiffer with a lower toughness/peak force ratio. The generally poor performance noted in the Ottawa trials is probably due to these very differences.

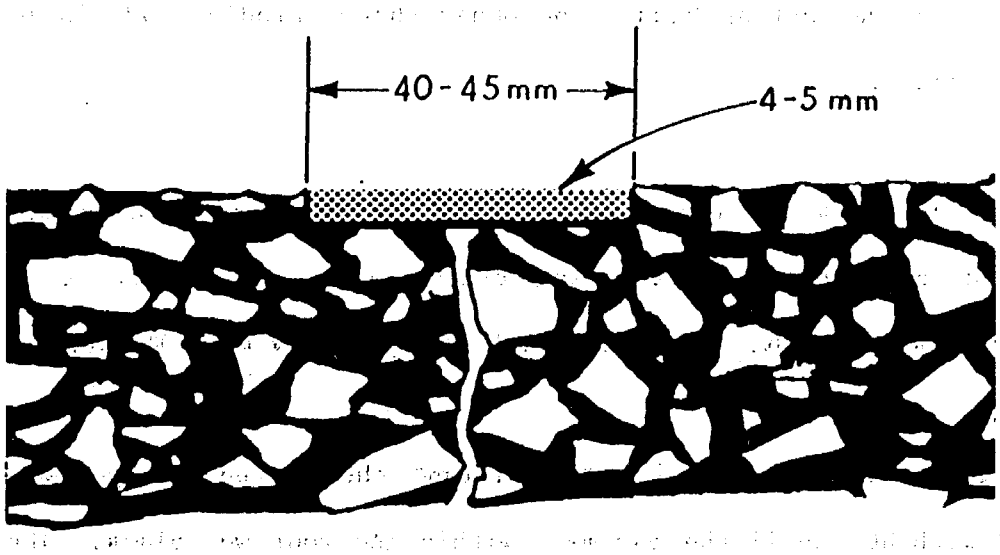
1983 PROGRAM

Although the convex overband seal proved to have superior performance, it was not generally accepted for aesthetic reasons. There were many objections to the wide black band on the pavement surface which drew attention to the pavement cracks. It is also subject to some snowplow damage at high points in distorted pavements, such as between wheel track ruts.

In an effort to maximize the benefit of the overband seal in combination with a controlled width of spread and the elimination of snowplow damage, a new geometry of rout was experimented with in 1983. This method, introduced by Klaruw of Holland to the Ministry, employs a shallow channel 40 mm wide and 5 mm deep cut in the pavement and filled flush with sealant. Countersinking the overband into the pavement eliminates the need for a strike-off operation. See Figure 6.

FIGURE 6

CONTERSUNK OVERBAND SEAL



Location

This method was constructed in two locations as part of a crack sealing contract:

1. Hwy. 400 southbound at Hwy. 89.

2. Hwy. 11 between Hwys. 117 and 118 near Bracebridge.

The Hwy. 11 location has a similar environment to both the Hwy. 69 and Ottawa locations and the Hwy. 400 location has a slightly less severe environment.

Construction

At the Hwy. 400 location two materials were used:

1. Southbound passing lane - Hydrotech 6160.

2. Southbound driving lane - Meadows 164.

The Hwy. 11 location was constructed entirely with Tremco THO 2051.

The contractors employed Tennant routers set up with two cutters spaced approximately 40 mm apart (outside measure) on every other of the six spindles on the cutting head. The other three spindles had single cutters set at centre.

The routing depth was found to be quite variable and usually deeper than required and with the meandering cracks often off centre of the rout. These problems can be associated with the geometry of a Tennant router.

Immediately ahead of placing the sealant the routed grooves were conditioned with HCA until the pavement within the rout was black. The sealant was then poured into the groove such that it bulged a little at the rout edges so that upon cooling and shrinking it was flush with the pavement surface.

1983 Trials Reviewed

The trials were evaluated three times over the winter of 1983-84 and all found to exhibit less than 10% splitting. Considering that the materials used were of standard quality (ASTM D-1190) and not high spec., this performance is excellent. The splitting that occurred was almost all where the edge of rout coincided with the crack below, i.e. where the crack was not centered in the rout.

Commentary

This method exhibits the following advantages over the convex overband seal:

1/ It is easier to follow the cracks when routing as the wider cut allows more latitude.

2/ The wide cut surface is easy to treat with HCA to improve bond as opposed to the standard 19 mm.

3/ Uses less than 50% of the sealant required for the shaped overband seal with 19 mm routing.

4/ The squeegee operation is eliminated.

5/ Allows for snow plowing without skipping of the one-way plows.

6/ Snow plows cannot pull at the sealant and break the bond.

7/ Longitudinal seals do not act as dams to the transverse flow of salt brine used to clear the pavement.

8/ Better overall performance.

OBSERVATIONS AND CONCLUSIONS

Equipment

1/ Tennant routers are highly productive on straight line cutting but are limited in their ability to follow meandering cracks.

2/ Crafco routers are lower in productivity than Tennant routers on straight line cutting but have the ability to follow most meandering cracks.

3/ Vertical spindle (Windsor) routers can follow any crack pattern but have very low productivity.

4/ Routers cut by impact and leave loosened aggregate in the cut faces of the rout.

5/ The Runway Crack Cutter (diamond saw) cuts very cleanly and can follow most cracks but has extremely low productivity.

6/ Melters with automatic temperature controls are an advantage in minimizing overheating of the sealant.

7/ Back pack blowers have limited ability to clean out routed grooves.

8/ The HCA lance is very effective in cleaning and drying routed or other

wise prepared cracks.

9/ The HCA lance permits work to proceed successfully in damp conditions.

Materials

1/ The cold poured sealants that were tried are not satisfactory for sealing asphalt pavement cracks on highways.

2/ Four hot poured rubberized asphalts formulated to meet A.S.T.M. D-3405 perform satisfactorily (with less than 20% splitting) during winter in locations with mean daily January temperatures -10°C:

Shell Cariphalte ELT,

Bemac Supergook,

Meadows Hi-Spec,

Hydrotech Sealz 6165.

3/ Four hot poured rubberized asphalts formulated to meet ASTM D-1190, perform marginally (with less than 50% splitting) during winter in locations area with mean daily January temperatures of -10°C:

Bakelite 590-13,

Hydrotech Sealz 6160,

Meadows 164R,

Paraseal 2065.

4/ Sealants with a low stiffness modulus, standing proud of the pavement surface, are prone to be removed by plows in the winter if not well bonded.

5/ Sealants with a high stiffness modulus standing proud of the surface are usually sheared off by plows in winter.

6/ Well bonded sealants with a high stiffness modulus will occasionally induce secondary cracking of the pavement close to the original sealed crack.

7/ The currently employed laboratory tests do not give an indication of a materials ability to perform satisfactorily in asphalt pavements during the winter.

8/ Laboratory tests which will more closely reflect sealing materials ability to perform in asphalt pavements must be identified.

Methods

1/ Moisture or dirt in the routed groove at the time of placing the hot sealant will lead to early failure of the sealant in winter.

2/ Sealant placed flush or slightly low in the routed grooves is prone to early failure through splitting at the edge of the rout during the winter.

3/ Sealing of narrow cracks ($\leq$ 8 mm) without routing can be expected to have poor winter performance.

4/ Sealant installed so as to overlap onto the pavement surface on each side of the rout has a higher success rate, particularly if the seal is formed with a convex shape over the rout.

5/ Sawing cracks instead of routing improves winter performance.

6/ Sealant placed in 19 mm routing performs better than in 13 mm routing.

7/ Water flushing of routed cracks prior to treatment with HCA is of dubious benefit.

8/ The use of hot compressed air to condition the prepared cracks, immediately ahead of sealing, significantly increases the bond of hot poured sealants to the pavement.

9/ Sealant placed in a contersunk 45x5 mm overband performs best.

SUMMARY

Mid-winter evaluation of crack sealing is essential in determining sealing effectiveness. Typically seals start failing through splitting in December and reach maximum distress around the end of January. With rising temperatures and traffic, most sealants will reseal over summer with some resealing as early as the end of March. This cycle will be repeated each year, with the seals appearing effective every summer.

Of the 22 products evaluated for sealing asphalt pavement cracks, only 8 performed well enough to be considered for use in Ontario's highways. All eight products are hot poured rubberized asphalts.

Based on mid-winter performance and minimal loss of sealant, four "premium" materials were approved by the Ministry for use in asphalt pavements throughout Ontario and a further four materials are considered acceptable for use in the southern part of the provinde only. Cold poured products were not found acceptable for use.

Routed cracks have to be both clean and dry at the time of sealant placement or premature failure is almost assured.

The use of hot compressed air to condition prepared cracks was found to enhance the bond of hot poured sealants and also permit work to proceed in damp conditions. HCA is now extensively used by the Ministry in its crack sealing projects.

A 13 mm rout combined with a convex overband seal is considered adequate for the southern part of Ontario but a 19 mm rout is essential for the north where crack movement is more extreme.

A countersunk 45 x 5 mm overband seal exhibits the best all round performance.

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Thin Concrete Block Overlay

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ABSTRACT

This paper introduces a new method of paving with thin concrete blocks and an asphalt adhesive.Discussion will cover theoretical and experimental works on the method.

Roads in Sapporo experiences rather severe climatic conditions,ranging from -15 degree C in winter toward 70 degree C in summer.Such extremes in temperatures are responsible for the strain on roads.Pavements also take a beating because of studded tires of vehicles in winter.Therefore even fine random cracks can be the cause of serious troubles.

Concrete ,if used as is, carries its own set of problems.The proposed remedy discussed here is to reduce the concrete surface down to an orderly mosaic of small and thin blocks,applied directly to the existing asphalt road surface with an asphaltic adhesive.The size and method of placement are critical to the success of this system,and set it apart from conventional overlay construction technology using concrete.

The optimal size for blocks in this system rests on three main factors ---the thermal cover effect,mechanical strength and handling costs.

Theoretical and experimental works indicate that random cracks in a thin concrete overlay on the existing asphalt concrete pavement can be avoided by sizing the blocks according to their thickness and the foundation--- 'Pre-fracturing' before loading can prevent such form of destruction.

A minimum thickness for the blocks can be determined in relation to the effects of thermal cover in order to avoid the extremes in temperatures in pavements.The asphalt adhesive can also prevent water seepage into the foundation.This is important in crack prevention.

Since both a block and an adhesive layer are thin,the degree of evenness and the transverse profile depend on the evenness of the existing surface.

THIN CONCRETE BLOCK OVERLAY

by

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1. Preface

In the cold regions of Japan, major roads generally are paved with asphalt. In Hokkaido alone, over 70 % of all roads are covered with asphalt. Construction of new roads is on the decline, but existing roads are difficult to maintain and repair, largely because of wear and tear from climatic conditions and studded tires.

Until recently, concern over damage to asphalt roads was limited to the cost of road repair; recently concern has centered also on environmental pollution created by the dispersion of asphalt dust.

The problem has been exacerbated by the growing use of larger vehicles which expose road surfaces to heavier loads for longer periods of time at higher temperatures, creating deep ruts. These crevices, in return, can result in cracking at low temperatures.

The problem of asphalt road has become a focus of concern for many organizations. The obvious alternative, rebuilding roads from the subgrade up, can be financially prohivitive.

This report will introduce another method of road restoration, covering the existing asphalt with a layer of precast, thin concrete blocks. Cost considerations aside, retaining the original asphalt layer as it is is desirable since it has proven durability from repeated use over time. Moreover, asphalt subjected to years of use has a value of its own since its bearing capacity has been established through actual use.

Why concrete overlay?

Covering the road with a thin layer of concrete blocks not only shield the asphalt from wear and tear, but also insulates against temperature fluctuation, reducing thermal stress on the asphalt. Rutting in summer and cracking in winter can be avoided with benefits for mix proportion reducing the layer thickness.

This study will focus on how to determine the optimal block size and how to assess the temperature distribution in order to design the most cost-efficient overlay.

2. Winter traffic problems in a cold region

Many Japanese cities must cope with the problem of trying to move traffic over icy roads as efficiently as during warm weather. The increase in volume and speed of traffic has made environmental damage seem necessary evil, since the only solutions until now have been using studded tires or applying defrosting chemicals like salt to the roads.

Both methods have their strong and weak points. Since Japan has opted for the former solution, it has faced the accompanying problems of road detrerioration and environmental damage:

①。 Road surface abrasion

a. Unevenness in the road surface impairs safe and comfortable driving

b. When filled with water, dents in the surface can cause the hydroplane phenomenon"

c. Rubbed-out speed, direction and other signs printed on the surface pose traffic hazards .

d. Ambient dust created by asphalt erosions leaves dirt deposits on the road surface and is a source of air pollution, a health hazard

②。 Studded tire abrasion

a. Various kinds of metal flouer can be absorbed into the animate and human bodies

b. When dry, studs are slippery and can make driving hazardous

c. Studs slow wheel rotation, increasing fuel consumption

d. Studs make for a bumpier, noisier ride

③。 Other problems

a. Frequent road closings for maintenance are a public nuisance

b. Increase of anonymous offences such as road damage due to studded tires can spoil the morale or the solidarity of the society

d. Frequent road maintenance is a costly drag on municipal budgets

Yet chemical defrosting agents have many drawbacks:

①。 Chemicals cause vehicles to rust more quickly

②。 The iron infrastructure of bridges and buildings can also rust

more quickly, an obvious hazard

③。 The use of chemicals, over time, has deterious effects on roadside greenery

④。 A sudden snow fall requires mechanical snow removal

⑤。 Special sewage treatment facilities must be installed to process the defrost run-off

Faced with such equally unatractive options, it is difficult to choose the least offensive. Japan has selected studded tires because of limited space, the incidence of heavy snawfall, a population concentrated in urban areas, the proximity of agricultural land to roads. Even in countries without Japan's natural disadvantages, chemical defrosting has started to decline in appeal.

In Sapporo and its environs, wear on roads, and on studded tires, is more pronounced where ice and snow are the least concentrated, as on busy roads, and in underpasses and tunnels, where snow and ice seldom reach. Abrasion is rapid right before, and after, periods of snowfall, since at those times studded tires have already been installed but the road surface remains directly exposed.

Sapporo vehicle owners use studded tires for nearly six months of the year, though evidence shows the tires perform better than snow tires only when the road surface is frozen and the ice temperature falls between -1 degree C and -7 degrees C. Using snow tires, city drivers could dispense with studded tires altogether, and the number of drivers overall who actually need studded tires is less than commonly believed; the situation is such that the number of occasions that actually call for studded tires is far fewer than common wisdom woud have it. A preferable alternative would be to install a stronger studded tire, but only when actually necessary. The end result would be less wear on roads throughout the winter.

Inserted into the ice, studs can work by increasing brake effectiveness on a limited area. The work energy required to brake and accelerate is considerable. Stud effectiveness depends on breaking the ice surface, so road surface erosion is inevitable.

Brake drums and linings in a car or locomotive, when worn can be replaced relatively easily. Ideally, a road surface should be comprised of a stretched lining which also can be replaced when worn out. A surface

of bricks and blocks constitutes this kind of replaceable lining. Using a combination of stronger studs and concrete-overlaid roads may be the answer to road wear problems in winter. Stronger studs would probably be safer than the smaller type in use now, particularly on steep grades or at intersections. At these higher-risk areas, using concrete blocks would overcome the increased problems of abrasion resulting from heavier studs.

3. An overlay of thin concrete blocks

Main reasons why concrete has not been predominant in Japanese cold region are said to be as follows:

①Conorete pavement is too rigid and cannot adjust to shifts in the contour of the foundation as a result of frost heave

②Subgrade, designed to resist frost heave, is thick and strong enough to bear and distribute loads so that concrete pavement is not necessary

③Repairing thin wear of concrete due to studded tires is difficult

④The long construction time required for installing a concrete pavement is problematic, since poor weather limits the time available for road work

⑤The quality and strength of concrete placed at the site can vary according to machinery and level of workmanship

⑥Expansion joints inevitable in concrete roadwork make for a bumpier ride

Asphalt pavement, of course, has its own set of disadvantages:

①The pavement is difficult to see in poorly lit places in fog or at night

②As mentioned, asphalt is vulnerable to road abrasion, with air pollution as a result

③Deformation of the road surface and staining, caused by asphalt's low viscosity at high temperatures

④Cracking at low temperatures

Concrete's higher resistance to stud erosion is causing it to be re-examined as a road material. It has a number of other merits as an overlay, (Ref. 1). Most of the problems mentioned in reference to asphalt

can be solved by using a layer of thin concrete blocks on top of the asphalt subgrade.
Miniature blocks can easily and quickly replaced.Provided the blocks are as strong and well-produced as concrete,right-of-ways can remain open during road work,since the curing process is not necessary.Since the blocks are small,the width of the joints can be reduced, cutting down on the noise created by driving over them.
Empirical limit for surface wear depth is about 3 cm.Blocks should be replaced when they erode to this depth.
Each block should be firmly attached to the existing asphalt surface to prevent it from flying off or scattering,since the effect of car passing over creates a strong suction.Adhesive should be fast-acting, strong,and easy to remove when necessary.

4.Block strength

Block strength depends on a number of factors,including size and condition of the foundation(Ref.2).The most adverse condition is when the block is pressed by a solid rod on which the wheel weight P, the product of tire pressure p and the contact area bw,is a concentrated load.
Cracks usually appear in the center of the block,so strength is calculated from the stress at the center,which varies with the condition of foundation,particularly the adhesion between the block and the base.
A block on an elastic foundation results in a stress of tension;a block united with the foundation creates a stress of compression.
When a solid substance manages to penetrate the adhesive membrane,the most adverse condition is when a solid acts as a simple beam supported at both ends.

①. block on elastic foundation

Calculation procedure is illustrated in Fig.1.The stress is calculated by the iteration method.The deformation and section forces are calculated under the assumption that the block is infinitely long.The differential equation with deformation y is as follows:

$$d^4y/dx^4 + 4 v y = 0 \cdots (1)$$

where $v = \sqrt[4]{(cw/4EI)}$

Boundary conditions are:

when $x = 0$, $dy/dx = 0$ and $dy^3/dx^3 = 0$

when $x = \infty$, $y = 0$

Consequently the first approximations are as follows:

$y = (w\,p/2c)\cdot\exp(-vx)\,(\cos vx + \sin vx)$ ...(2)

$M = (bwp/4v)\cdot\exp(-vx)\cdot(\cos vx - \sin vx)$ ...(3)

$Q = (bwp/2)\cdot\exp(-vx)\cos vx$ ...(4)

where M is a bending moment and Q the shear force.

A bending moment and a shear force calculated from the above equations should be zero at both ends, since the block length is finite. Therefore the bending moment and the shear force with negative sign are applied to both ends of the block to calculate the deformation. The vertical components of forces must be in equilibrium. Any imbalance should be adjusted by the iteration procedure.

②block united with foundation

It is postulated in the proposed method that blocks are firmly attached to the foundation. The most adverse case for compression at the bottom of the block is shown in Fig. 2, in which blocks act as if they were extensions of the foundation, and bear the load on the wedge-shaped core. The stress may be calculated from Airy's stress function Φ, as follows:

$$\nabla^4 \Phi = 0$$

Consequently $S_r = (b/a)\cdot\cos^2 q\cdot(-2pb)/(2t + \sin 2t)$ ...(5)

where $t = \tan^{-1}(a/2b)$

When $q = 0$, the maximum stress is

$$S = -2p\cdot(b/a)/(2t + \sin 2t) \quad \text{...(6)}$$

③when hard granules enter the adhesive layer

Assuming that the load per unit length is p and the block behaves as a simple beam supported at both ends of the block, the maximum fibre stress is as follows:

The maximum fibre stress S is $S = (3/4)p\,(b/a)^2$ ....(7)

Actual conditions fall between the two extremes mentioned above. Fig. 3 shows that stress values vary with the flatness ratio in one example in which the block thickness is about 2 cm, $p = 5.5$ kg/cm^2 and $c = 1000$kg/cm^3.

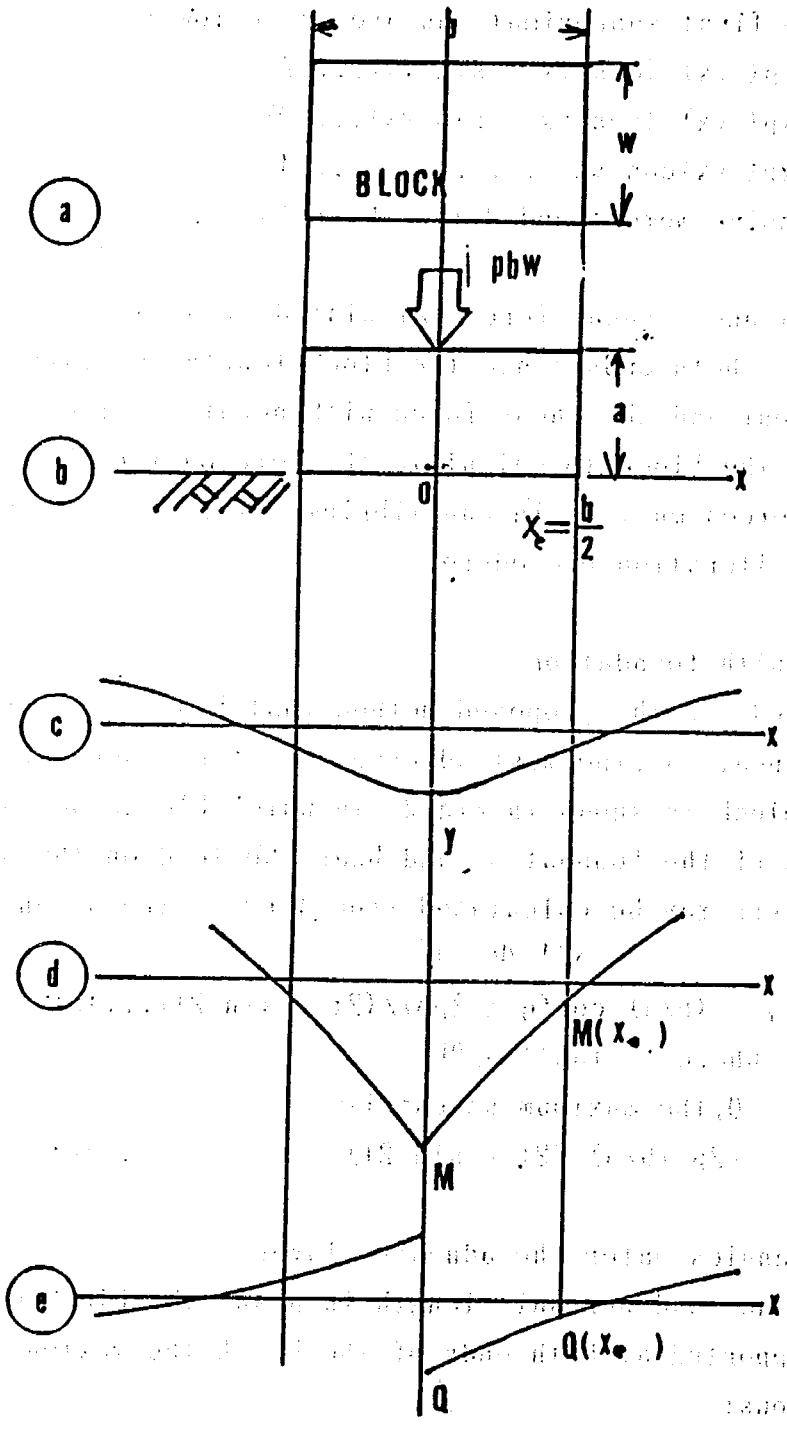

Fig.1 Deformation of block on elastic foundation and section forces

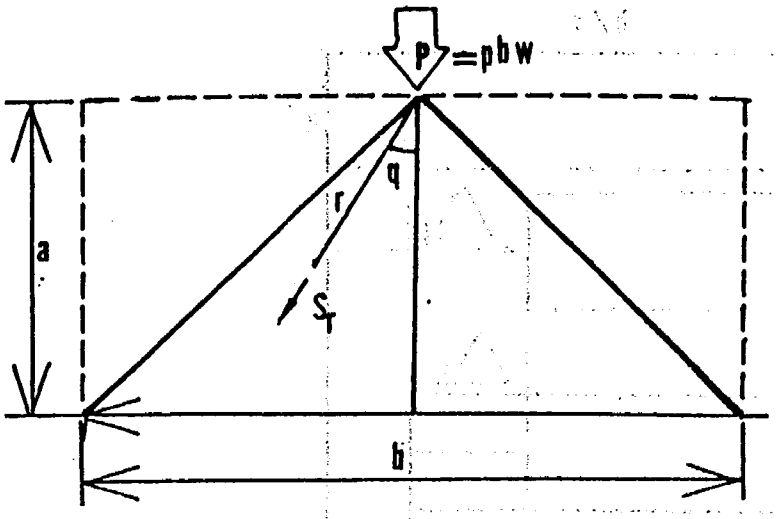

Fig.2 Stresses of block united to foundation

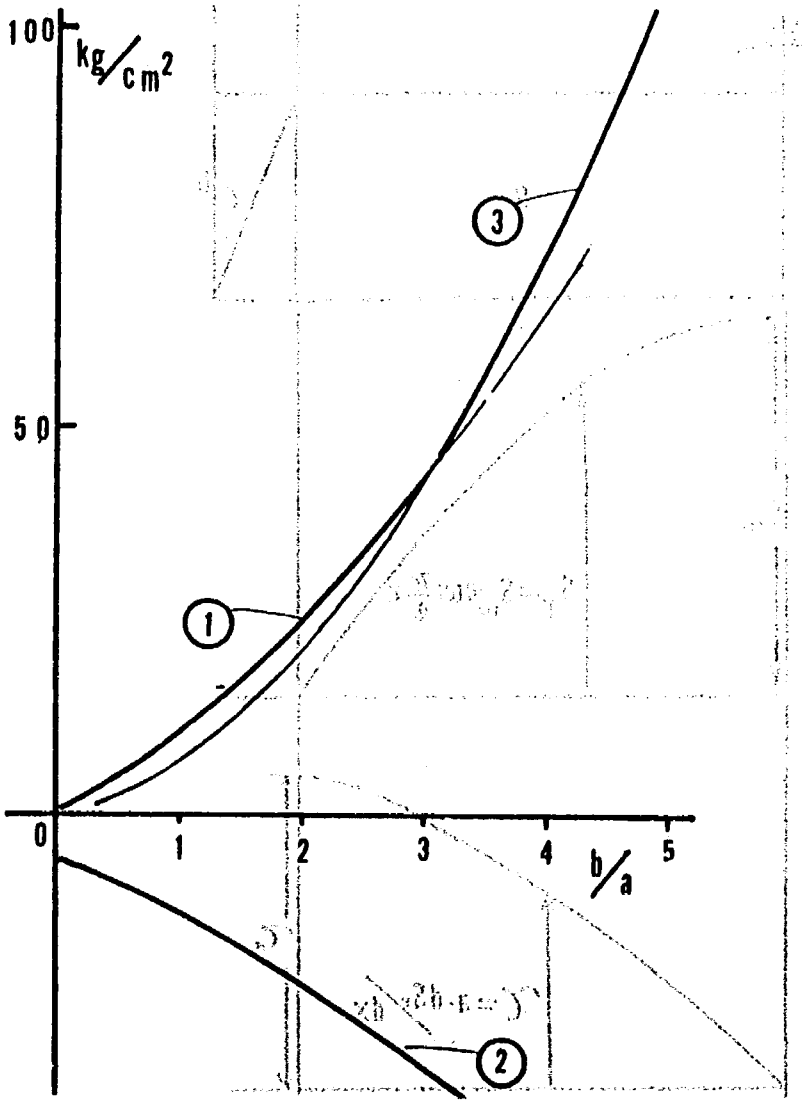

Fig.3 Flatness ratio and principal stress

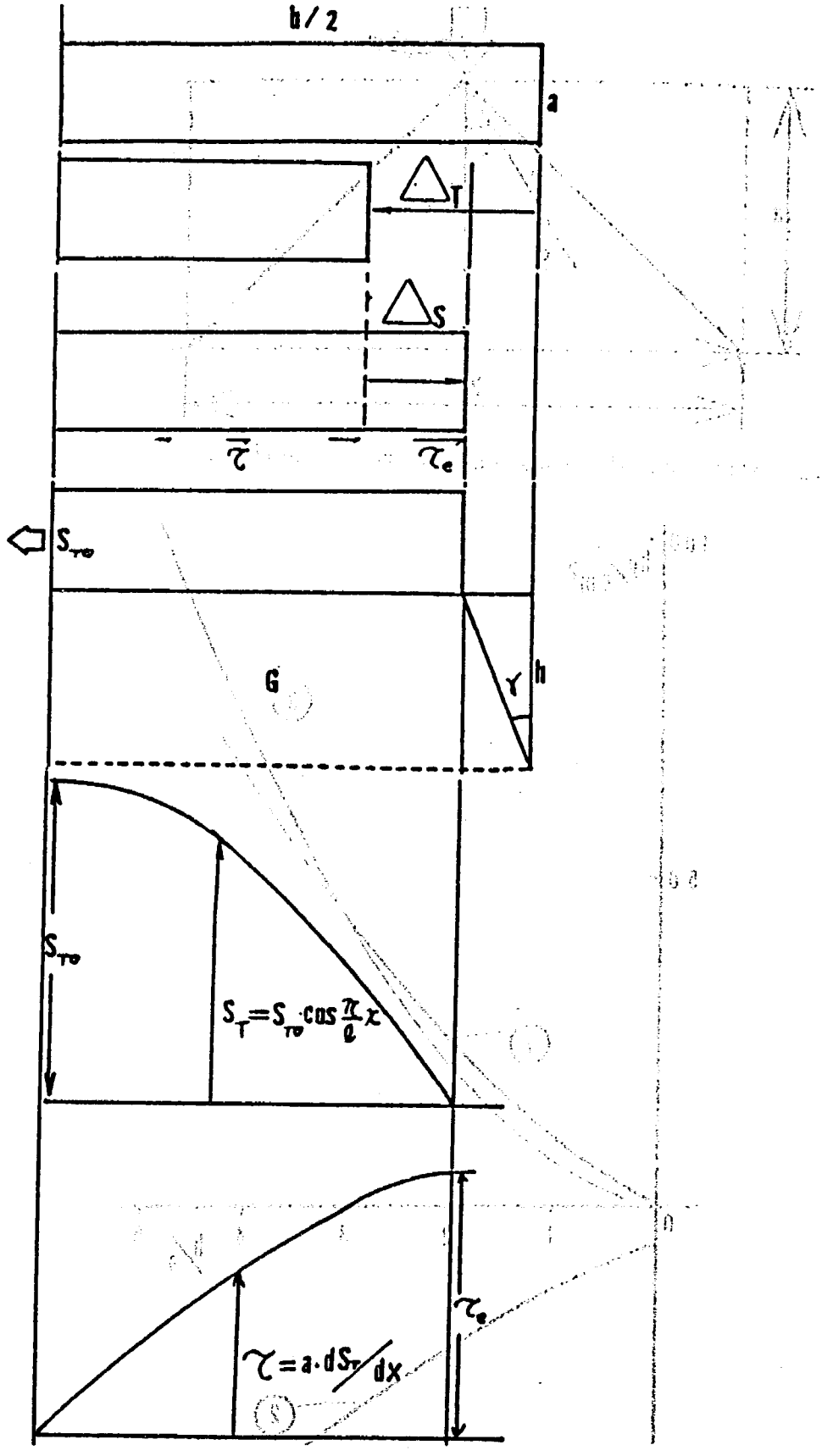

Fig.4 Block size and thermal stress

④Effects of adhesive

The adhesive should be strong enough to prevent from flying into the air. However, when the adhesive acts against thermal deformation, it can cause thermal stress. It is assumed that thermal stress is distributed as shown in fig. 4. The equation is

$$S_T = S_{T0} \cos(\pi x/l) \quad \cdots\cdots\cdots (8)$$

The block centre does not move since the distribution is symmetric. The free deformation of the block end is

$$\Delta_T = \varepsilon \cdot dT \cdot (b/2) \quad \cdots\cdots\cdots (9)$$

where dT is the temperature difference between the block and the foundation.

The shear stress at a given point can be derived from the equilibrium equation.

$$\tau = -a\, dS/dx \quad \cdots\cdots\cdots (10)$$

The shear restraint can shove back the block end by

$$\Delta_S = \int_0^{a/2} (S_T / E)\, dx \quad \cdots\cdots\cdots (11)$$

Consequently at the block centre,

$$S_{Tc} = (\pi \varepsilon \Delta T \cdot E/2) \{1 - \eta/(b^2 + \eta)\} \quad \cdots (12)$$

where $\eta = (h/G) \cdot \pi^2 E a$

If the shear is given as the friction force τ_y

$$S_{T0} = (\tau_y / \pi) \cdot (b/a) \quad \cdots\cdots (13)$$

Therefore that the flatter the block, the stronger it is. When a block is weak, it will be broken into smaller pieces until its flatness ratio stabilizes.

5. Cover effects of blocks

Experiments were conducted to measure temperature distribution in the existing asphalt, under the blocks. The thicknesses of the blocks are 0 cm (bare asphalt), 2cm, 4cm and 6cm. Temperature was measured by thermocouples distributed in the asphalt and recorded continuously on a plotter. The layer structures and the mix proportion of asphalt and concrete were kept constant. The block specimens were sliced to specified thickness using a cutter, and attached to the asphalt surface using an adhesive membrane 2 mm thick and made of straight asphalt. There are two buildings at the experimental location to the east and

the north. Specimens were horizontal and exposed to sunlight from the south.

a. summer

Asphalt rutting due to heat may be mitigated by covering it to prevent a rise in temperatures. Measured results of temperatures are shown from Figs.5 to 8, recorded on Sapporo's hottest day of 1983. When the block thickness is less than 2 cm, the inversion of temperature distribution is clearly linked to the circadian rhythm. The inversion can alternatively cause compressive and tensile stresses. However, the cracking is reduced in summer, since the temperature gradient is less steep during cooling than during heating, and the elastic modulus is low. The temperature of bare asphalt can rise to 68℃ and rutting can occur particularly during prolonged traffic loads. The relationship between the block thickness and the highest temperatures for asphalt is shown in Fig.9. As is shown, the the highest temperature for asphalt is 48 ℃, but cooler by 20℃, under 2cm thick blocks, which can help prevent asphalt rutting. The cover blocks may permit the use of more workable asphalt and a low softening point adhesion. Consequently thin blocks are a simple means of preventing rutting.

b. winter

The circadian rhythm of asphalt temperatures on the surface, measured on a day when the temperature plunged to -15℃ at night, is shown in Figs.10 to 13. The temperature differential between the midday high and the midnight low can be reduced considerably by using a cover of thin blocks. Reducing the fluctuation in temperature can prevent shrinkage, and consequently cracking. The highest asphalt temperatures can be reduced by blocks as shown in Fig.14.

6. Discussions

The maximum flatness ratio, for safety reasons, of a thin block subjected to wheel weight in the most adverse condition, is noted in Fig.3. Concrete with the uniaxial compressive strength of 700kg/cm^2 and the tensile strength of 70kg/cm^2 is applied to the example. Assuming that the thickness is 2 cm, the flatness ratio can be about 4. Thus the block scale can be 8cm x 8cm x2cm thick. In order to examine the actual block strength, the maximum side lengths of the specimens were 6, 10, 12 and 24 cm. All blocks were placed along the wheel lane, attached to the existi-

ng asphalt surface with a 3mm thick asphalt membrane. The adhesion is 6 kg/cm² at 20℃. These specimens were subjected to actual winter traffic of about 15,000 cars with studded tires per day for six snowy months(approximately 2,700,000 vehicles total) and also subjected to high temperatures for 8 months in summer. No fractures were found except on 24cm long blocks,which may prove that the adverse conditions mentioned above may be too severe for actual conditions. The flatness ratio thus obtained may be overly cautious.Since the overlay fracture depends on the flatness ratio,the fracture growth will cease when the ratio falls below the critical figure.Therefore,random fractures can be avoided by "pre-fracturing" the concrete overlay.The thin blocks are a concrete slab reinforced by pre-fracturing in a well-organized way.(see Ref.3)

In the traditional block practice,the larger the side length,the less the cost of construction per area.Block thickness was listed considering the fact that a block should not be fractured on sand cushion.The thickness can be reduced considerably ,if the thickness is given from the allowable wear depth or the cover effects.In this way,however,the side length has to be shortened to keep the flatness ratio below the critical level. Therefore,these blocks may be combined to form larger panels. Fibers of various materials should be experimented with,as a means of combining small and thin blocks to make such panels in the future.

The asphalt membrane can serve not only as the adhesive, but also as the impermeable layer.However,this membrane asphalt should have a high apparent viscosity resistant to high temperatures in summer.

7. Conclusion

①Both theoretical and the experimental work show that random cracks can be prevented by keeping the flatness ratio under a figure given for conditions such as the block thickness and the foundation coefficient.The maximum flatness ratio found in Fig.3 appears sufficient.

②The cover effects of concrete overlay may prevent rutting in summer and cracking in winter resulting from temperature changes.The highest temperature for asphalt can be reduced at a rate of 10 ℃/cm with the thickness of less than one inch.

③An adhesive membrane,with an adhesive strength of over 2 kg/cm² even in midsummer, enables easy and quick construction.In addition,the mem-

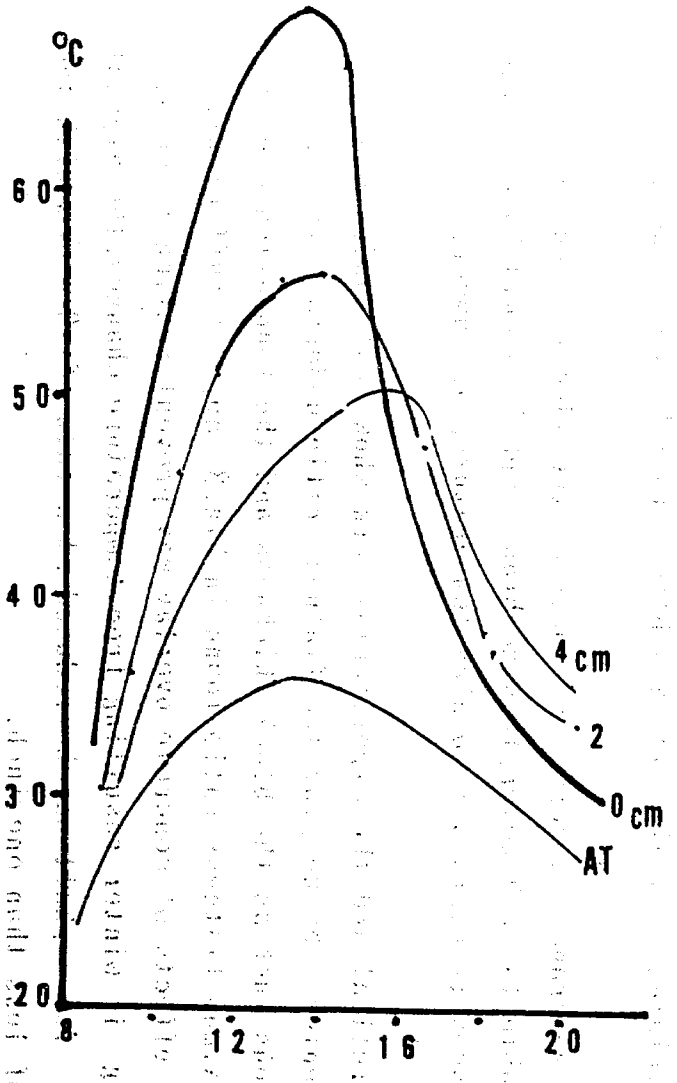

Fig.5 Temperature change in bare asphalt in summer

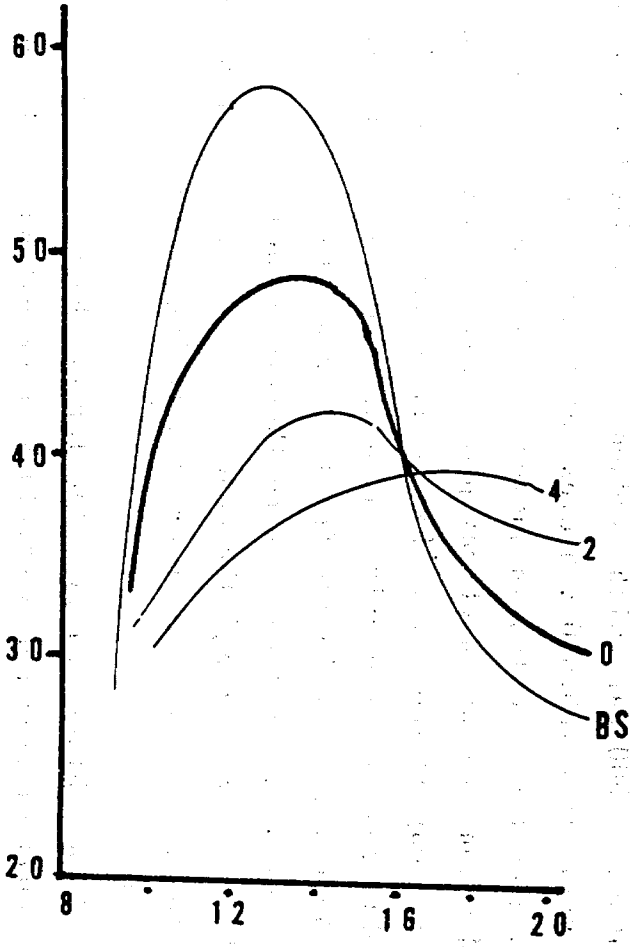

Fig.6 Temperature change in asphalt with concrete less than 2cm thick in summer

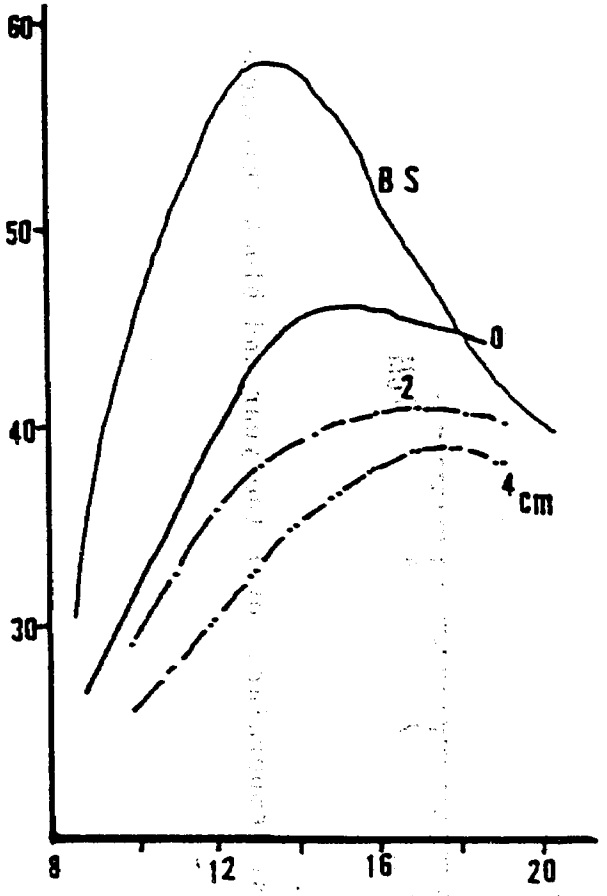

Fig.7 Temperature change of asphalt with concrete less than 4cm thick in summer

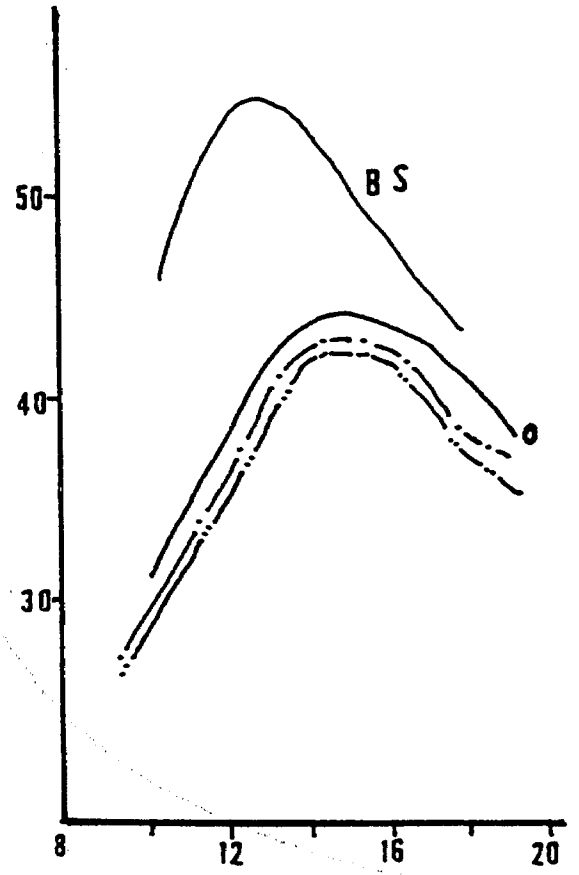

Fig.8 Temperature change of asphalt with concrete less than 4cm thick in summer

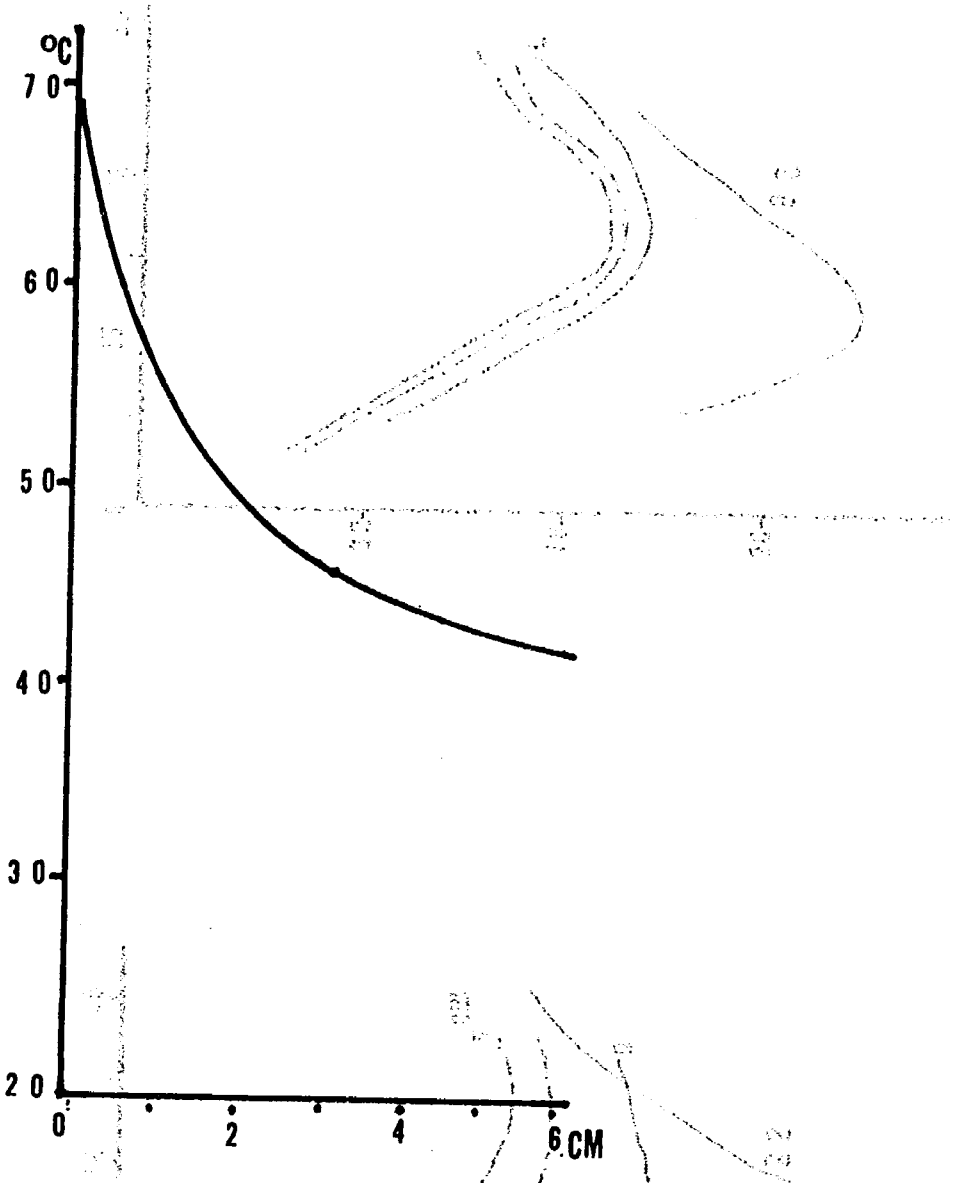

Fig.9 Highest temperatures over the year and block thickness

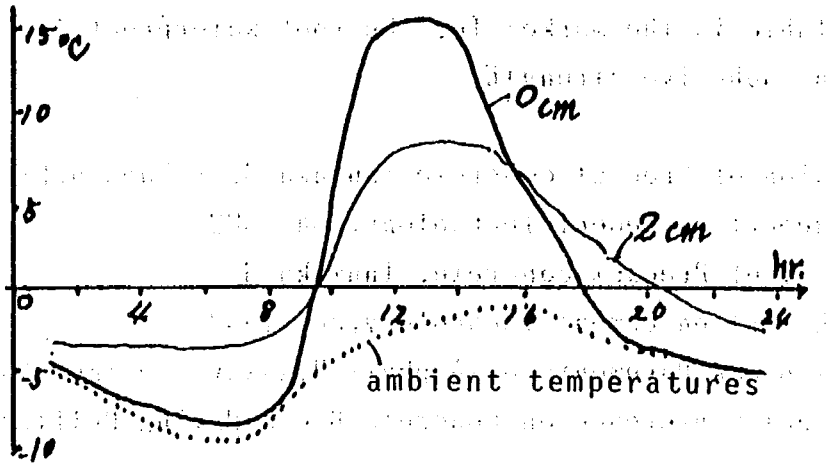

Fig.10 Temperature change in bare asphalt, in winter

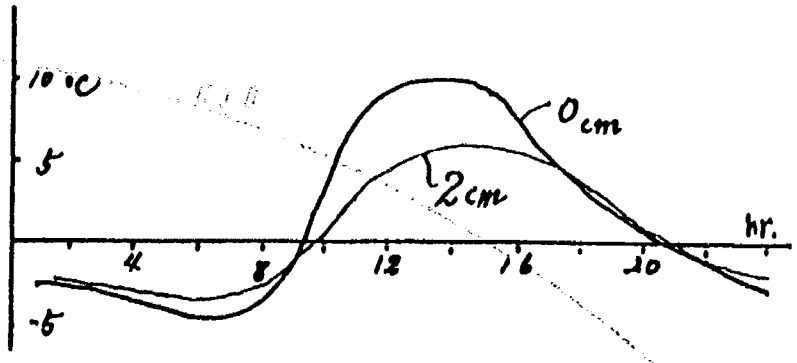

Fig.11 Temperature change in asphalt with concrete layer less than 2cm thick in winter

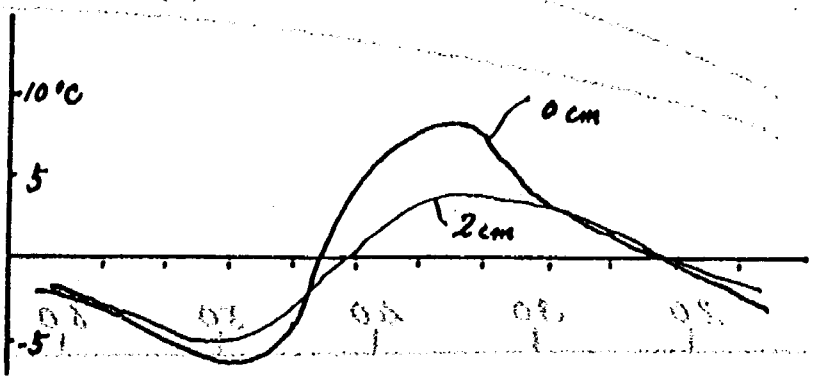

Fig.12 Temperature change in asphalt with concrete layer less than 4cm thick in winter

brane can form an impermiable layer beneath the blocks. The asphalt membrane, available in the market for the roof waterproof, is generally of insufficient adhesive strength.

Ref.1 On Abrasion of Precast concrete. Inuzuka, M. & Yamaguchi, T.
Annual Report on Cement Technology, JCA, 1982
Ref.2 On Overlay of Precast concrete. Inuzuka, M.
Annual Report on Cement Technology, JCA, 1983
Ref.3 Concrete Block Pavements on Asphalt Highway. M. Inuzuka & I. Sato
2nd International Conference on Concrete Block Paving/Delft/April 1984

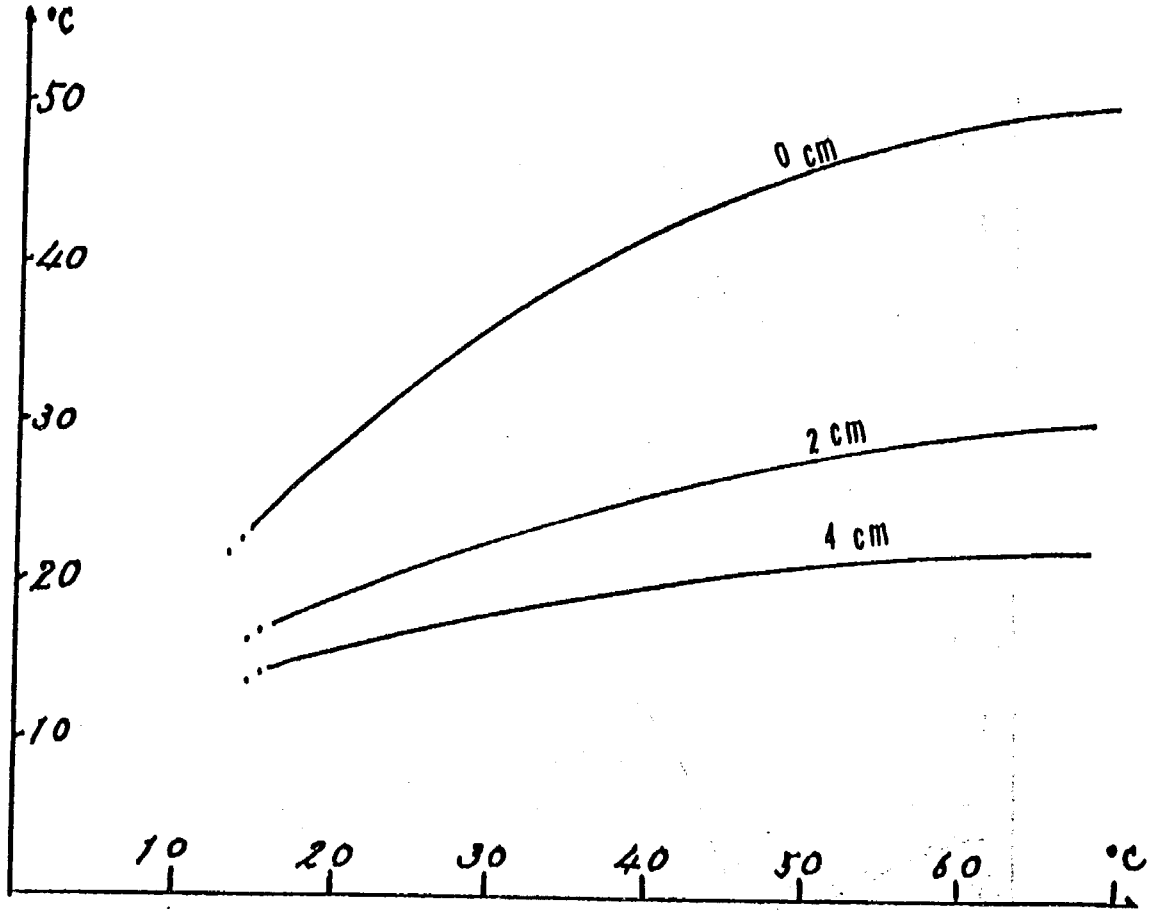

Fig.13 Differential temperatures in circadian rhythm and block thickness

Pavement Reinforcement with Tensar Geogrids for Reflection Cracking Reduction

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Dr. R. L. Lytton
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ABSTRACT

PAVEMENT REINFORCEMENT WITH TENSAR GEOGRIDS FOR REFLECTION CRACKING REDUCTION

by

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Robert L. Lytton, Texas A & M University

The viability of using Tensar geogrids in pavement overlay applications for reflection cracking reduction is being studied in a program of laboratory pilot testing. Described are characteristic properties of geogrids and a proposed mechanism for reinforcement with geogrids. This paper also discusses initial laboratory results using the Texas Transportation Institute Overlay Tester which indicates that the presence of reinforcing grids improve the fracture retarding performance of the mix. Tensar laydown equipment and specific installation technology has been developed for full-scale overlay application.

PAVEMENT REINFORCEMENT WITH TENSAR GEOGRIDS FOR REFLECTION CRACKING REDUCTION

by

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Texas A & M University

Background

Tensar geogrids represent a new generation of polymeric materials with high strength and long-term stability. The properties of these polymers with a mesh or grid-like configuration has been dramatically improved by stretching processes yielding a high degree of molecular orientation. In polyethylene and a few other polymers, orientation led to spectacular improvements in stiffness and tensile strength. The work on the development and potential application of these novel grid structures has been reported at the "Symposium on Polymer Grid Reinforcement in Civil Engineering", in London, March 22 - 23, 1984 (Ref. 1).

Tensar geogrid AR1 with a biaxial open mesh structure (approximate mesh opening 46 X 71 mm) has been specifically designed for asphalt pavement reinforcement. This geogrid is manufactured from polypropylene polymer with increased resistance toward paving hot mix temperatures. The characteristic tensile strength is 27.4 and 16.1 kN/m in the transverse and the longitudinal direction respectively. Various aspects of pavement reinforcement i.e. laboratory pilot studies, construction, and field application have been reported in the literature (Ref. 2 - 7). The objective of this paper is to describe ongoing studies and the viability of using Tensar geogrids in pavement overlay, i.e. applications for reduction of reflective cracking with

emphasis on thermally induced reflection cracking.

Problem Definition

Many pavements which are considered to be structurally sound after the construction of an overlay, prematurely exhibit a cracking pattern similar to that which existed in the old pavement. The cracking in the new overlay surface is due to the inability of the overlay to withstand shear and tensile stresses created by movements of the underlying pavement. This movement may be due to traffic loading causing differential deflections in the underlying pavement layers, expansion or contraction of subgrade soils, or expansion or contraction of the pavement itself due to changes in temperature. Pavement movement, induced by any of the above causes, creates shear or tensile stresses in the new overlay. When these stresses become greater than the shear or tensile strength of the asphaltic concrete, a crack develops in the new overlay. This propagation of an existing cracking pattern from the old pavement into and through a new overlay is known as "reflection cracking".

The occurrence of reflection cracking may take place several years after overlay construction or after only a few months. This form of cracking, together with its accompanying effects, is the primary cause of overlay deterioration. When reflection cracking occurs, the continuity of the overlay surface is destroyed, the structural strength of the pavement is decreased, and water is allowed to enter the pavement system, leading to further deterioration. Thus, the occurrence of reflection cracking prematurely shortens the useful life of asphalt overlays by extending the same problems which weakened the original pavement into the new overlay.

A variety of new materials have been used to retard reflection cracking and to reduce the amount of water that enters the sublayers once a crack has occurred. The principal methods are reinforcing fibres, fabrics, strain-absorbing membrane interlayers of asphalt-rubber, and reinforcing grids. The three techniques employ one or more of the following in extending the life of an overlay:

1. Reinforcing
2. Strain relief
3. Undersealing

Causes of Reflection Cracking

Both traffic loads and temperature changes cause cracks in an old pavement to reflect through the overlay. Every time a load passes over a crack in the old pavement, three pulses of high stress concentrations occur at the tip of a crack as it grows up through the overlay, as illustrated in Figure 1. With each pulse of high stress concentration, the crack grows a little bit more. The first stress pulse that the crack feels is a maximum shear stress pulse shown at Point A in Figure 1. The second stress pulse is a maximum bending stress pulse shown at Point B in Figure 1. The third stress pulse is again a maximum shear stress pulse, except that it is in the opposite direction to the previous shear stress pulse. Also, because there is a void beneath the old surface course at this point, the maximum shearing stress at Point C is larger than at Point A. These stress pulses occur in a very short period of time, on the order of 0.05 seconds. The stiffness of the asphaltic concrete in the overlay and in the old pavement at these high loading rates is fairly high.

The change of temperature in an overlaid pavement can also cause a reflection crack to grow. The thermal stresses in the overlay are due to temperature changes

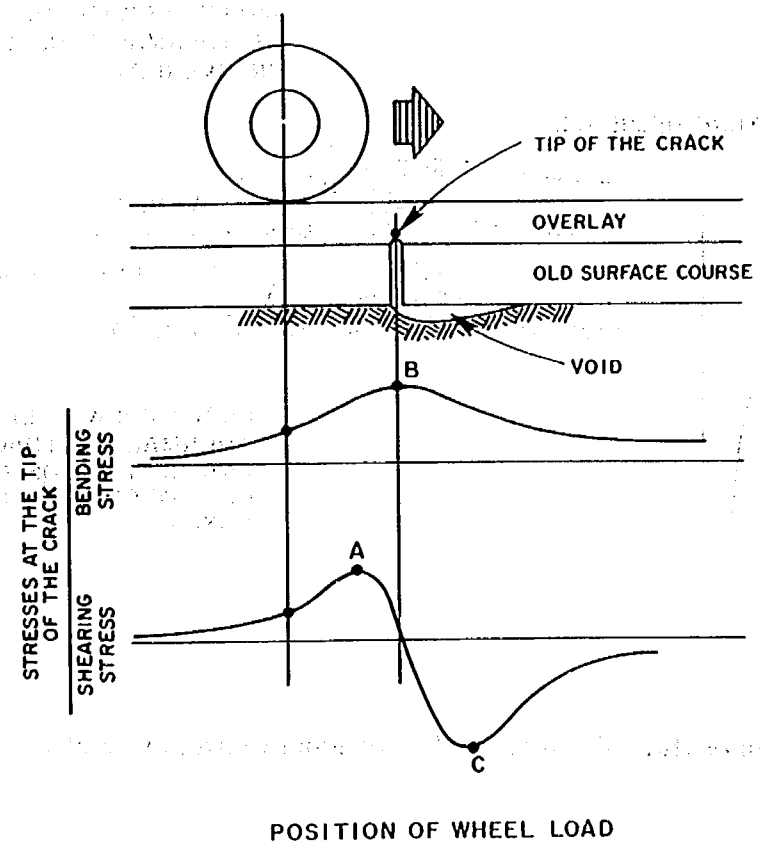

Figure 1: Stresses and Crack Growth in Overlays Due to Traffic

at the surface as shown at Point A in Figure 2, and to the concentration and curling of the underlying old pavement surface as shown at Point B in the same figure. It is observed that thermal stresses can cause cracks to propagate both from the top and the bottom of the overlay. The contraction and curling of the old pavement surface layer applies a shear stress along the bottom of the overlay and produces a concentration of tensile stress at Point B. The change of temperature in a pavement occurs very slowly, over a period of several hours or even the major part of a day. The stiffness of the asphaltic concrete in the overlay and in the old pavement is very low, as much as 1000 times lower than the modulus that these same materials exhibit under traffic loads. Every time a load passes and every time the temperature decreases in an overlaid pavement the reflection crack grows a

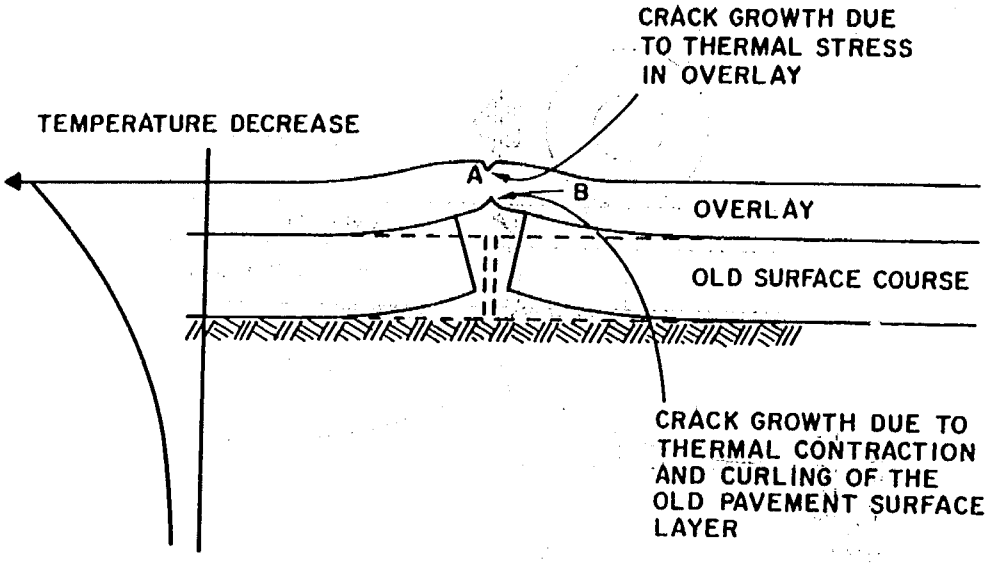

Figure 2: Temperature Changes and Consequent Crack Growth in Overlays

little more. The major hope that engineers have of retarding the growth of such reflection cracks is in the selection of the material properties of the overlay and of the materials that are used to reinforce it so as to reduce, as much as is possible, the growth of these cracks.

Application of Fracture Mechanics to the Design of Overlays

The fracture of overlays at temperatures below about 70°F (21°C) occurs according to Paris' Law which is:

$$\frac{dc}{dN} = A(\Delta K)^n \qquad (1)$$

where

dc/dN = the "crack speed", i.e. the rate of crack growth with the application of each stress cycle.

A = the fracture coefficient.

n = the fracture exponent.

ΔK = the change of "stress intensity factor" that occurs during a stress pulse.

"Stress intensity factors" are related to the concentration of stress around the tip of the crack and have the dimensions of stress x (length)$^{1/2}$. If the objective of using reinforcing grids is to retard reflection cracking, this must be done by reducing the fracture properties, A and n, and thus having a slower rate of crack propagation. Unfortunately, it has been found experimentally (Ref. 8) and theoretically (Ref. 9) that A and n are related to one another, so that when one decreases, the other increases. The relation between the two is:

$$n = a + b \log_{10} A \qquad (2)$$

where both $\log_{10} A$ and the coefficient, b, are always negative.

This equation relating the two material properties shows that when A gets smaller (more negative), n will get larger. These observations suggest that the sum of $\log_{10} A$ and n can be regarded as a measure of resistance to fracture.

Typical relations between $\log_{10} A$ and n are shown in Figure 3 for two different mixes with the same aggregate and gradation. Mix A, which is the mix with the higher slope (1.429), has an asphalt percentage of 3.8% by weight and 3 - 5% air voids whereas Mix B has a 4.5% asphalt content and 7 - 9% air voids, has a lower slope (0.79), and is less fracture resistant for values of n less than 2.0. The test results shown in Figure 3 were measured using the Texas Transportation Institute Overlay Tester (Ref. 8,10). An overlay sample is epoxied to two plates, one fixed and the other moveable horizontally. By opening and closing a gap between the two plates, a crack is propagated upward through the sample allowing all of the

measurements to be made that produce one value of A and n for each test. The device was built to simulate the contraction in an old pavement due to temperature changes. Tests are normally run at room temperature (77°F or 25°C) or at freezing temperature (32°F or 0°C), and at a load cycle rate of one every 10 seconds. The modulus of the overlay material that is produced at the higher temperature and loading rate is similar to what is produced by thermal contraction over a six-hour period at temperatures around 20° to 25°F (-7° to -4°C).

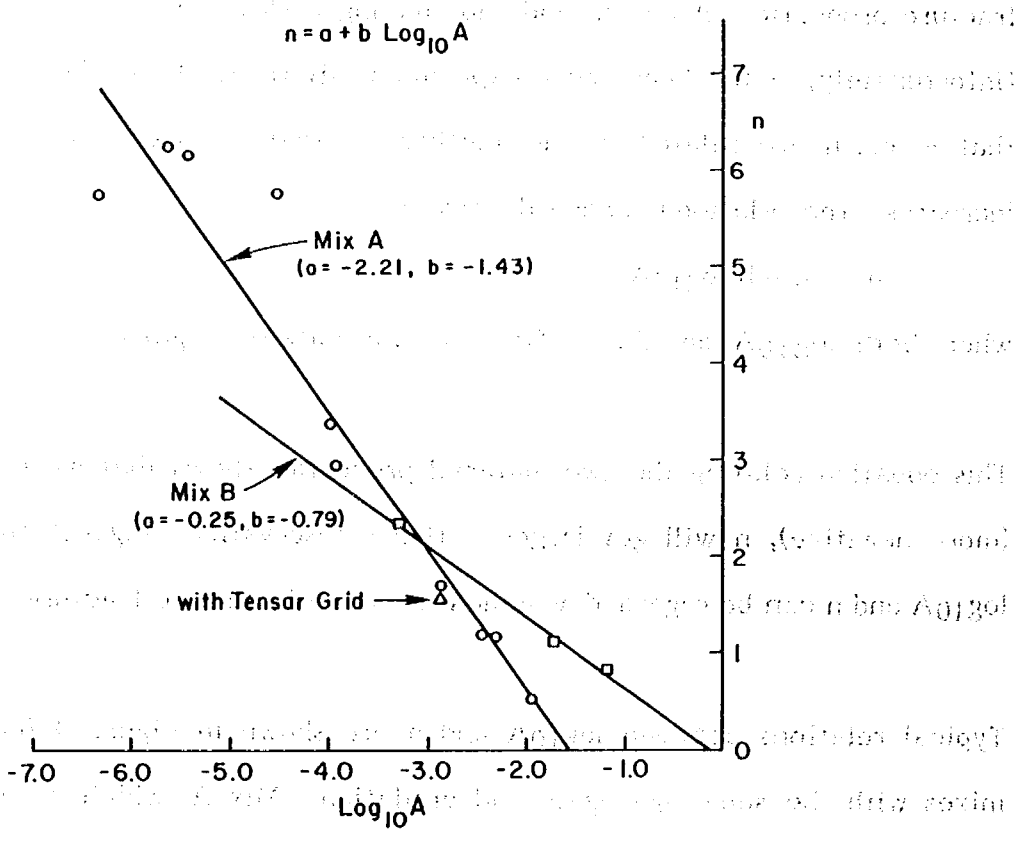

Figure 3: Fracture Properties of Overlay Samples

This test is used alone because it is against this type of movement that reinforcing grids are most effective. The test is simple, reliable, and repeatable. As will be seen in the remainder of this paper, much can be learned from a careful study of the test data of the reasons why reinforcing grids are effective in reinforcing overlays.

Testing Reflection Cracking Properties of Overlay

Because reflection cracking has a variety of contributing causes including traffic, temperature changes, and moisture changes in the base course and subgrade, and because the application of shear and tensile stresses to the overlay occurs at different rates and temperatures, it is difficult to devise a single laboratory test that will exactly duplicate the behaviour of an overlay under field conditions. The Federal Highway Administration now has under contract a research project which is attempting to build, test and verify the applicability of such a laboratory testing device. The test must be set up to simulate a single chosen field condition in which the traffic, crack spacing, temperature change in the underlying cracked pavement, subgrade stiffness, presence or absence of voids beneath the pavement surface, degree of aggregate interlock across the crack in the old pavement, overlay thickness, and position of reinforcing layers are specified. Because each of these affect the rate at which the reflection crack propagates through the overlay, the extension of the test results to other field conditions requires a new test setup and a separate evaluation. The evaluation of the relative effectiveness of various fabrics at different positions within the overlay must be done on a project-by-project basis provided that all of the variables that are noted above can be specified and simulated in the testing apparatus.

There is an alternative to this approach which makes use of relatively simple tests, each of which determines the "fracture properties" of an overlay due to one of the three major means by which cracks propagate through the overlay: bending, shear, and contraction of the underlying pavement. The "fracture properties" are material properties of the overlay and depend upon the properties of the asphaltic concrete mixture, the reinforcing grid, and its position within the overlay. The "fracture properties" for fracture due to bending, shear, and contraction are then put into a

simple computer program along with data on traffic, daily temperature change, crack spacing, and overlay thickness to calculate the number of load applications and temperature cycles that a given overlay can withstand. The advantage of this approach is that more of the simple tests can be made for each mode of fracture separately to permit a more careful investigation of the best reinforcing grid properties and positions within an overlay to reduce as much as possible the rate of crack growth through an overlay. These tests can show more clearly the contribution of the reinforcing grid to the retardation of reflection cracks in each fracture mode separately and can lead more directly to rules, guidelines, and specification limits on the use and application of reinforcing grids in overlays. This is the alternative that has been adopted by the Texas Transportation Institute with the assumption that the fracture properties due to bending and shear are the same. A computer program has been written which can take "fracture properties" as measured in the beam fatigue and overlay tester laboratory testing devices and can predict how long a specific overlay will last under specified traffic and daily temperature changes.

The Role of Reinforcing in Overlays

If cracks in an old pavement beneath an overlay open and close due to temperature changes, there must be relative displacement between the overlay which does not move and the underlying pavement which does. Tensile stresses build up in the overlay and its reinforcing grid while shearing stresses in the opposite direction build up in the asphalt beneath the grid. These stress distributions are illustrted in Figure 4a and 4b. It is assumed that at some value of relative displacement, u_{max}, a slippage occurs between the grid and the asphalt below it, after which the maximum shearing stress that can be mobilized over the debonded length is τ_{min}. stress and relative displacement is assumed to have the form shown in Figure 4c.

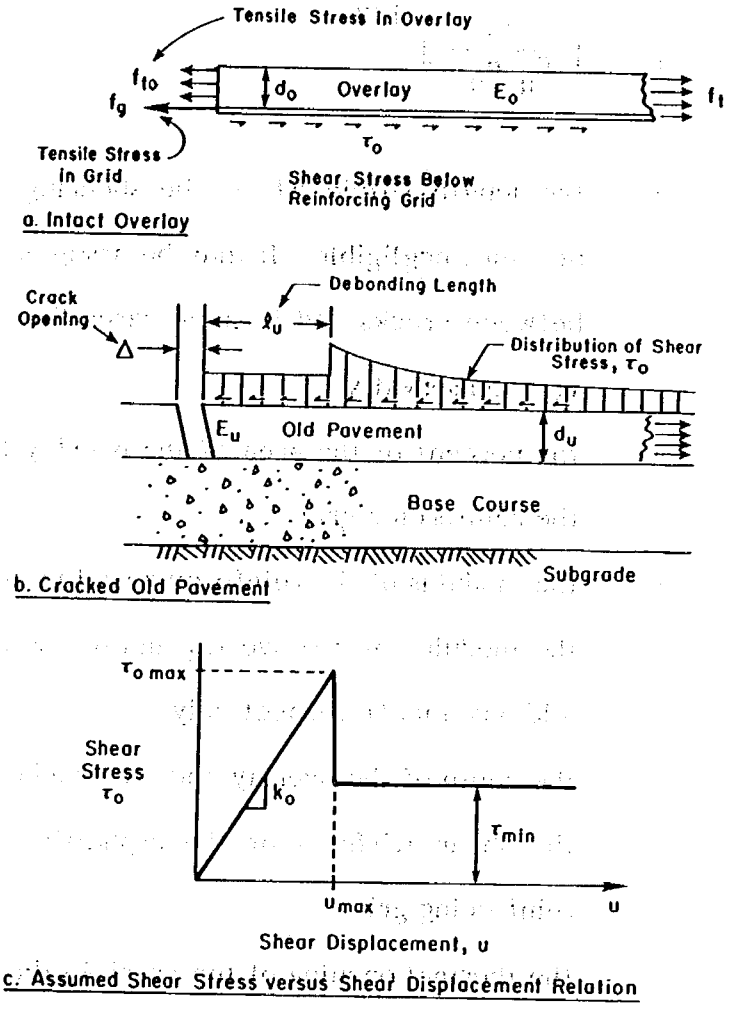

Figure 4: Horizontal Forces Acting in a Reinforced Overlay

Once the equation of horizontal equilibrium has been written, it is possible to solve for the concentrated tensile stress that occurs in the overlay immediately above the crack in the old pavement. The equation, in terms of the quantities identified in Figure 4, is

$$f_{to} = \frac{\dfrac{k_o\,\Delta}{d_o \sinh\left(\frac{\beta w}{2}\right)}\left[\cosh\left(\frac{\beta w}{2}\right) - 1 + f_2\right]}{\left[1 + P_g \dfrac{M_g}{E_o}\right]} \qquad (3)$$

where

$$\beta = \left[\frac{k_o}{E_u d_u}\right]^{1/2} \quad (4)$$

w = the length within which the shearing stress distribution becomes negligible. It may be assumed to be the spacing between cracks and in some cases this assumption will be very conservative.

P_g = the percent of the area of the overlay that is occupied by the reinforcing grid.

M_g = the modulus of the reinforcing grid material.

E_o, E_u = the modulus of the overlay and of the cracked underlayer (old pavement), respectively.

d_o, d_u = the depth of the overlay and the cracked underlayer.

k_o = the shear stiffness of the asphaltic concrete below the reinforcing grid.

Δ = the thermal opening of the crack in the old pavement.

f_t = the thermal stress in the overlay at a distance that is at least the length, w, away from the crack.

f_{to} = the maximum tensile stress that can be allowed in the overlay in order for it to last through a desired number of thermal cycles, N_f.

The equation for the desired number of thermal cycles is obtained from fracture mechanics:

$$N_f = 1 + \frac{d_o^{\frac{2-n}{2}} (1-z_o^{nq+1})}{A\,[af_{to}]^n\,(nq+1)} \quad (5)$$

where

A, n = the fracture properties of the overlay material.

a, q = constants describing the variation of the stress intensity factor with the length of a crack within the overlay.

z_o = the ratio of the initial crack length to the depth of the overlay. The initial crack length can be taken as the radius of the largest aggregate in the mix.

With these equations in hand, it is possible to see several important features of the effect of reinforcing on retarding reflection cracking in overlays:

From Equation 3, it is seen that the larger the modulus of the reinforcing grid or the larger its cross-sectional area, the smaller that the design tensile stress, f_{to}, will be.

From Equation 5, it is seen that the number of thermal cycles that can be withstood by an overlay increases as the design tensile stress decreases. Thus, the more stiff and the greater area of reinforcing that is provided, the longer will be the reflection cracking life of the reinforced overlay. Also, the smaller the fracture properties A and n are, the longer the overlay will last.

Considering Equation 3 again, there is an important collection of variables which also dictates the size of the design tensile stress. The term is:

$$\frac{k_o \Delta}{d_o \beta} = \frac{\Delta}{d_o} (k_o E_u d_u)^{1/2} \quad (6)$$

The smaller this term is, the smaller will be the design tensile stress. The term is increased by the thermal opening, Δ, and by the stiffness and thickness of the old cracked pavement, and is decreased by a greater thickness of overlay.

The two equations, Equations 3 and 5, taken together constitute a set of design equations for reinforced overlays to withstand thermal reflection cracking. As is apparent from the data plotted on Figure 3, the values of A and n that have been measured in the laboratory on a grid reinforced overlay sample and corrected to the same level of void content are below the lines that have been measured previously on unreinforced samples, indicating that the presence of the reinforcing also improves the fracture retarding performance of the mix. This fact, taken together with the beneficial effect of reinforcing itself, indicates that reinforcing grids may be expected to increase the thermal reflection cracking life of overlays.

Installation and Construction Procedure

Concurrent with the development of a mechanism for modelling and testing reflection cracking properties of overlays, considerable effort has been expended to develop installation and construction technology for reinforcement of overlays with Tensar grids. Full-scale field trials will be required to substantiate the effectiveness and cost benefit of overlay reinforcement.

Previous attempts to reinforce overlays, especially the use of welded wire or expanded steel mesh, have underlined the importance and need for proper installation and construction technology. Busching et al (Ref. 11) reviewed extensive field trials with steel mesh reinforcement of airport runways and highway pavements in the U.S.A. and Canada throughout the 1950's and 1960's. In spite of reports claiming significant reflection cracking reduction, the use of steel mesh in

asphaltic concrete has been essentially discontinued. The major cause for failure has been attributed to installation and construction problems in addition to corrosion and cost of steel. According to Davis (Ref. 12), the steel mesh had a tendency to curl at the edges and irregularities in the grid produced bulges and then caused pavement failure. Furthermore, transverse cracks developed at the mesh splices due to mesh expansion at hot mix temperatures, some creep of the mesh occured in front of the paver causing significant bulges to develop, and springiness under rolling and compaction resulted.

Similar problems were encountered during the development of the installation of the polymer grid ARI. Laboratory and full-scale field trials established that the following requirements and features must be applied in order to achieve proper installation:

1) A tension force of about 1.0 to 1.5 kN/m must be applied in order to flatten the grid.

2) The grid must be held absolutely flat against the underlying pavement surface in order to achieve proper interlock and compaction.

3) A levelling course may be required before placement of the grid, if the old pavement is badly distorted and uneven.

4) Tension forces of 2.5 to 3.0 kN/m must be applied to counteract the shearing forces acting on the geogrid by the wheels or crawlers of the paving equipment.

5) Best results are achieved when the grid or mesh is covered with a chip or slurry seal prior to the application of hot mix.

The chip or slurry seal protects the grid against any potential damage or buckling by the movement of the paving equipment and the heat and drag of the hot paving mix and the motion of the screed. This additional step also provides the best bonding

with the old pavement and good interlock with the grid resisting any movement or shrinkage of the grid.

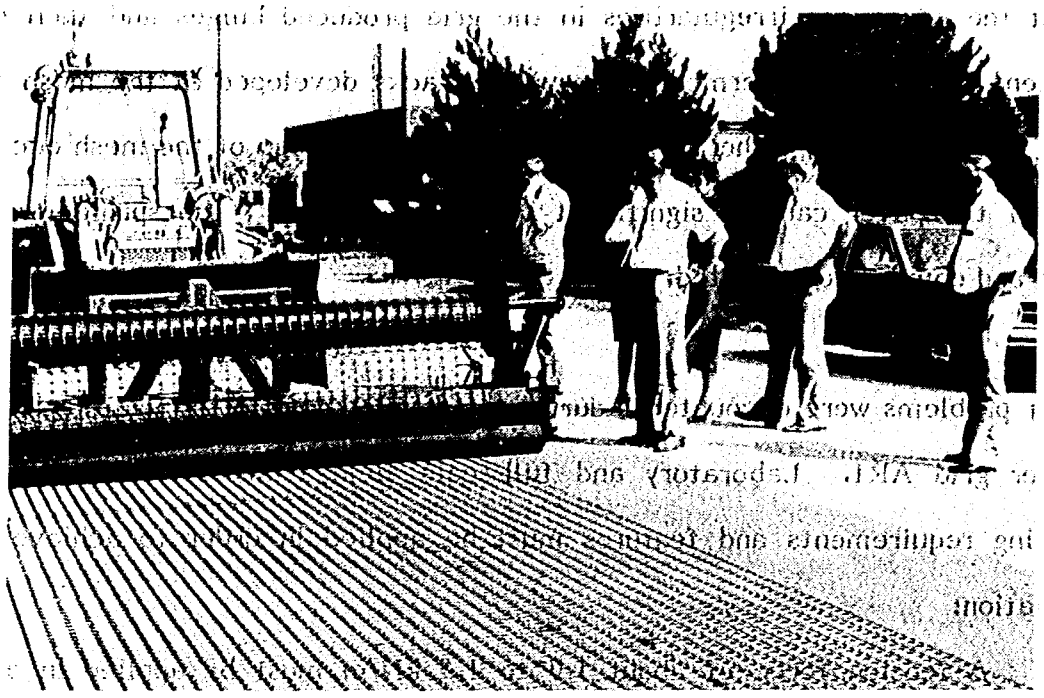

Figure 5: Tensar Laydown Equipment Takes a Test Run Demonstrating the Tensioning Method

Tensar laydown equipment has been developed for holding the grid under tension flat against the paving surface prior to the chip seal and overlay application. The first prototype tensioning equipment, shown in Figure 5, was designed and constructed by K.C. Welding Corporation in Bartlesville, Oklahoma. It consists of a 4 metre wide steel frame with a dispensing hanger for a roll of Tensar geogrid and two rubber-sleeved steel drums. These drums function as nipping rollers and release the mesh as the equipment moves ahead. The lower roller is piloted and maintained at a pre-set tension by a disc brake at each end.

The laydown equipment can be used as an attachment to a front-end loader or tractor with a 3-point hitch. For shipment from job site to job site, the equipment is mounted on a small skid (1.2 X 3.7 m) and transported on a truck or carrier.

Conclusion

Significant progress has been made to define and test the concept and mechanism for overlay reinforcement with polymer grids. Furthermore, installation and construction technology has been developed and is now available for full-scale field applications. The main conclusions of this study and recommendations for further activities are summarized in the following:

1) Fracture mechanics have been applied to describe and model the mechanism and demonstrate the effectiveness of reinforcing grids in overlay design.

2) Preliminary experiments using the Texas Transportation Institute overlay tester which simulates crack growth due to thermal contraction, indicates that the presence of the reinforcing grid improves the fracture retarding performance of the mix.

3) Tensar laydown equipment and specific installation technology has been developed for full-scale field application.

4) Field data must be obtained to support and confirm laboratory results and to develop application design procedures.

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Recent Canadian Tests with Vergrimit Pavement and Comparison with Other Test Results

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INTRODUCTION

The subject of this paper is a specially formulated highway pavement which prevents surface ice formation under certain conditions. This pavement is obtained by adding to the asphalt mix a product known under the trade name of Verglimit. The product releases small quantities of salt solution in contact with condensation or precipitation on the highway, thus delaying the onset of surface icing when the temperature is falling below the freezing point.

The paper starts by describing tests conducted in Manitoba from 1977 to 1982 with this product and relates this experience with other tests performed in North America and Europe. An attempt is made to correlate these experiences and to make recommendations based on this correlation.

In order to place the Manitoba tests into perspective, it is useful to point out that Verglimit pavements have been used and tested since they were first incorporated into critical highway sections in Austria in 1974, in particular with the pavement on the Europa Bridge on the Brenner Autoroute in the Alps. Since then, over a hundred other Verglimit pavement sections were constructed in a dozen European countries, while over fifty such sections were built in North America, and in 1983, some 19 were built in Japan. In all these installations, the highway authorities were hoping to alleviate severe winter icing conditions, rather than to find a cure for all winter driving hazards. Most of these installations were regarded as tests, since the site-specific effectiveness in actual service and other life-cycle factors governing the cost effectiveness of such installations were as yet unknown or unproven. Some of these tests have been since scientifically described in the literature, as exemplified in the bibliographic notes. This, together with the results from the

Manitoba tests, should enable highway officials to draw some useful conclusions, particularly concerning the applicability of this technology to highways in colder regions, the subject of this Seminar.

The Manitoba tests:

The Manitoba tests were performed by a research team of the University of Manitoba. They were performed on a highway interchange within the city limits of Winnipeg. This location certainly represents the coldest and most ice-prone highway environment in which this type of pavement has been put to the test, as is known to date.

The research project can be divided into four stages:

STAGE I: Trial mixes were prepared during the winter of 1977-78 using several previously used local aggregates. These mixes were tested in order to ascertain the range of Marshall stability, percent air voids and densities that could be expected of the actual mix, and to see if any special problems would be encountered. The aggregate graduation used was that of the current Manitoba specifications and the Verglimit content was recommended by the supplier. Samples were tested independently by the Transport Department Laboratory at Winnipeg Airport and the Manitoba Department of Highways Material Laboratory.

The laboratory exercise was repeated in the spring of 1978 when the contract was awarded, and the aggregates were selected for the mix.

STAGE II: The second stage was that of plant production and placing of the bituminous mix. Representatives of the University of Manitoba inspected the plant arrangement for incorporating the Verglimit into the bituminous mix. The layering, compacting and finishing of the mix, as well as place of thermocouples, were monitored by the research group.

The plant mix was sampled by Department of Highways personnel for laboratory testing.

STAGE III: This stage involved the ongoing process of data collection by the University of Manitoba's research staff. This data included reports of surface conditions from visual examination of the test area, traffic volumes, weather, temperature profiles of the test and control sections during the test period, but particularly during the critical winter months of 1978-1979 and 1979-1980.

STAGE IV: The fourth and final stage of the research project consisted of the collation and analysis of data, from which conclusions on the performance of the de-icer additive were drawn.

Site and Surface Characteristics

The test location selected for the research is the traffic interchange for P.T.H. No. 100 at P.T.H. No. 1-W and Portage Avenue in Winnipeg. (Figure 1).

The traffic flows for 1979 are given in Table 1.

Table 1

Average Daily Traffic on Verglimit Test Site 1979

Verglimit South Bound Approach Ramp	= 2376
Verglimit Bridge Deck	= 7814
Verglimit Loop	= 5438
Plain North Bound Exit Ramp	= 2739
Plain Bridge Deck	= 4011

The existing surfaces were of Portland cement concrete. Prior to the placing of the bituminous mix, the existing joints between the pavement

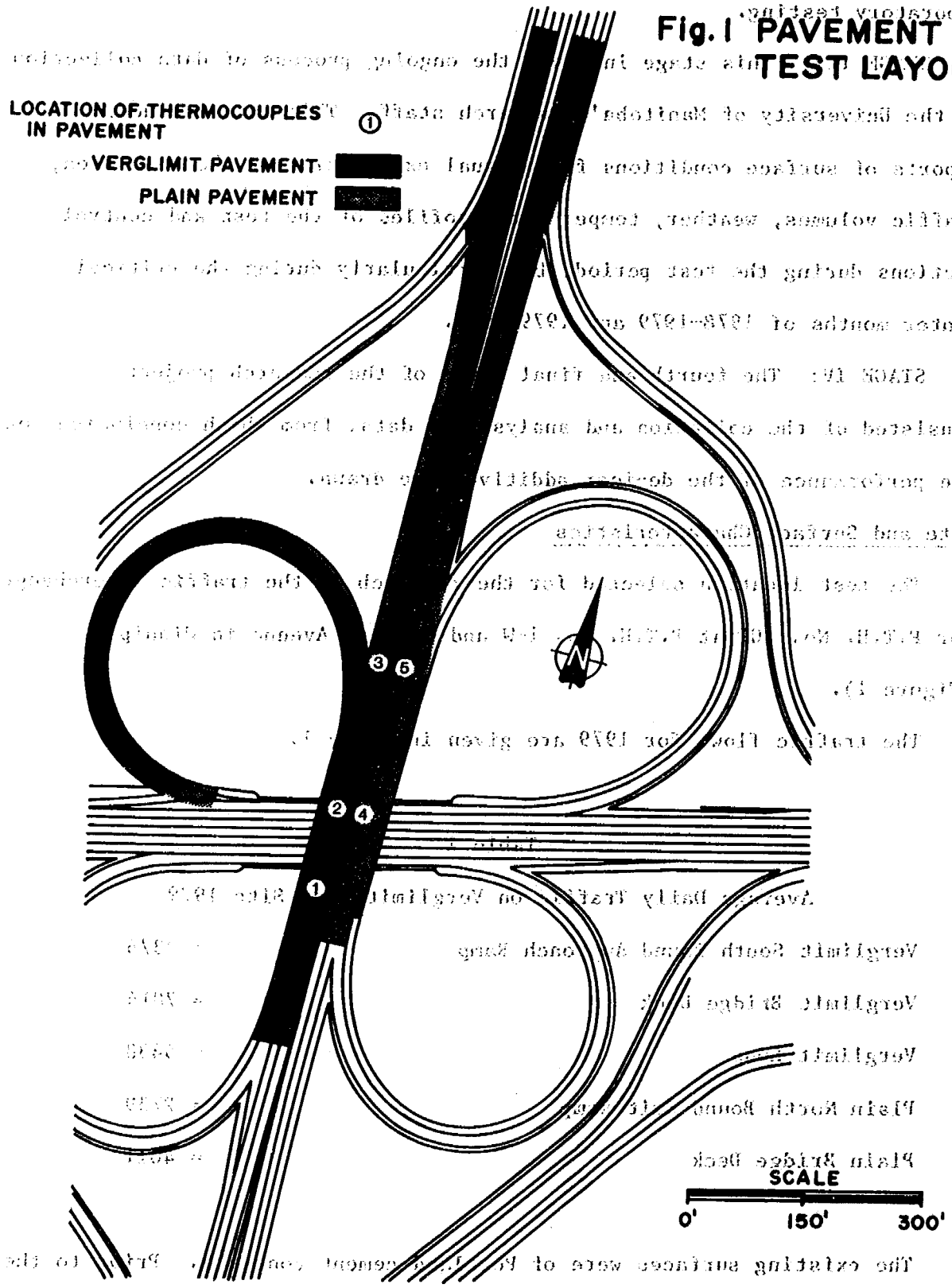

Fig. 1 PAVEMENT TEST LAYOUT

slabs were recut and filled with sealant (Neiroprene). Where the shoulders had dropped beneath the level of the Portland cement slabs, they were brought up to grade before overlaying.

Two types of bituminous mixes were put down, one containing Verglimit and the other, a control mix, containing no Verglimit as indicated on Figure 1.

The pavements were constructed in June 1978. The timing of construction was carefully chosen to be in the summer so that compaction would be effective. By constructing the pavement early, there was a chance for traffic to wear down the treated surfaces and expose Verglimit particles before the season for icy conditions began. The contracting company used by the Province of Manitoba Highway Department was B.A.C.M. Ltd.

When the paving was completed, joints were cut into the new surface directly over the joints in the underlying Portland cement concrete pavement. These were cut by diamond saw and they were carefully filled with a hot pour made by Uniroyal (Sealz 6140). The concern was that there be no reflection cracks in the new pavement, where deterioration of the bituminous mix might occur. It was anticipated that any Verglimit flakes exposed by such cracks, would react with water in the joint, and cause excessive swelling pressures leading to possible deterioration along the cracks.

Asphalt Mix Design and Production

The bituminous mix was designed with an aggregate graduation compatible with current Manitoba specifications. The proposed Verglimit

and void contents were 5.3% and 3-5% respectively with the optimum asphalt content being based on criteria used in the Marshall mix design (3).

Plain mix was prepared using Shell 150 #200 high viscosity asphalt and aggregate supplied. This mix was based on a combination of: 34.5% stone, 34.5% intermediate, 17% # 10 additive, and 14% # 40 additive.

The optimum asphalt content was 5.8%. At this asphalt content the mix had the following properties:

Density	150+ pcf	V.M.A.	15.5%
Stability (water soaked)	1000+ lb.	Voids filled	77%
Flow	9	Asphalt absorption	0.6%
Air voids	3.5%		

Mix containing Verglimit was prepared using Shell 150 # 200 high viscosity asphalt and aggregate supplied. This gradation was based upon a combination of: 32.7% stone, 32.7% intermediate, 16.1% # 10 additive, 13.3% # 40 additive, and 5.2% Verglimit. Asphalt content: 5.5% (dry aggregate weight basis) when Verglimit is considered part of the aggregate, or 5.8% dry aggregate weight if the Verglimit is excluded.

Verglimit content: 4.93% of total mix or 5.2% of dry aggregate weight.

Mix Properties:

Density	147.5 pcf	V.M.A.	14.5-15%
Stability (water soaked)	1200+ lb.	Voids filled	75%
Flow	11	Asphalt absorption	0.6%
Air voids	3-4%		(assumed)

The total tonnage of pavement which was laid on June 18th, 21st, and 22nd was 961.1 tons of plain bituminous mix and 651.6 tons of bituminous mix containing Verglimit.

Data Collection and Analysis

The positions at which data was collected were painted for easy identification. There were three positions on the northern approach to the bridge, three on the bridge deck and two on the Verglimit paved loop (Figure 1).

Two methods were used to collect data.

The first method involved bi-weekly visits by the University of Manitoba research team to the sites. The inspection included taking of photographs, temperature readings and the recording of any other relevant conditions.

The second method involved the assistance from the maintenance workers who recorded on special site forms the weather reports and pavement conditions, maintenance treatment and other general remarks.

Analysis of Data:

The Verglimit was particularly visible immediately after rolling. During the following few months, the presence of exposed Verglimit flakes was evidenced by the visual difference between the Verglimit paved sections and the plain paved sections.

There was enough evidence of the presence of Verglimit to judge that there was sufficient exposed admixture for a reasonable trial of its performance under winter conditions.

By the late fall of 1979, based on visual examination and chemical tests, there appeared to be a definite reduction in the presence of exposed Verglimit particles. In this case the entire test area was wetted

and tested with silver nitrate and the reduction in Verglimit exposure was confirmed.

Conditions of precipitation and limited cold were not experienced for very long in the early winter period in 1978. The transition into very cold temperatures was fairly abrupt. This was followed by a very long period during which the temperature was below 0°C and was infrequently in the range of 0°C to -10°C. It may be stated that the winter of 1978-79 was the coldest in a forty year period. The amount of snowfall for December and January was below normal, but the combination of the elevated position of the test site, the surrounding open areas and prevailing north-northwest winds cause frequent drifting of snow across the pavements.

Up to the middle of March 1979, the Verglimit paved sections were treated 88% of the times that the plain pavement was treated. This treatment was usually done with winter sand. Almost 75 percent of the times that the Verglimit paved sections were treated occurred during frosty conditions with temperatures less than -7°C and snowfall of less than 1.0 cm. The remaining 26 percent occurred at varying periods. These include periods of heavy snowfall during which the temperature may have been as high as -3°C, and two out of the three occasions when there was freezing rain.

The times when the plain pavements were treated and the Verglimit paved sections were not, were mainly days of light snowfall or frost. The temperatures of these days ranged from -7°C to -30°C.

There were some days on which the Verglimit paved sections were treated later than the plain paved sections. On four occasions when both types of pavement were treated at the same time, the Verglimit paved

sections were observed to improve more quickly. There was also one occasion when the Verglimit paved bridge deck was more slippery than the plain paved bridge deck. This was after a light overnight snowfall with ambient temperatures of around -30°C.

The Verglimit paved sections appeared in general to be less slippery during icing conditions than the plain paved sections but, usually, only marginally so. Manual snow and ice removal was observed by the university team to be easier on the Verglimit pavements. The north approach to the Verglimit paved bridge deck was usually in better condition (in terms of traction) than the other Verglimit sections although there were times when it was as bad as, or worse than the bridge deck or loop.

The early part of winter 1979-80, specifically November and December, was milder and drier than normal. There were several days in both months when the combination of temperature and moisture created conditions that necessitated maintenance treatment. These conditions prevailed on the plain and the Verglimit paved surfaces.

For the months of January and February there were very few days when the mean temperature was above -10°C. There were only two days in each of these two months when the precipitation exceeded 2.5 cm. Nevertheless, there appeared to be several occasions when, in the judgement of the maintenance personnel, treatment of all surface areas was required; frost on the surfaces was usually a factor.

There was no apparent difference in the surface conditions of the Verglimit and the plain paved pavements, for the entire winter, with one or two exceptions. There was no difference in the conditions of the pavements that led to sanding being done on one type of pavement and not on the other. There were two reported occasions when, after treatment,

the Verglimit paved sections appeared to improve more rapidly than the plain pavement.

Condition of Pavement Surfaces:

The physical condition of the pavement surfaces in the test area can be summarized over various parts of the test period as follows: During fall 1978, all test area surfaces were generally not very worn by traffic since the construction of the pavements. The surfaces, generally, appeared tight and in good condition.

There was some evidence of traffic wear on the Verglimit treated surfaces in spring 1979. However, the surfaces still appeared to be generally tight. The plain paved surfaces were not visibly changed much by the traffic of the winter months. There were some cracks on both pavement types of the bridge decks; the cracks were more apparent on the Verglimit paved section. There was also a crack on the Verglimit paved loop at the edge of the right shoulder and the lane.

During fall 1979, surfaces of the entire test area appeared only slightly more worn than they were the previous spring. The cracks were sealed and the surfaces were generally still in good condition.

In spring 1980, the surfaces of the more heavily travelled lanes of the Verglimit paved north bridge approach (i.e., not the median lanes), the Verglimit paved bridge deck (except the median lane) and the Verglimit paved loop all showed enough wear to clearly expose the coarse aggregate. There was however, no "pebbly" condition and the surfaces could still be rated as good.

A crack once again developed on the loop and the right edge of the travelled lane. This suggests that some movement of the embankment has opened up the joint between the lane and the shoulder.

The plain paved sections of the test area were only slightly more worn than in the fall of 1979, i.e., their condition was generally good.

There appeared to be no excessive deterioration of the Verglimit pavement mix along the cracks and joints of the pavement.

The controlled tests on this road section were terminated after the winter of 1979-80. Subsequently in 1983 the pavement was found in need of replacement and removed. No substantial changes had been observed in the intervening time as far as ice prevention properties.

Conclusions of Manitoba Tests:

It was expected that the Verglimit admixture should prove itself in the winter of 1978-79 since, based on visual examination and the silver nitrate tests, Verglimit flakes appeared to have been sufficiently exposed. Although there were portions of the test area, specifically the north bridge approach and loop, which had 4.1% of Verglimit, the performance of these areas compared favourably with the bridge deck, which had 4.9%.

For the two years of study, relatively short transition periods, in terms of weather conditions, occurred between the fall and winter seasons and between the winter and spring seasons. These are transition periods when temperatures normally hover around the freezing point, and it should be expected that the admixture would prove to be most effective. However, there is no definite evidence that the performance of the Verglimit pavement area was improved when compared with the plain pavement.

Prolonged periods of cold weather are common in Manitoba. A particularly long period of very cold weather developed during the winter months of 1978-79, and again in January and February 1980, when the mean

daily ambient temperatures scarcely rose above -10°C. Despite the intense cold, it appears that the combined effect of winter sun and black asphaltic surfaces, aided by frequent drifting of snow across the roadways, lead to icing of the test area. The Verglimit pavement surfaces were hardly less resistant to icing in this temperature range than the plain pavement in the winter of 1978-79, and no more resistant to icing than the plain pavement in the winter of 1979-80.

For ambient temperatures in the range of 0°C to -10°C, in the winter of 1978-79, there was some indication of a slower development of icy conditions on the Verglimit pavement compared to the plain pavement. When ice did form on both Verglimit and plain paved sections, it appeared there may have been less bond of the ice to the Verglimit pavement than in the case of the plain pavement. No such differentiations were observed during the winter of 1979-80.

Under conditions of freezing rain, it was felt that both Verglimit and plain pavements required treatment.

During the winter of 1978-79, winter sand was applied often because of the icy conditions of the test surfaces. During the winter of 1979-80 there were frequent applications of a combination of sodium chloride and winter sand. The need for frequent treatment is indicative of the inability of the Verglimit admixture to cope with local icing conditions.

There is evidence, in the laboratory, that compacted, unconfined mixes are subject to expansive forces, resulting from the hygroscopic action of the calcium chloride. However, there was no deterioration of the mix at cracks, joints or edges of the Verglimit paved sections.

Comparative review of other tests on Verglimit pavements:

The most substantial and extended experience with Verglimit pavements originates from Europe. The early experiences seem to have focussed on the mechanical properties of the pavement in the presence of traffic and environmental wear and tear. In general, with careful adherence to construction and mix composition guidelines, the European tests have established that Verglimit pavements have a sufficiently long structural life and retain their salt solution releasing properties. Furthermore, under the temperature and precipitation conditions prevailing (usually within 0°C to -3°C), the prevention or delaying of sheet ice formation on the pavement was considered useful. These results ranged from cases where additional winter salt or sand spraying could be reduced, to other cases where prevention of unforeseen icing reduced the need for anticipatory salt or sand spraying.

The more limited North American tests, with the exception of the Manitoba tests, have also established the satisfactory mechanical and structural behaviour of Verglimit pavements. Concerning de-icing properties, the tests showed a gradation, where southern locations with milder winter conditions showed more readily possibilities for a reduction of additional salting or sanding; and where more northerly locations, in Canada, tended to show safety advantages, particularly in cases of intermittent icing conditions in temperatures between 0°C and -10°C on exposed sites, but little reduction in the need for salt or sand applications.

The Manitoba tests show that useful Verglimit properties cannot be exploited in climatic conditions where prolonged temperature spells below -10°C are experienced and where traffic or environmentally generated sheet

ice formation is nevertheless prevalent. These results can be interpreted as establishing some application limit to this pavement technology and are as such very significant. Some observations regarding all the preceding results are however in order, since some interesting additional considerations can be brought out.

Unanswered questions concerning Verglimit pavement tests:

Taken from a somewhat detached view point, a number of criticisms can be raised concerning technical reports published to date on Verglimit pavements. Firstly, while a lot of care was exercised in documenting quantitatively all manner of details in construction techniques and subsequent structural behaviour of the pavement, the ice formation/prevention evaluation was in general left open to a large degree of subjectivity and consequently comparability of experience is very difficult. Vehicle dynamic behaviour on the experimental highway segments under icing conditions is largely undocumented and accident statistics are too sparse.

Secondly, even if a subjective evaluation was made that the Verglimit pavement was effective, a recurrent theme appears in the reports on the uncertainties regarding cost-effectiveness of this expensive technique. On the one hand, highway authorities point out that only exceptionally exposed highway segments, such as elevated sections, should be considered as candidates for such pavement, given the prevailing cost of the proprietary product. On the other hand, safety considerations and user benefits are also invoked, with very elastic claims as to their monetary value. Decisions are then taken, largely on opinions of individuals, rather than objective criteria.

It is also interesting to note a fairly unanimous view that conventional winter salting or sanding represents by far the most effective procedure, provided it can be effected on a timely basis.

Thirdly, views originating outside of transportation-oriented organizations and concerning environmental effects are being expressed in a manner which tends to confuse even more a complicated cost-benefit trade-off. The very existence of a de-icing technique which demonstrates lower water pollution characteristics than the conventional highway winter treatment raises expectations of officials and of the public to possibly unjustified heights, given the wide variety of results reported.

Conclusions and recommendations:

These considerations, taken in total, lead us to propose the following recapitulation and recommendations, in view of the aim of this Seminar:

1.) An interesting highway pavement technology has developed over the last ten years, which can contribute to improved safety relative to surface ice hazards. This technology seems however to be limited in application to temperature conditions to a narrow range below 0°C, related to the freezing point of the salt solutions brought into effect.

2.) The relevance of this technology to highway construction in cold areas depends on the specific meaning given to "cold areas". It obviously does not apply to highways where safe driving conditions have to be ensured despite severe winter conditions over prolonged periods, or where heavy winter maintenance is required in any case. It does, however, apply where winter temperatures are frequently crossing the freezing point and where winter maintenance is light on the average, with occasional and unpredictable instances of heavier salting or sanding requirements.

3.) Given the degree of success achieved to date, but taking the high cost of this technology into account, further experimental evidence should be sought to quantify the road safety effects attributable to this type of pavement and to translate these effects in monetary terms useful to highway authorities.

4.) In those locations and jurisdictions where Verglimit pavements are utilized, careful quantification of long term ecological benefits should be established. In the absence of such quantifiable evidence, ecological considerations cannot be included in the highway construction and maintenance decision making process.

5.) Finally, the development of other de-icing pavement formulations, which might be effective in more severe temperature conditions, should not be ruled out all together. There is room for more laboratory experiments simulating the microclimate on a pavement surface and the action of various chemicals which might further delay sheet ice formation. Such tests might also elucidate some of the variations observed concerning life-cycle property trends of chemically active surfaces. And most importantly, there is also the prospect of lowering the cost of such technology.

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Resistance of Cement-Asphalt Emulsion Mixtures to Freezing and Thawing

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ABSTRACT

This paper describes resistance of cement-asphalt emulsion mixtures (CEM) to freezing and thawing with the emphasis on the behaviour of water.
Portland cements and asphalt emulsions have long been used separately for base course construction. The advantages of using these two materials in dense-graded mixtures have been well established. However, it has been also pointed out that the former has a disadvantage of shrinkage cracking and that the latter lacks initial bearing capacity.
In Japan, CEM was first developed as the base of New-Tokaido Line about 15 years ago. Since then it has been experimentally used as road bases, too.
So far the author has been investigated mechanical properties of CEM through Marshall Test, Uni-axial Test, Bending Test, Stress Relaxation Test before this paper.
The ASTM C-666 test method was utilized for resistance of CEM to rapid freezing and thawing. In this procedure, the following variables were included : (1) cement content (2) emulsion content (3) curing time (4) freezing and thawing time (5) method of evaluation.
This paper primarily describes the effect of all the above variables.

RESISTANCE OF CEMENT-ASPHALT EMULSION MIXTURES TO FREEZING AND THAWING

by

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1. Introduction

Portland cement and asphalt emulsion have been used in pavement construction for the treatment of base course granular materials, and the potential advantages of using these materials have been reasonably well established. However, the disadvantages of using them have been also recognized by pavement engineers in the field. They are shrinkage cracking for cement treated mixtures and lack of initial strength for emulsion treated mixtures. Accordingly, investigations have been made into the effect of adding small amounts of asphalt emulsion to cement mixtures (CM) or portland cement to emulsion mixtures (EM). So far considerable number of papers has been published, but the nature of cement-asphalt emulsion mixtures (CEM) has not been well cleared.

The author has been studying the mechanical properties of CEM and this paper describes the resistance of CEM to freezing and thawing with the emphasis on the bihaviour of water.

2. Methods of Experiment

(1) Materials

Materials used for making specimens include portland cement, asphalt emulsion and aggregates. The properties of asphalt emulsion used and the quality standards by the Japan Asphalt Emulsion Association[1] are shown in Table 1. The crushed aggregate used in this study was from Kuzuu, Tochigi Prefecture. It was separated by sieving and then recombined to give a appropriate grading as shown in Figure 1.

(2) Preparation of Specimens

The specimens used in this study were 12.7cm high by 10cm diameter cylinders compacted at a constant level of effort according to the typical method defined in the Manual for Design and Construction of Asphalt Pavements[2] by the Japan Road Association.

The variables included in the specimens are as follows :

1) cement contents for CM : C=2%, 4% (2 kinds)
2) emulsion contents for EM : E=6% (one kind)
3) cement and emulsion contents for CEM : combinations of C=2%, 4%, E=2%, 4%, 6%, 8% (8 kinds)

C and E are determined by the following formulae :

$$C(\%) = W_C / (W_C + W_E + W_A) \times 100$$

$$E(\%) = W_E / (W_C + W_E + W_A) \times 100$$

where W_C : weight of cement
W_E : weight of emulsion
W_A : weight of aggregate

Appropriate amount of water was added to these materials before compaction in order to give the mixtures the optimum moisture content.

(3) Curing of Specimens

After compaction and stripping from molds, the specimens were cured at room temperature(20°C) and approximately 60~80 percent relative humidity for 7,14,28,56 days.

(4) Freezing and Thawing Test

Freezing and thawing tests were performed using the method similar to that of ASTM C-666[3]. Weight and dynamic modulus of elasticity of specimens were measured after each cycles of freezing and thawing, and finally uni-axial compression test was conducted after 30 cycles.

3. Determination of Optimum Moisture Content

The optimum moisture content for CM (cement + aggregate + water added), EM (asphalt emulsion + aggregate + water added) and CEM (cement + emulsion + aggregate + water added) was determinated first.
Figure 2 shows the results for CM obtained by specimens changing cement content and amount of additional water. It can be concluded from the figure that the optimum moisture content of CM is constant (8%) without being influenced by cement content. The amount of additional water which give CE the optimum dry density vary according to emulsion content as shown Figure 3. However, if the amount of water added and included in emulsion is used as a parameter instead of amount of water added, the optimum moisture content becomes nearly constant (8%) as shown Figure 4. This leads to the conclusion that the water included in emulsion also works as a lubricant when EM is compacted.
Based on the results mentioned above, it is easy to presume that the optimum moisture content of CEM will be also dependent on the amount of water added and included in emulsion. Figure 5 shows that the presumption could be reasonable.

4. Experimental Results

(1) Weight Change of Specimens

The weight of specimens changes according to evaporation and absorption of water during curing, immersion in water and test. Figure 6 indicates the typical trend of weight change for CM, EM and CEM.
The weight of specimens just after compaction (Phase I) decreases owing to evaporation during curing in the air until it reaches Phase II. Then specimens are immersed in water and the weight increases by soaking water. At Phase III, freezing and thawing test is begun. During the test, the weight of specimens increases at first, but it gradually decreases and specimens are broken finally.
This process of weight change will be discussed in the followings.

(2) Weight Change by Evaporation

This is a weight change which occurs during the process from Phase I to Phase II. When C=2% (Figure 7), the weight of CM and EM decreases to 94.0% and 93.5% of initial weight respectively independent of curing time. However, in the case of CEM, the more emulsion content causes the less evaporation of water at 7 days, while the weight is constant 94.0% (the same value as that of CM) at 56 days independent of emulsion content. This means that cement content is the principal factor of controling evaporation and that emulsion has the effect of delaying evaporation.

Similar trend was obtained from specimens containing 4% of cement as shown in Figure 8.

(3) Weight Change by Absorption

This is the weight change from Phase II to III. The relative value of absorption $W_0(\%)$ was defined here as follows :

$$W_0(\%) = (W_2 - W_1) / W_1 \times 100$$

where W_1 : specimen weight before immersion

W_2 : specimen weight after immersion

Figure 9 and 10 show the change of relative value of absorption. The value is largely influenced by emulsion content for all curing days. It is apparent from the figures that emulsion in CEM limits water absorption.

(4) Weight Change during Tests

The change of relative weight of specimens containing 2% of cement after curing of 7 days is shown in Figure 11. As for CM, the relative weight decreases to 94% according to curing in the air for 7 days. Then it increases up to 99% by 4 hours' immersion. This specimen was broken by only one cycle of freezing and thawing test, decreasing the weight rapidly. The relative weight of EM decreases to 93% according to curing, but very little increase is gained by immersion. Then the relative weight increases during test and is still increasing at 30 cycles. The specimen of CEM with C=2%, E=2% behaves likely as CM. But the effect of adding 2% of emulsion is found in reduction of evaporation and absorption, and increase of resistance to freezing and thawing. It is easily realized

from the figure that the more emulsion content, the more resistant mixtures are gained.
Test results for C=4% are shown in Figure 12. General trend is similar to that of Figure 11, though resistance to freezing and thawing becomes higher.

(5) Change of Relative Dynamic Modulus of Elasticity
The relative dynamic modulus of elasticity E_R(%) is calculated as follows:

$$E_R(\%) = E_A / E_B \times 100$$

where E_A : dynamic modulus of a specimen after any cycles
E_B : dynamic modulus of a specimen before freezing and thawing

This is used as an indicator for resistance to freezing and thawing.
Figure 13 and 14 show the change of dynamic modulus during test for specimens cured for 7 days. The modulus generally decreases according to the increase of cycles, however, the rate of decrease is small if emulsion content is rich.
Figure 15 and 16 show the results tested after curing of 56 days. Resistance of specimens to freezing and thawing in this case is higher than that of in Figure 13 and 14.

The relative dynamic modulus after 30 cycles is compared in Figure 17 and 18 showing the effects of cement content, emulsion content and curing time. Though the effect of cement content is not so clear, emulsion content and curing time influence considerably on the values of the modulus.

(6) Uni-axial Compressive Strength
The compressive strength after 30 cycles is shown in Figure 19 and 20. Comparing both figures, it is realized that the strength of CEM with C=4% is nearly two times larger than that of CEM with C=2%. This means that the level of strength is primarily determined by cement content even after freezing and thawing. These figures also indicate the existence of the optimum emulsion content around 5%.

5. Conclusions

Results of experimental studies on the resistance of CEM to freezing and thawing are summarized as follows :

(1) The optimum moisture content of CEM could be determined based on the amount of water added and included in asphalt emulsion.

(2) Asphalt emulsion in CEM has the effect of limiting water absorption.

(3) Under the conditions of this experiments, the resistance of CM to freezing and thawing is very low, but compressive strength of CEM after freezing and thawing is controled by cement content.

(4) The optimum emulsion content may exist as for the compressive strength after 30 cycles, but larger amounts of cement and asphalt emulsion generally give the mixtures higher resistance to freezing and thawing.

Acknowledgement

The author wishes to express his thanks to Mr.Toshiaki Nomura for the assistance of experiments and preparation of the paper.

References

1) The Japan Asphalt Emulsion Association, "The Comparison of Old and New Standards for Modified Asphalt Emulsion", Asphalt Emulsion, No.70, 1982

2) The Japan Road Association, "Manuals for Design and Construction of Asphalt Pavements", 1978

3) ASTM C 666-80, "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing"

Table. 1 Quality Standards and Test Results for Asphalt Emulsion.

			Quality Standards by the Japan Asphalt Emulsion Association	Test Results
Engler Viscosity, at 25℃			2~30	4.4
Residue on 1190μm Sieve		%	max. 0.3	0.06
Stability to Mixing with Cement		%	max. 1	0.3
Residue from Evaporation Test		%	min. 57	58.0
Residue	Penetration at 25℃		60~300	162
	Ductility at 15℃	cm	min. 80	min. 100
	Solubility in Trichloroethane	%	min. 97	99.2
Settlement, 5days		%	max. 5	1.3
Particle Charge			No Regulation	Neutral

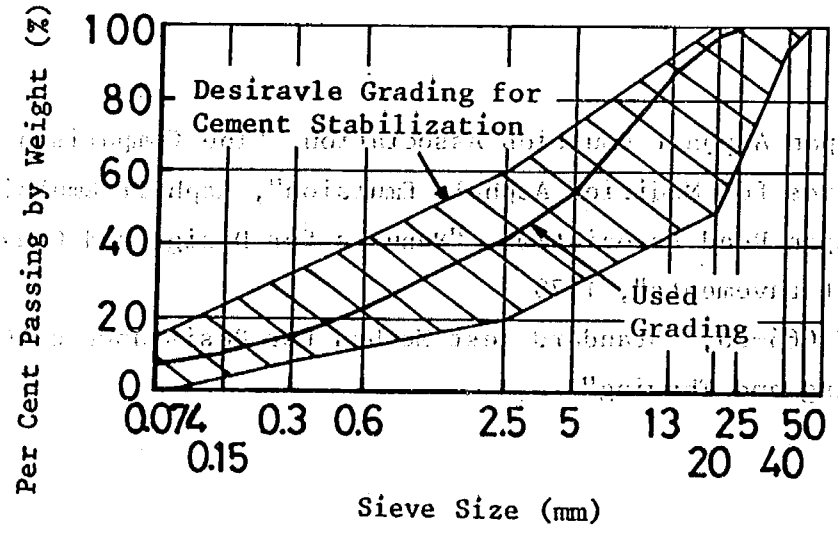

Fig. 1 Grading Curves for Aggregates

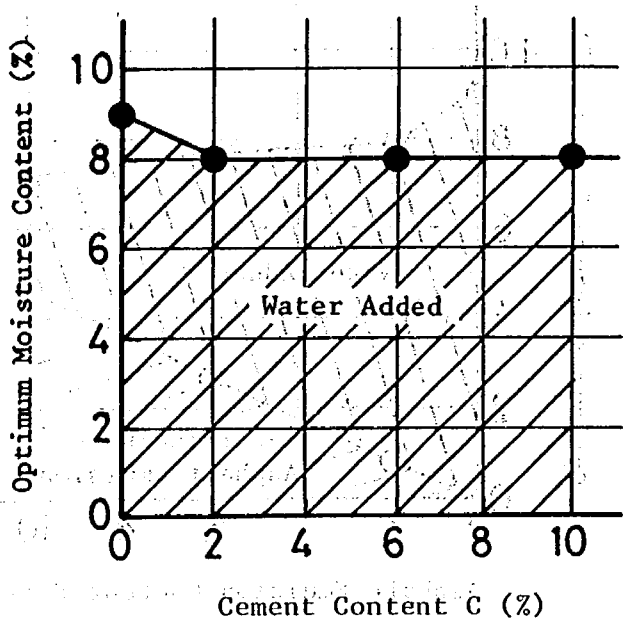

Fig. 2 Optimum Moisture Content of Cement Mixtures

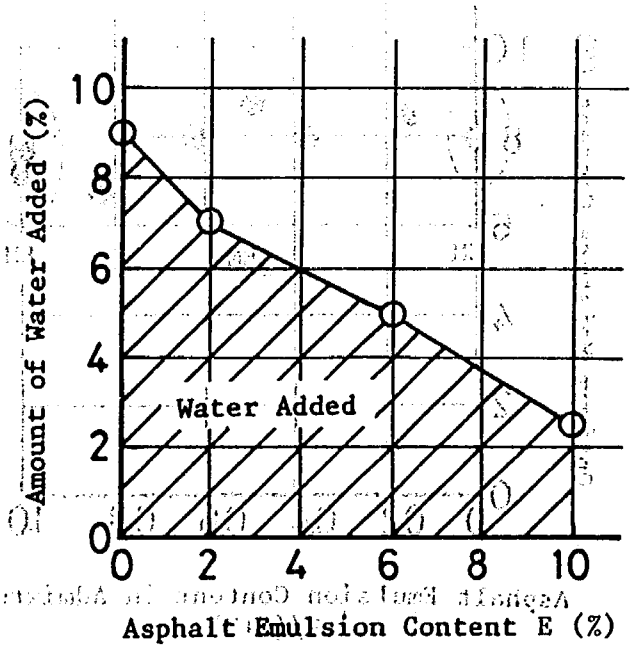

Fig. 3 Amount of Water Added for Asphalt Emulsion Mixtures

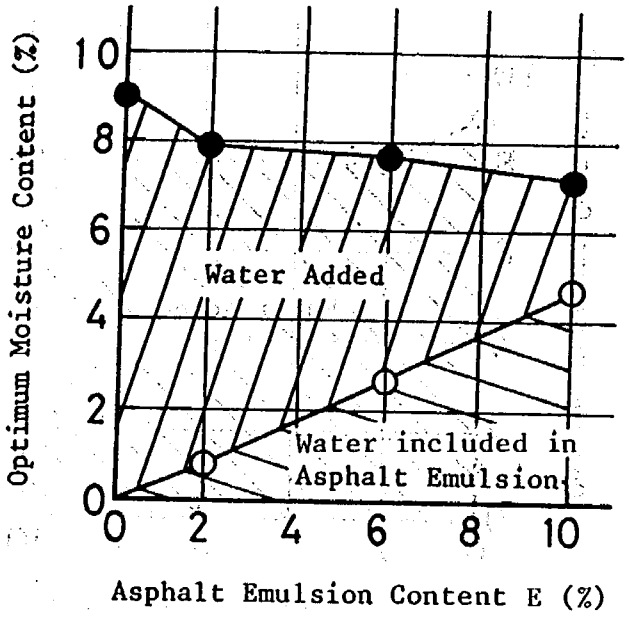

Fig. 4 Optimum Moisture Content of Asphalt Emulsion Mixtures

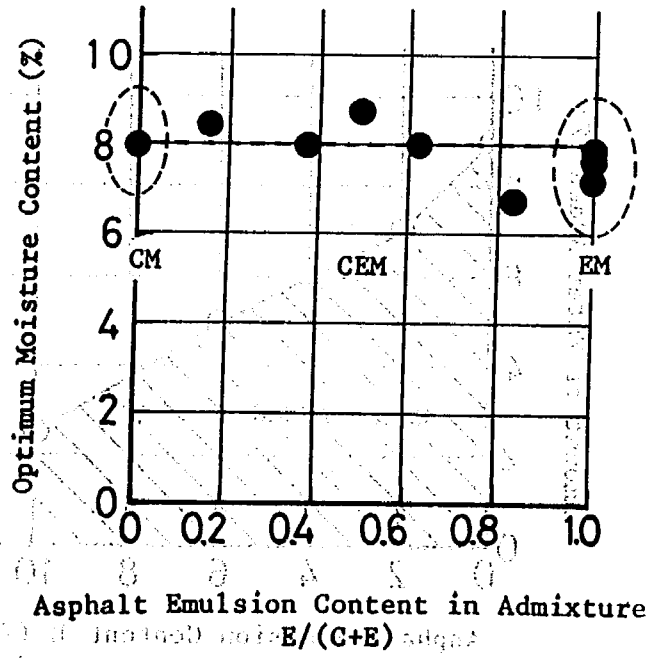

Fig. 5 Optimum Moisture Content of Cement-Asphalt Emulsion Mixtures

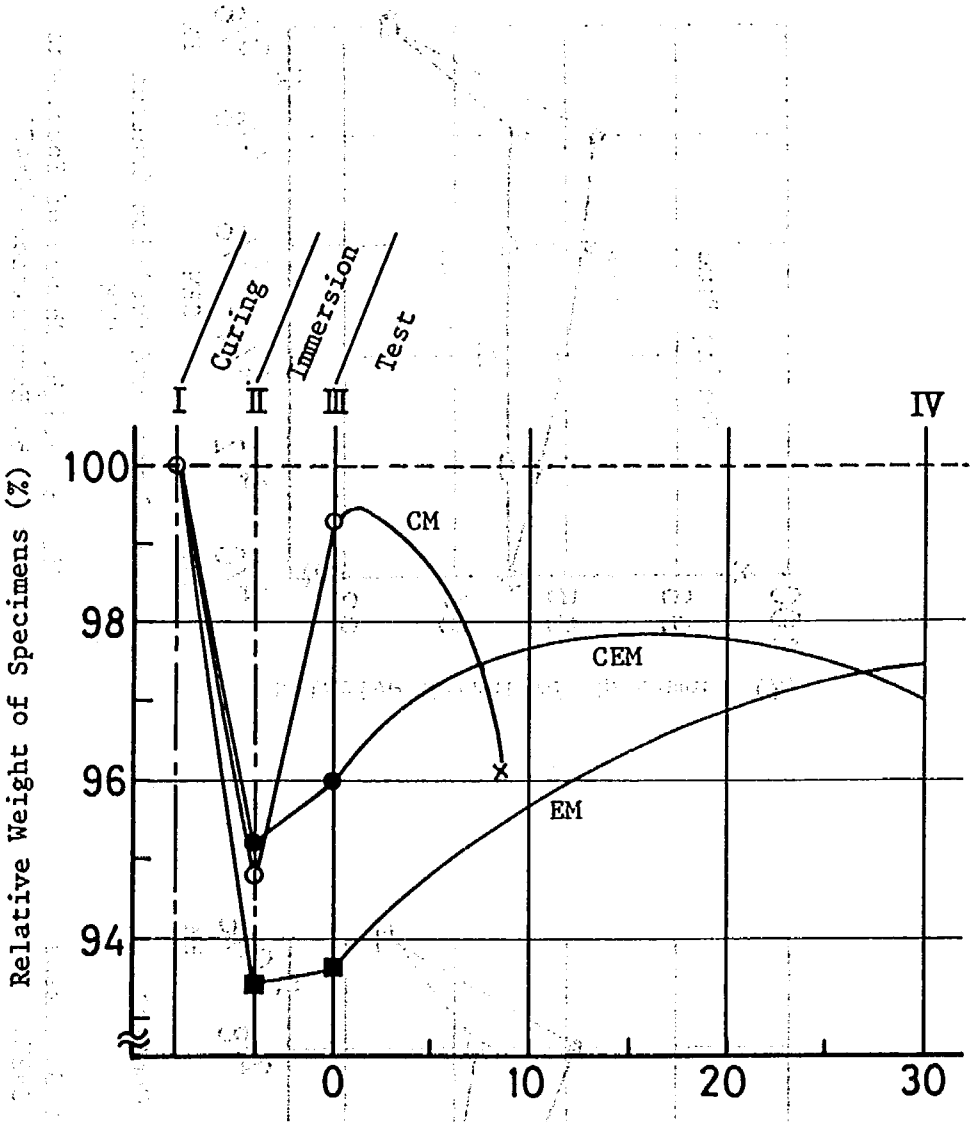

Fig. 6 Typical Trend of Weight Change of Specimens

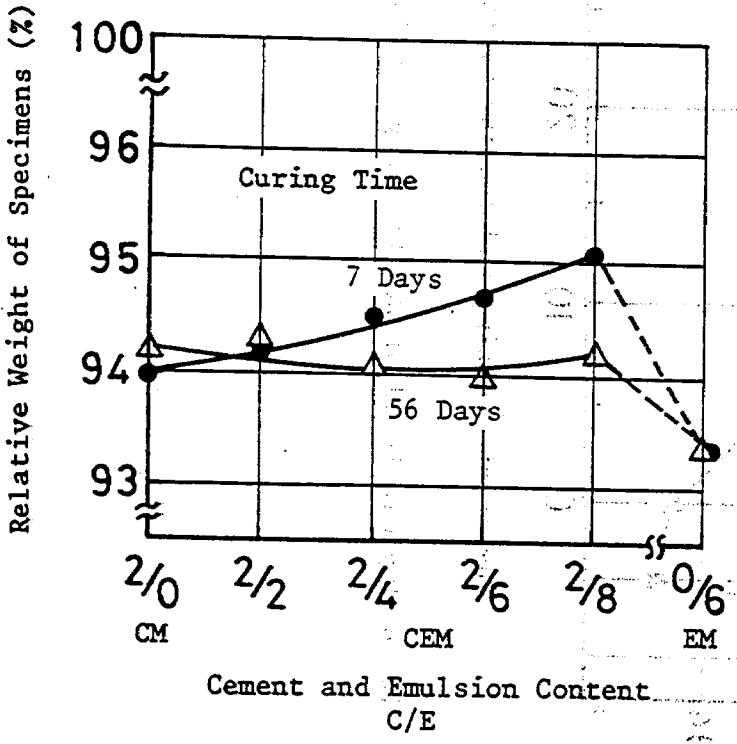

Fig. 7 Weight Change of Specimens by Evaporation (C=2%)

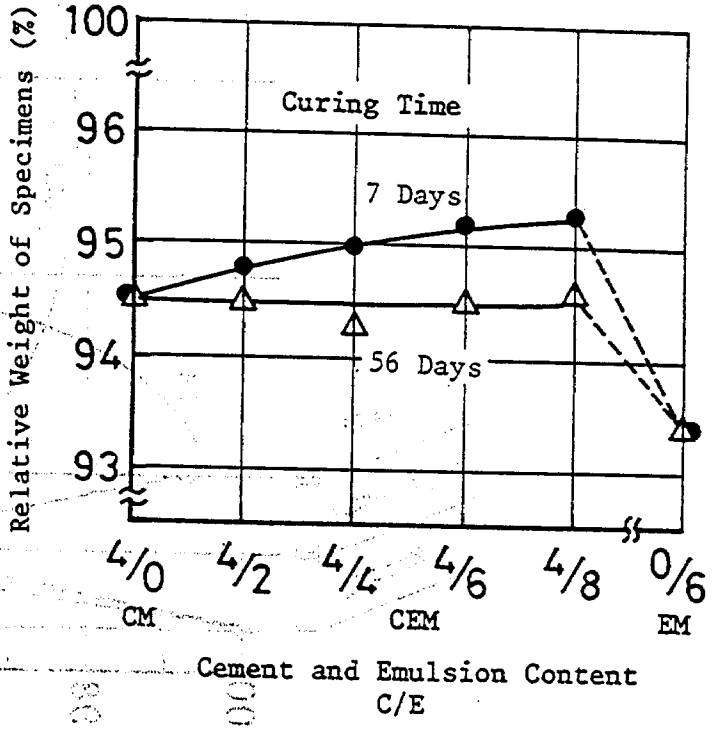

Fig. 8 Weight Change of Specimens by Evaporation (C=4%)

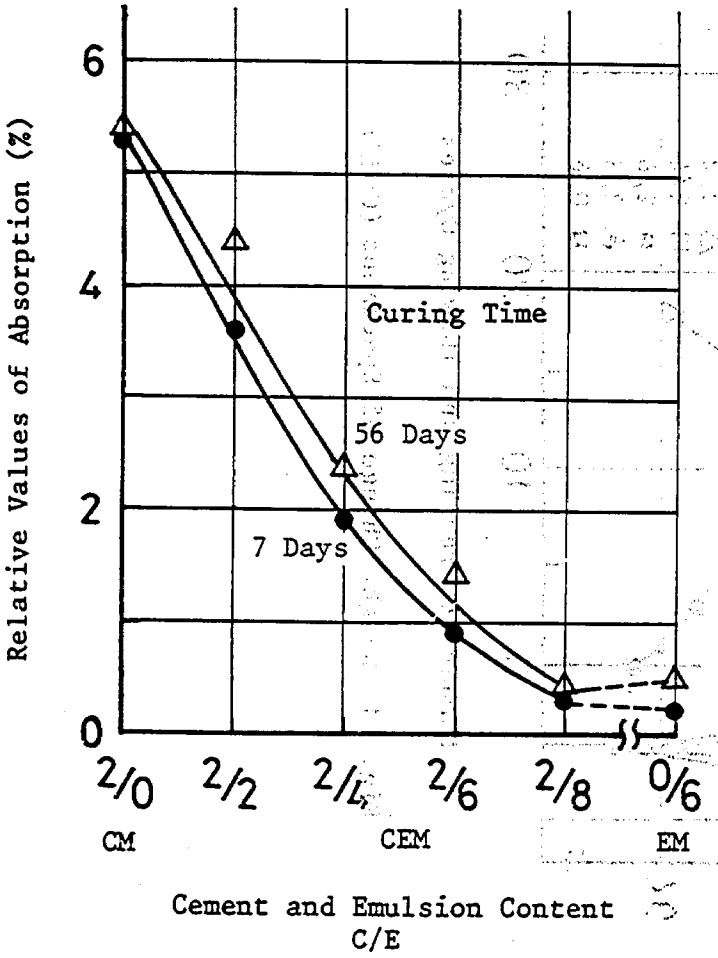

Fig. 9 Absorption Change of Specimens by Immersion (C=2%)

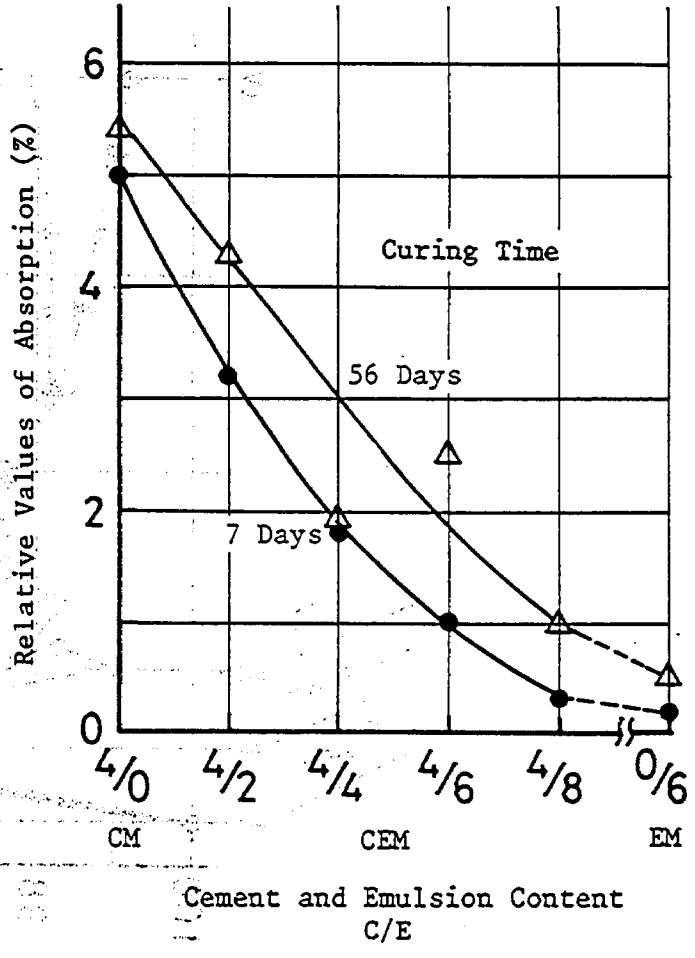

Fig. 10 Absorption Change of Specimens by Immersion (C=4%)

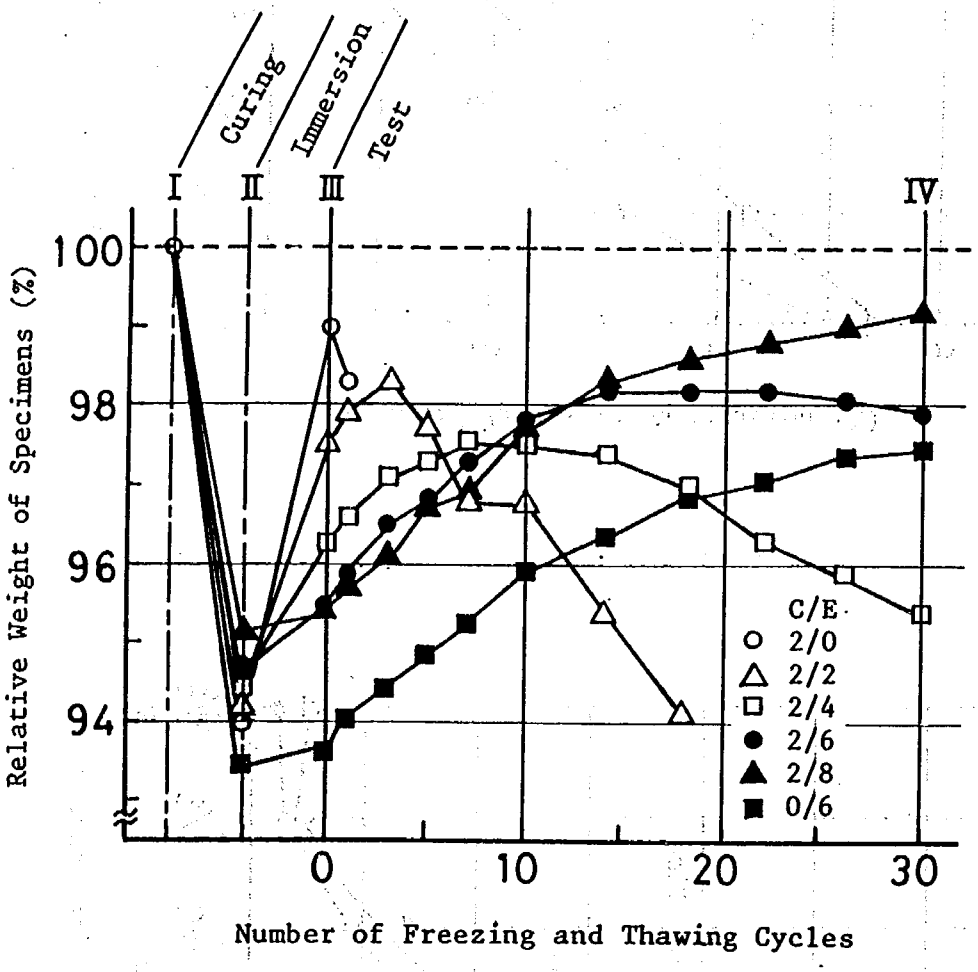

Fig. 11 Weight Change of Specimens (C=2%)

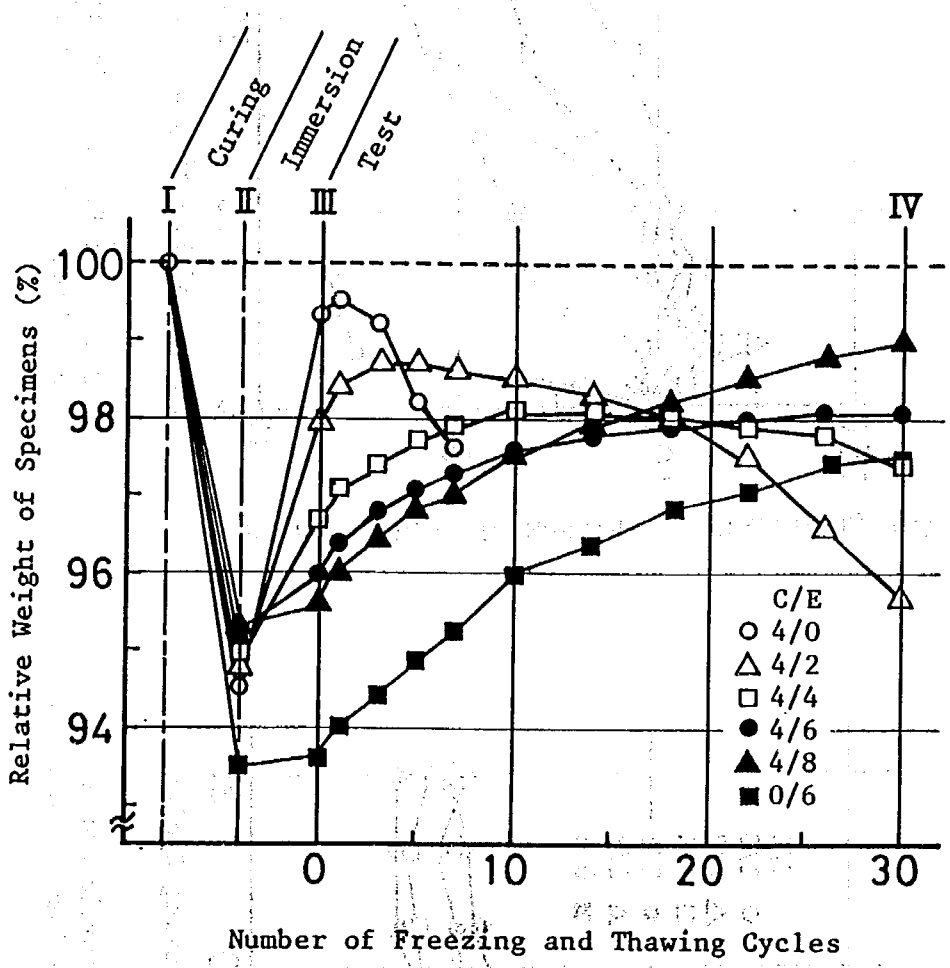

Fig. 12 Weight Change of Specimens (C=4%)

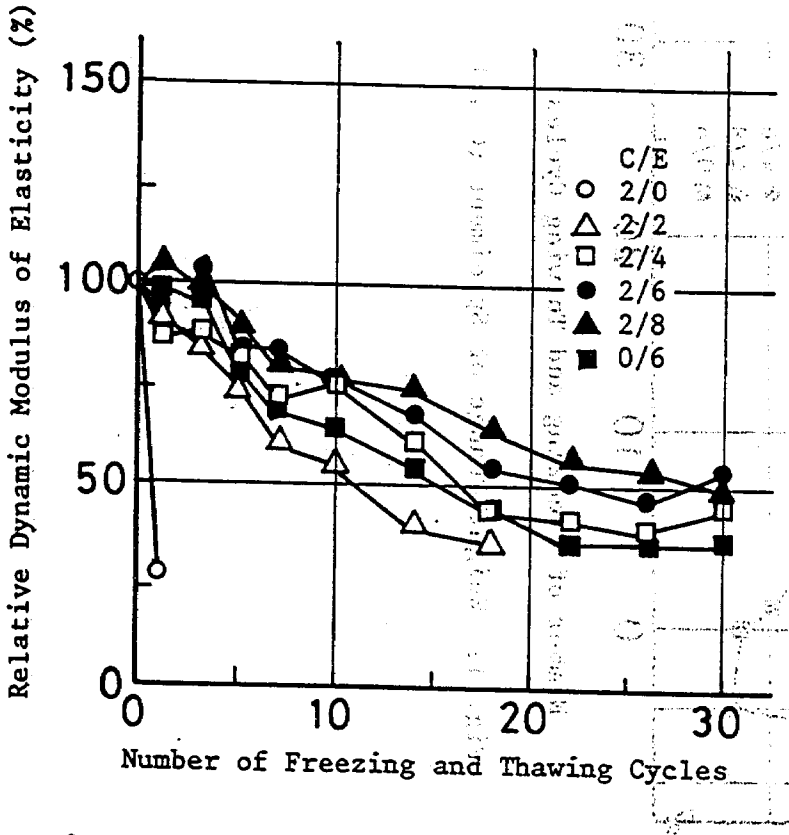

Fig. 13 Change of Relative Dynamic Modulus of Elasticity (Curing Time 7 Days, C=2%)

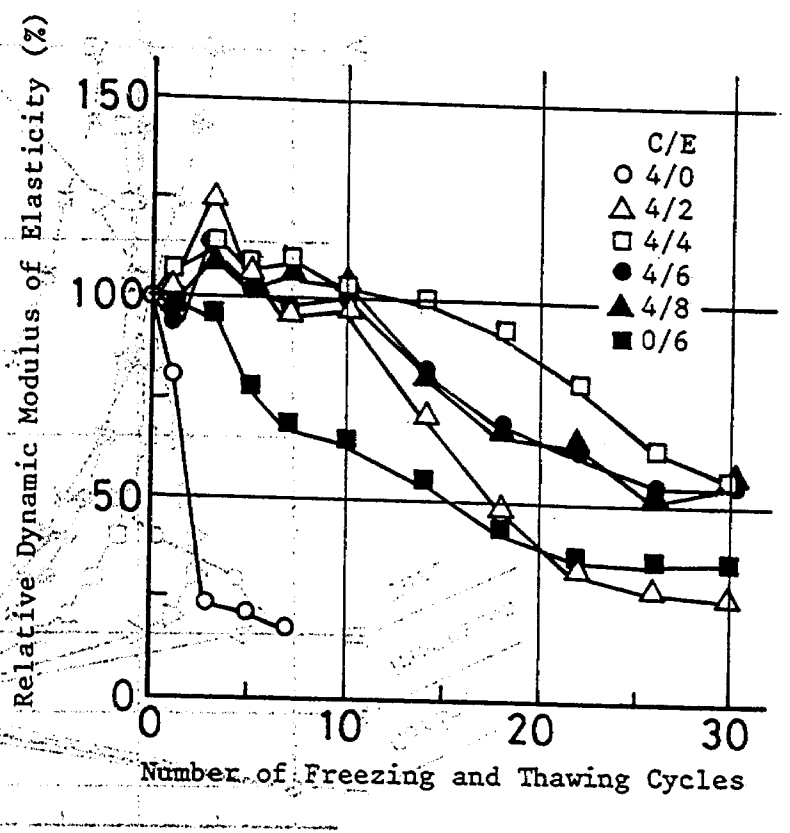

Fig. 14 Change of Relative Dynamic Modulus of Elasticity (Curing Time 7 Days, C=4%)

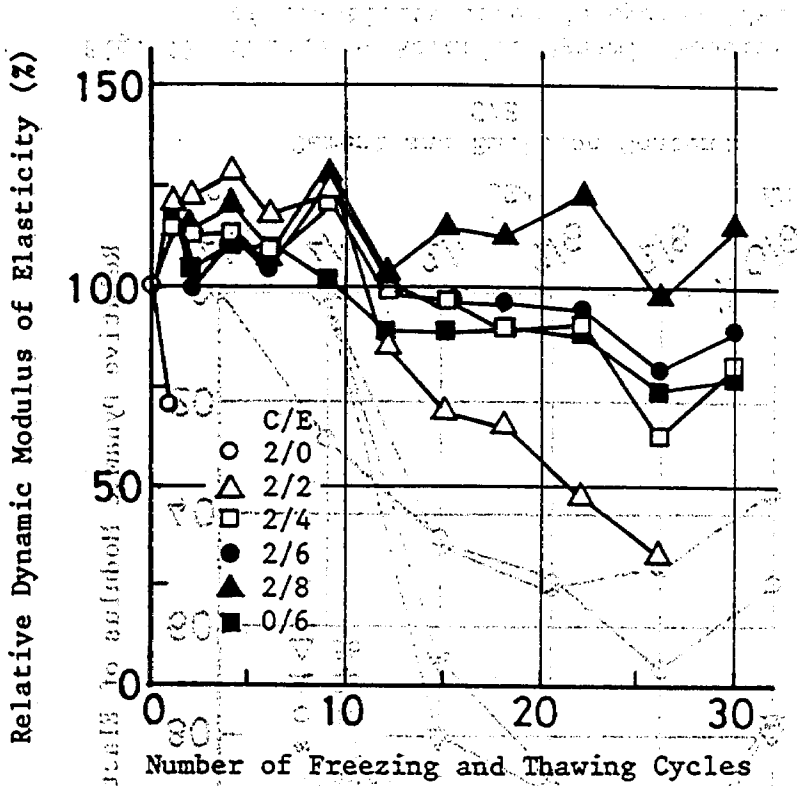

Fig. 15 Change of Relative Dynamic Modulus of Elasticity (Curing Time 56 Days, C=2%)

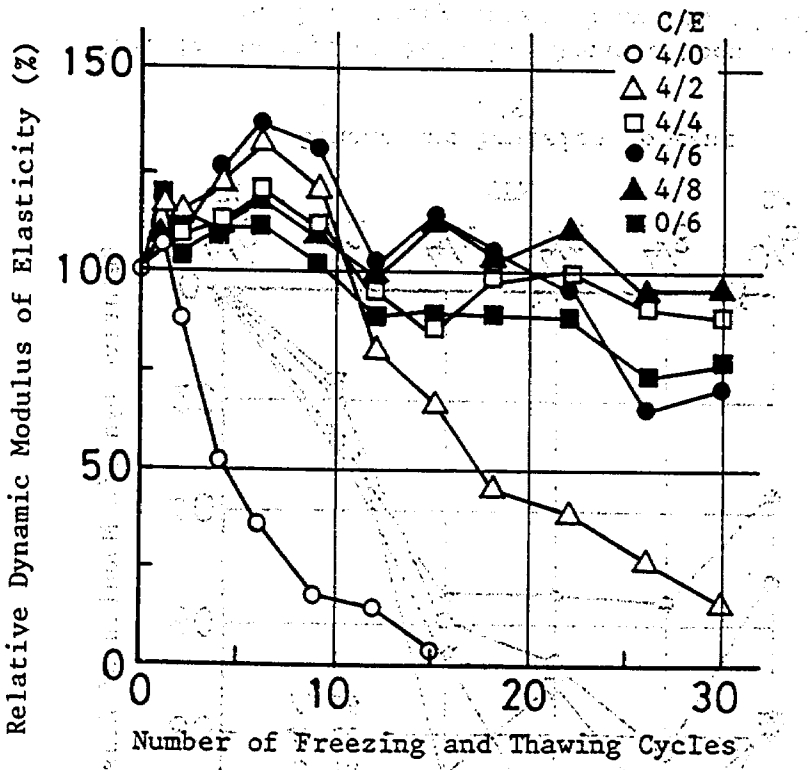

Fig. 16 Change of Relative Dynamic Modulus of Elasticity (Curing Time 56 Days, C=4%)

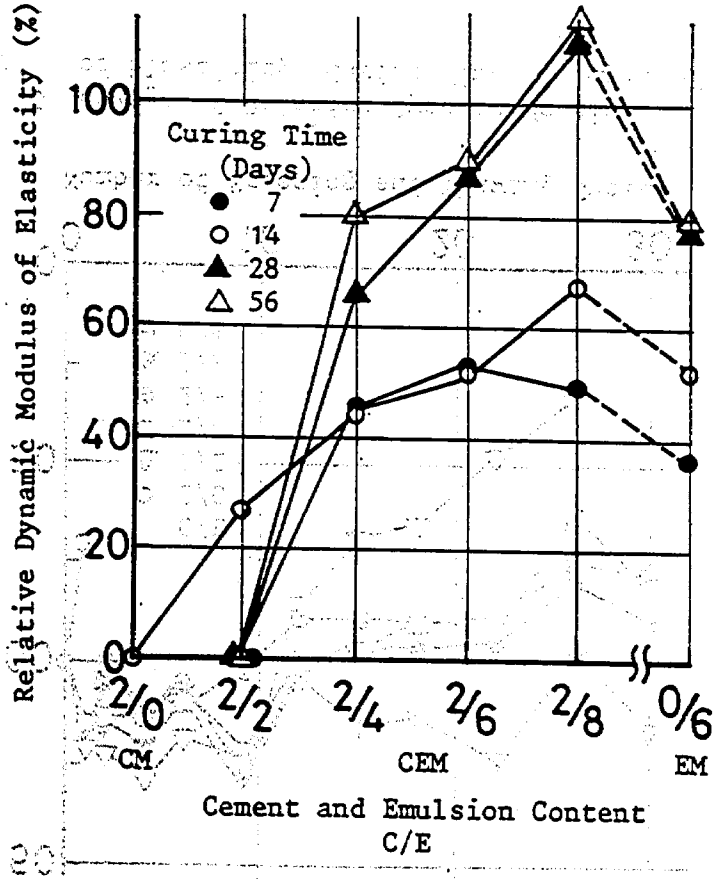

Fig. 17 Change of Relative Dynamic Modulus of Elasticity after 30 Cycles (C=2%)

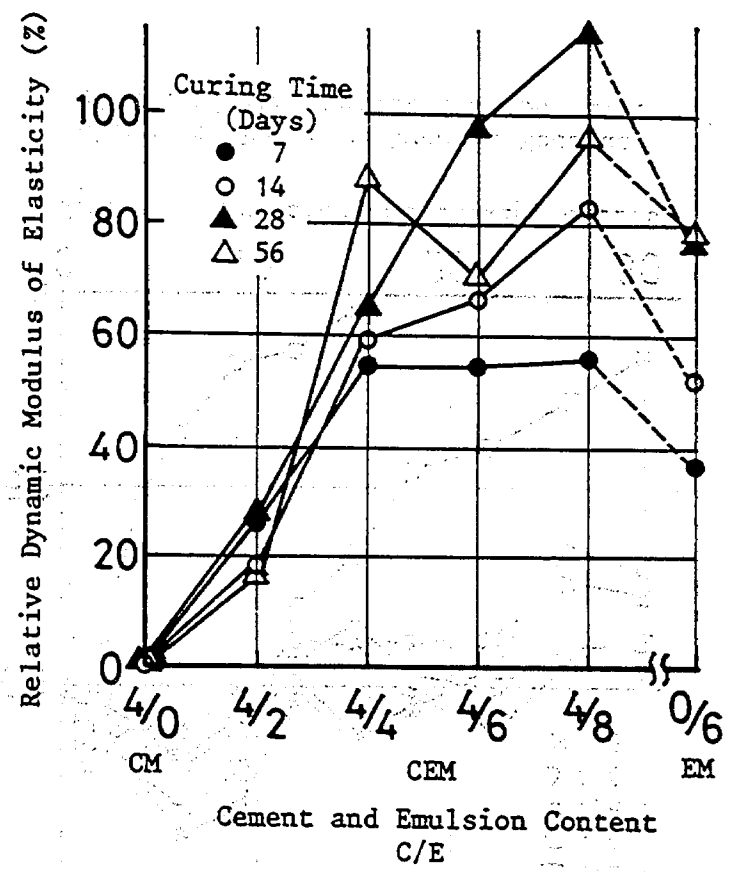

Fig. 18 Change of Relative Dynamic Modulus of Elasticity after 30 Cycles (C=4%)

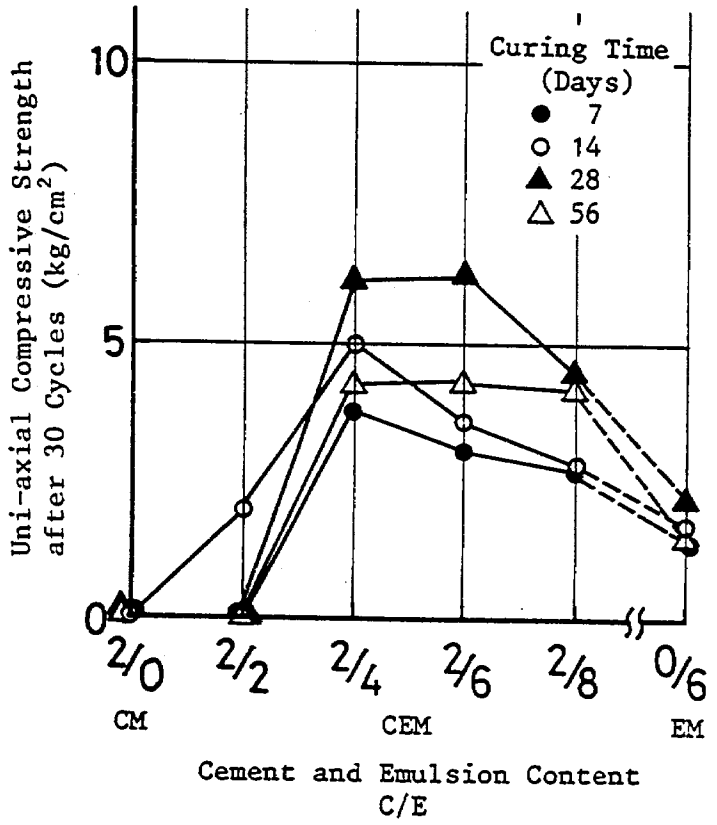

Fig. 19 Compressive Strength of Specimens after 30 Cycles (C=2%)

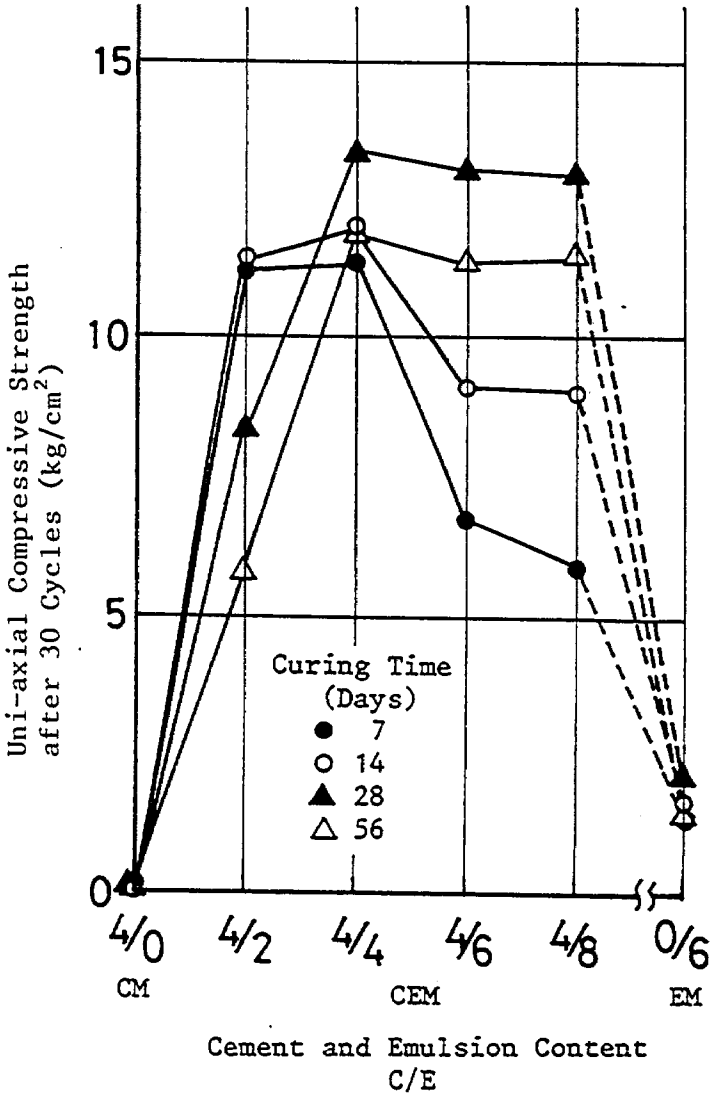

Fig. 20 Compressive Strength of Specimens after 30 Cycles (C=4%)

PROCEEDINGS OF PAVING IN COLD AREAS MINI-WORKSHOP, CANADA/JAPAN SCIENCE AND TECHNOLOGY CONSULTATIONS

土研資料第2136号
昭和59年10月
建設省土木研究所

キーワード

要旨

建設省土木研究所で開催された日加科学技術協議「寒冷地舗装第2回ミニワークショップ」の発表論文をとりまとめたものである。

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