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of PWRI No. 2897

PROCEEDINGS OF THE 4TH WORKSHOP ON PAVING IN COLD AREAS VOLUME 2

CANADA/JAPAN SCIENCE AND TECHNOLOGY AGREEMENT SEPTEMBER, 1990

Pavement Division
Road Department
Public Works Research Institute
Ministry of Construction

1, Asahi, Tsukuba-shi, Ibaraki-ken, 305 Japan

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by
Pavement Division, Road Department, PWRI

Synopsis:

This Technical Memorandum is the proceedings of the 4th workshop on "Paving in Cold Areas", held in Sapporo, September 4-6, 1990. There are 16 Japanese papers and 16 Canadian papers. The topics of the workshop are as follows:
-Paving Material for Cold Areas
-Low Temperature Cracking
-Development Testing Method for Paving Material and Performance in Cold Areas
-Deterioration of Pavement in Cold Areas and its Rehabilitation
-Concrete Pavement
-Evaluation of Pavement Structure Using Non-Destructive Method
-Management and Maintenance of Pavement

Key Words: Canada/Japan Science and Technology Agreement, Paving in Cold Areas, Modified Asphalt, Concrete Block, Coupling Agent, Low Temperature Cracking, Reology, Unti-Wear, Freezing Detector, Rutting, Mixture Design, Rehabilitation, Surface Recycling, Roller Compacted Concrete Pavement, Continuous Reinforced Concrete Pavement, Falling Weight Deflectometer, Surface Treatment, Pavement Management System

Foreword

The workshop on "Paving in Cold Areas", which is one of the important activity of the 4th Canada/Japan Sciece and Technology Agreement, will be held on September 4-6 in 1990. 16 Japanese papers and 16 Canadian papers were submitted to the Workshop. This Technical Memorandum is a proceedings, includes 15 papers which are to be presented in the latter half of the Workshop. There are a lot of useful information for designing and studying not only pavement in cold areas but also pavement in other areas.

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COMPARATIVE STUDY ON PERFORMANCES OF ASPHALT PAVEMENTS IN COLD AREA

by

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ABSTRACT

In this study, performance data contained in a technical report "Analysis on the Long-term Performance of Asphalt Pavements" published by the Public Works Research Institute was utilized. The data was dealt stochastically by a multivariate analysis method.

The main results are as follows:

(1) The indices representing the surface condition of asphalt pavements were related to the factors regarding pavement structure and traffic history in both cold and warm areas.

(2) Dominating factors of surface damages in cold areas were discussed.

(3) Comparing with the data of warm areas, characteristics of pavement damage in cold areas were discussed.

COMPARATIVE STUDY ON PERFORMANCES OF ASPHALT PAVEMENTS IN COLD AREA

by

Zuo Ming Wen, Yasushi Takeyama and Tadashi Fukuda
Faculty of Engineering, Tohoku University

1/ INTRODUCTION

The performance of an asphalt pavement is greatly affected by climatic conditions. Particularly, at cold areas of Japan, pavement surfaces are subjected to wear due to heavy traffic of studded tires and chained tires in the wintertime. Even in the cases of these cold areas, temperatures become fairly high in the summertime, and rutting occurs due to plastic deformation of asphalt.

This study was carried out on the performances of asphalt pavements in such cold areas, that is, the transitions in amounts of damage according to traffic history, and the modes of damage (rutting, cracking) making evaluations by comparing with cases of asphalt pavements in warm areas. With regard to correlation analyses of structural conditions and traffic conditions of asphalt pavements and their performances, Quantification Theory I was employed, as for discriminant analyses of modes of damage to asphalt pavements, Quantification Theory II was used.

2/ DATA USED

Data contained in a technical report, "Analysis on the Long-term Performance of Asphalt Pavements," [1] of the Public Works Research Institute of the Ministry of Construction were used in this study. These data were obtained on a continuous basis providing survey stations at 258 places on national highways throughout the country to collect basic data for maintenance and administration of asphalt pavements. These data are recorded classified according to areas in general and snow-covered areas depending on the location investigated. In this study, areas were divided into cold and warm, with data of areas in the Hokkaido, Tohoku, and Hokuriku Regions that get snow cover used for cold areas and data of general areas in the Chubu, Chugoku, Shikoku, and Kyushu Regions used for warm areas. The survey sites and the Regions are shown in Fig. 1.

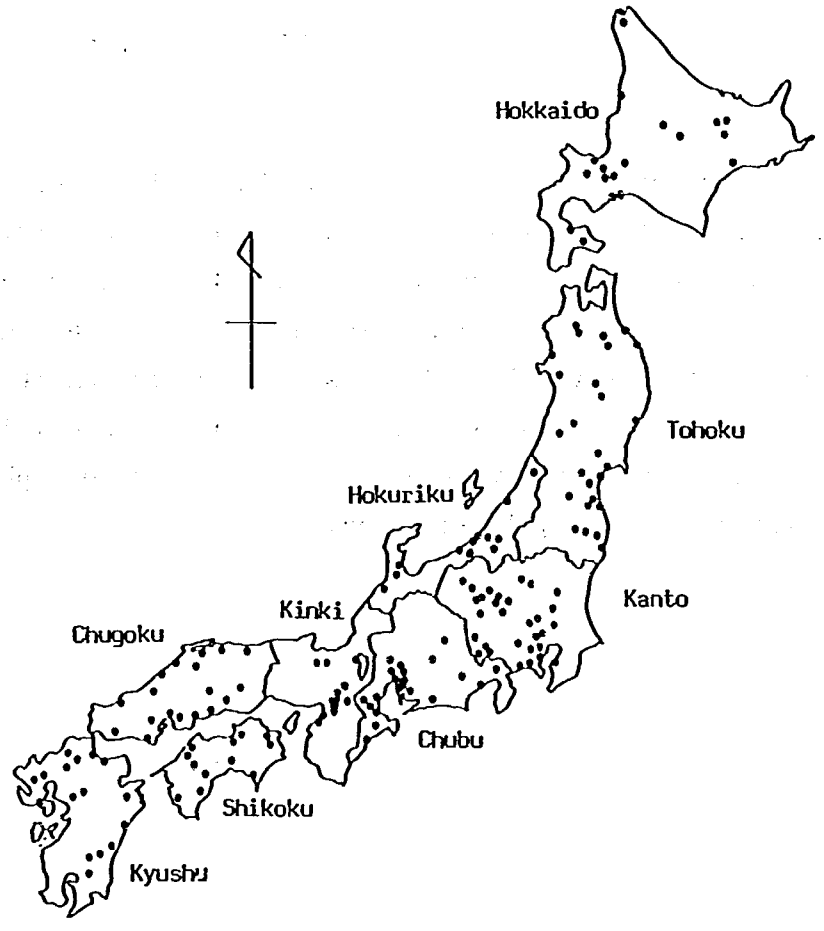


Fig. 1 Survey Sites [1]

3/ QUANTIFICATION THEORY ANALYSIS

Quantification Theory analysis is a technique of multivariate analysis with which not only numerical data, but also qualitative data can be dealt with. Quantification Theory I corresponds to multiple regression analysis, and the estimated value Y of the criterion variable is expressed as follows by employing a dummy variable X_{ij} for the explanatory variables which include qualitative data.

$$Y = \overline{Y} + \sum_{i=1}^{m} \sum_{j=1}^{n_i} X_{ij} A_{ij} \quad \cdots\cdots (1)$$

where, $\overline{Y}$: average value of criterion variables Y

m : number of items of explanatory variables

n_i : number of categories of individual items

X_{ij} : dummy variables

1 ; case of sample of criterion variables belonging to category j in item i

0 ; all other cases

A_{ij} : category weight

Quantification Theory II discriminates the group to which the sample belongs by the criterion variables of the sample. If this sample score is taken as the criterion variable Y, the relationship of the two may be expressed by the same form as Eq. (1) employing the dummy variable X_{ij} for the explanatory variable. Further, the category weight A_{ij} in this case is determined by the condition that the ratio between the dispersed value among the groups and the entire dispersed value becomes a maximum.

4/ ANALYSIS CONCERNING PERFORMANCE (QUANTIFICATION THEORY I)

4.1/ CATEGORY WEIGHT

The following performance index given in "The Manual for Road Maintenance and Repair" [2] was used as the overall evaluation index of the surface condition of an asphalt pavement.

$$PSI = 4.53 - 0.518 \log \sigma - 0.371 \sqrt{C} - 0.174 D^2 \quad \text{.....(2)}$$

where, PSI : Present Serviceability Index
σ : standard deviation of evenness in longitudinal direction (mm)
C : cracking ratio (%)
D : average depth of rutting (cm)

In the Quantification Theory I analysis, PSI, rutting, cracking, and evenness were employed as criterion variables. The 12 items of area, number of years elapsed, total traffic volume, traffic volume of heavy vehicles, cumulative traffic volume of heavy vehicles, design CBR value, total thickness, equivalent thickness T_A, type of surface course, thickness of asphalt mixture, base and subbase course thickness, and base course material were employed as explanatory variables in the initial analysis. Next, explanatory variables of narrow ranges and small partial regression coefficients in the analysis results were eliminated, and finally, analyses were performed concentrating to the 5 items given in Tables 1 and 2. Tables 1 and 2 show the analysis results of category weights concerning cold areas and warm areas, respectively.

4.2/ COMPARISONS OF ANALYSIS RESULTS

Using the category weights in Tables 1 and 2, it is possible to calculate time-dependent changes in PSI, rutting, cracking, evenness, etc. by Eq. (1). Fig. 2 shows calculations made of the following case as an example:

Input data

Traffic volume of heavy vehicles : 1,000 - 2,000 / day
Equivalent thickness (T_A) : 20 - 30 cm
Base course type : asphalt stabilization

Table 1 Results of Quantification Theory I Analyses (Cold Areas)

Region		Hokkaido				Tohoku				Hokuriku			
Number of Data		102				113				77			
Criterion Valiable		PSI	Rutting	Cracking	Evenness	PSI	Rutting	Cracking	Evenness	PSI	Rutting	Cracking	Evenness
Mean Value		3.540	19.157	0.259	1.464	3.335	21.195	0.340	1.866	3.756	13.481	1.216	1.569
Multiple Correlation Coefficient		0.721	0.779	0.731	0.540	0.721	0.767	0.474	0.798	0.781	0.808	0.509	0.695
Item	Category												
Numbers of Years	< 2	0.584	-7.085	-0.223	-0.104	0.749	-8.836	-0.325	-0.222	0.580	-7.123	-0.955	-0.212
	2 - 4	0.172	-2.111	-0.048	-0.092	0.432	-3.271	-0.324	-0.138	0.314	-3.084	-0.638	-0.044
	4 - 6	-0.225	2.849	0.104	0.027	-0.335	3.851	0.089	0.008	-0.259	3.181	0.418	0.079
	6 - 8	-0.667	7.934	0.201	0.230	-0.901	8.321	0.658	0.416	-0.908	9.561	1.767	0.215
Traffic Volume of Heavy Vehicles	< 1000	0.594	-7.737	-0.143	-0.053	0.328	-4.469	0.160	0.090	0.390	-4.366	-0.608	-0.171
	1000 - 2000	-0.163	2.336	0.080	0.045	-0.160	1.932	0.011	-0.070	0.119	-1.031	-0.287	0.030
	2000 <	-0.760	9.410	0.084	-0.005	-0.202	3.294	-0.295	0.003	-0.410	4.146	0.790	0.059
TA	< 20	-0.493	6.355	0.230	-0.077	-0.222	2.882	0.592	-0.992	-0.420	5.674	-0.457	0.873
	20 - 30	-0.450	4.901	-0.004	0.255	0.123	-1.302	-0.139	0.095	0.004	0.047	0.150	-0.124
	30 <	0.720	-8.849	-0.144	-0.200	-0.779	7.824	0.624	0.018	0.328	-5.290	-0.976	0.382
Surface Courses	Coarse Graded	—	—	—	—	-0.503	5.242	0.142	-0.209	—	—	—	—
	Dense Graded	0.307	-4.934	-0.062	0.217	-0.724	8.145	0.602	-0.400	—	—	—	—
	Revised Topeka	—	—	—	—	0.325	-2.979	-0.479	-1.377	-0.575	7.507	-0.797	0.514
	Topeka	—	—	—	—	0.151	-1.407	0.052	0.491	—	—	—	—
	Dense-and-gap	0.275	-0.252	-0.072	-0.247	—	—	—	—	—	—	—	—
	Fine-and-gap	-0.052	0.722	0.015	-0.009	0.208	-1.935	-0.232	-0.189	0.086	-1.120	0.119	-0.077
	Other	-0.033	-2.557	-0.020	0.144	0.022	-2.025	-0.004	0.333	—	—	—	—
Base Courses (Type of Stabilization)	Asphalt	-0.052	1.016	-0.080	-0.022	-0.120	1.929	-0.029	-0.321	-0.013	-0.285	0.019	0.128
	Mechanical	-0.039	0.745	-0.233	0.075	0.319	-5.078	0.076	0.849	0.048	1.006	-0.066	-0.453
	Cement	0.580	-11.4	1.241	0.061	—	—	—	—	—	—	—	—

Table 2 Results of Quantification Theory I Analyses (Warm Areas)

Region		Chubu				Chugoku				Shikoku				Kyushu			
Number of Data		152				61				127				105			
Criterion Valiable		PSI	Rutting	Cracking	Evenness	PSI	Rutting	Cracking	Evenness	PSI	Rutting	Cracking	Evenness	PSI	Rutting	Cracking	Evenness
Mean Value		3.721	11.480	4.041	1.503	3.727	12.902	1.400	2.346	4.021	8.417	1.387	1.658	3.882	8.238	1.927	1.862
Multiple Correlation Coefficient		0.485	0.543	0.354	0.325	0.666	0.543	0.353	0.569	0.614	0.438	0.509	0.429	0.638	0.562	0.495	0.465
Item	Category																
Numbers of Years	< 2	0.543	-4.131	-3.701	-0.330	0.264	-3.927	-0.419	-0.068	0.358	-1.402	-1.674	-0.322	0.395	-3.196	-1.896	-0.124
	2 - 4	0.218	-1.088	-1.809	-0.138	0.095	-1.149	-0.449	-0.064	0.098	-0.534	-0.498	0.046	0.063	-1.347	-0.032	0.033
	4 - 6	-0.182	0.604	2.418	0.089	-0.167	2.072	0.810	0.312	-0.013	0.188	0.055	-0.061	0.054	-0.397	-0.383	-0.233
	6 - 8	-0.426	3.540	1.793	0.293	-0.180	2.771	0.069	-0.287	-0.170	0.661	0.845	0.096	-0.347	3.400	1.600	0.313
Traffic Volume of Heavy Vehicles	< 1000	-0.048	-2.699	1.934	0.006	0.234	-0.706	-1.061	-0.360	-0.095	0.084	0.685	0.158	0.358	-3.218	-1.950	-0.444
	1000 - 2000	-0.035	-1.000	0.501	0.139	-0.057	0.173	0.260	0.088	0.117	-0.103	-0.841	-0.194	0.022	-0.388	0.007	0.010
	2000 <	0.019	0.854	-0.562	-0.034	—	—	—	—	—	—	—	—	-0.229	2.316	1.070	0.233
TA	< 20	—	—	—	—	—	—	—	—	-0.050	-2.401	1.011	-0.133	—	—	—	—
	20 - 30	0.202	-0.328	-2.613	-0.051	0.040	-0.759	0.287	0.243	0.051	2.440	-1.027	0.135	0.025	1.035	1.884	0.070
	30 <	-0.102	0.166	1.319	0.026	-0.083	1.556	-0.589	-0.498	—	—	—	—	0.307	-1.278	-2.325	-0.087
Surface Courses	Coarse Graded	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Dense Graded	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	Revised Topeka	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Topeka	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Dense-and-gap	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Fine-and-gap	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	Other	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Base Courses (Type of Stabili-zation)	Asphalt	0	0	0	0	0.182	-1.265	-0.571	-0.309	0.122	-2.944	0.152	-0.266	-0.197	-0.220	1.400	-0.329
	Mechanical	—	—	—	—	-0.262	1.822	0.823	0.444	-0.084	2.401	-0.105	0.184	0.293	0.277	-1.025	0.086
	Cement	—	—	—	—	—	—	—	—	—	—	—	—	-0.191	-0.651	1.703	0.113

Surface course material : Fine and gap-graded asphalt concrete
(case of cold area)
Dense-graded asphalt concrete
(case of warm area)

The following may be comprehended from Fig. 2:

(1) At cold areas, rutting is severer and cracking lighter compared with warm areas. The order among cold areas in severity of rutting is Hokkaido > Tohoku > Hokuriku, while with regard to cracking, it is worst in Hokuriku among the three Regions.

(2) Regarding evenness, there is practically no difference between warm and cold areas, with the exception of the case of the Chugoku Region.

(3) Concerning decline in PSI, it is greater in general at cold areas compared with warm areas. Parenthetically, it is shown that in cold areas rutting caused by studded tires and chained tires in the wintertime has a great influence on performance of asphalt pavement.

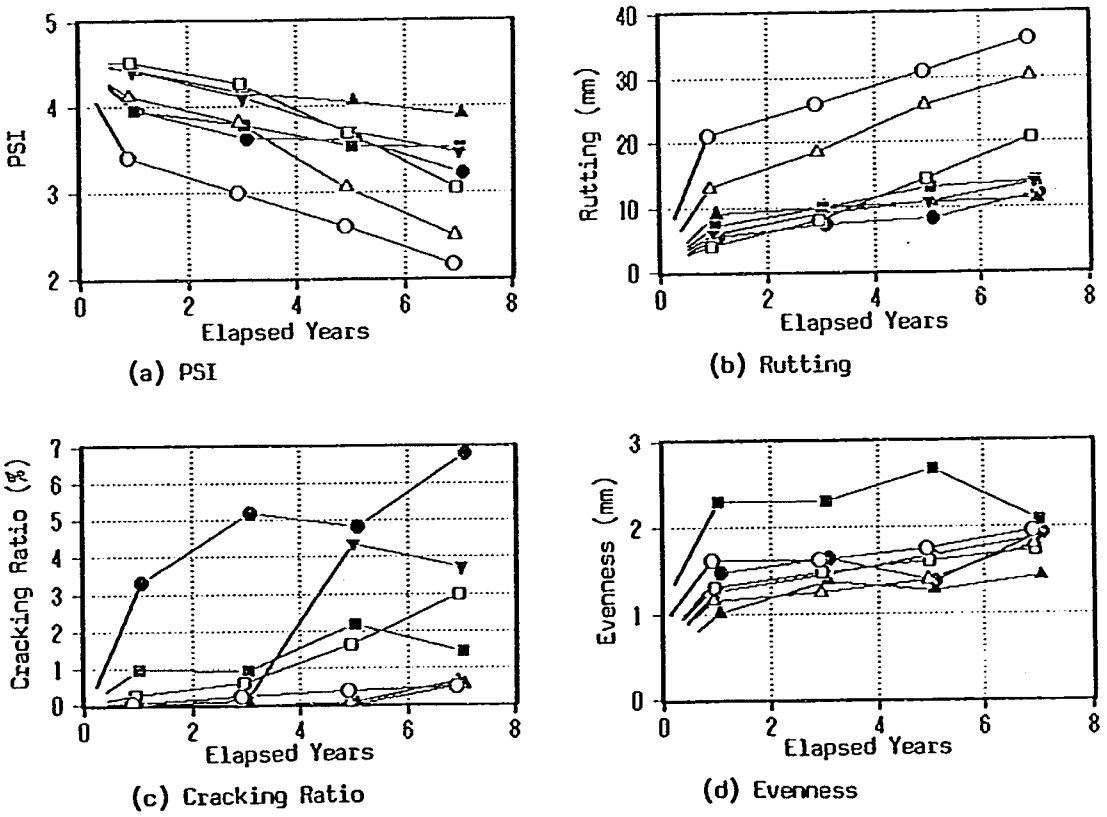


Fig. 2 Time-dependent Change in Criterion Variables

5/ DISCRIMINANT ANALYSIS OF DAMAGE MODE (QUANTIFICATION THEORY II)

5.1/ CATEGORY WEIGHT

The PSIs of cold areas and warm areas are greatly influenced by the modes of damage of rutting and cracking as can be seen from the results of the analyses in 4. Thereupon, a conditional equation by which identical effects would be brought about by both rutting and cracking was sought by Eq. (2), and the following was obtained:

$$C = 0.220\ D^4 \quad \text{.....(3)}$$

The groups to which samples belonged were classified according to rutting and cracking by Eq. (3), and Quantification Theory II analyses were performed. Because of limitations in the number of data, the analyses were made including "area" among the factors. The results of category weight analyses are given for cold areas and warm areas in Tables 3 and 4, respectively.

Table 3 Results of Quantification Theory II Analyses (Cold Areas)

Numbers of data belonged "Rutting" Group: 266 Mean: 0.0303
"Cracking" Group: 29 Mean: 0.1180

Weighted Mean $\bar{Y}$ = 0.0388

Item	Category	Category Weight	Range	Partial Regression Coefficient
Region	Hokkaido	-0.030	0.1129	0.3521
	Tohoku	-0.0247		
	Hokuriku	0.0799		
Numbers of Years	< 2	-0.0267	0.0493	0.1550
	2 - 4	-0.0077		
	4 - 6	0.0226		
	6 - 8	0.0078		
Traffic Volume of Heavy Vehicles	< 1000	0.0148	0.0260	0.0834
	1000 - 2000	-0.0045		
	2000 <	-0.0111		
T_A	< 20	-0.0126	0.0636	0.2006
	20 - 30	-0.0126		
	30 <	0.0510		
Surface Course	Coarse/Dense Graded	0.0199	0.1103	0.2304
	Topeka, Revised Topeka	0.0010		
	Dense/Fine-and-gap	-0.0141		
	Other	0.0962		
Base Course (Type of Stabilization)	Asphalt	-0.0015	0.2046	0.2590
	Mechanical	-0.0181		
	Cement	0.1865		

Table 4 Results of Quantification Theory II Analyses (Warm Areas)

Numbers of data belonged "Rutting" Group: 219 Mean: -0.0388
"Cracking" Group: 216 Mean: 0.0027

Weighted Mean $\bar{Y}$ = -0.0186

Item	Category	Category Weight	Range	Partial Regression Coefficient
Region	Chubu	-0.0094	0.0921	0.2787
	Chugoku	0.0249		
	Shikoku	-0.0424		
	Kyushu	0.0497		
Numbers of Years	< 2	-0.0610	0.0959	0.3112
	2 - 4	-0.0210		
	4 - 6	0.0197		
	6 - 8	0.0349		
Traffic Volume of Heavy Vehicles	< 1000	0.0114	0.0184	0.0670
	1000 - 2000	0.0003		
	2000 <	-0.0070		
T_A	< 20	0.0687	0.0910	0.2454
	20 - 30	-0.0222		
	30 <	-0.0054		
Thicness of Surface Course	< 10	0.0028	0.0318	0.1143
	10 - 15	-0.0148		
	15 <	0.0169		
Base Course (Type of Stabilization)	Asphalt	0.0057	0.0169	0.0638
	Mechanical	-0.0100		
	Cement	0.0068		

Suface Course Material is Dense Graded Asphalt Concrete

5.2/ COMPARISONS OF ANALYSIS RESULTS

It is judged using Eq. (3) which of rutting and cracking is predominant as the mode of damage to asphalt pavements at the locations surveyed. Next, the sample scores for these pavements are calculated using the category weights in Tables 3 and 4, and when the frequency distributions are illustrated according to cold and warm areas, the results will be as shown in Fig. 3. According to this figure, cracking is predominant if the sample score is increased, although not so clearly in warm areas, where in many cases it is indicated that the two modes of damage occur intermixed.

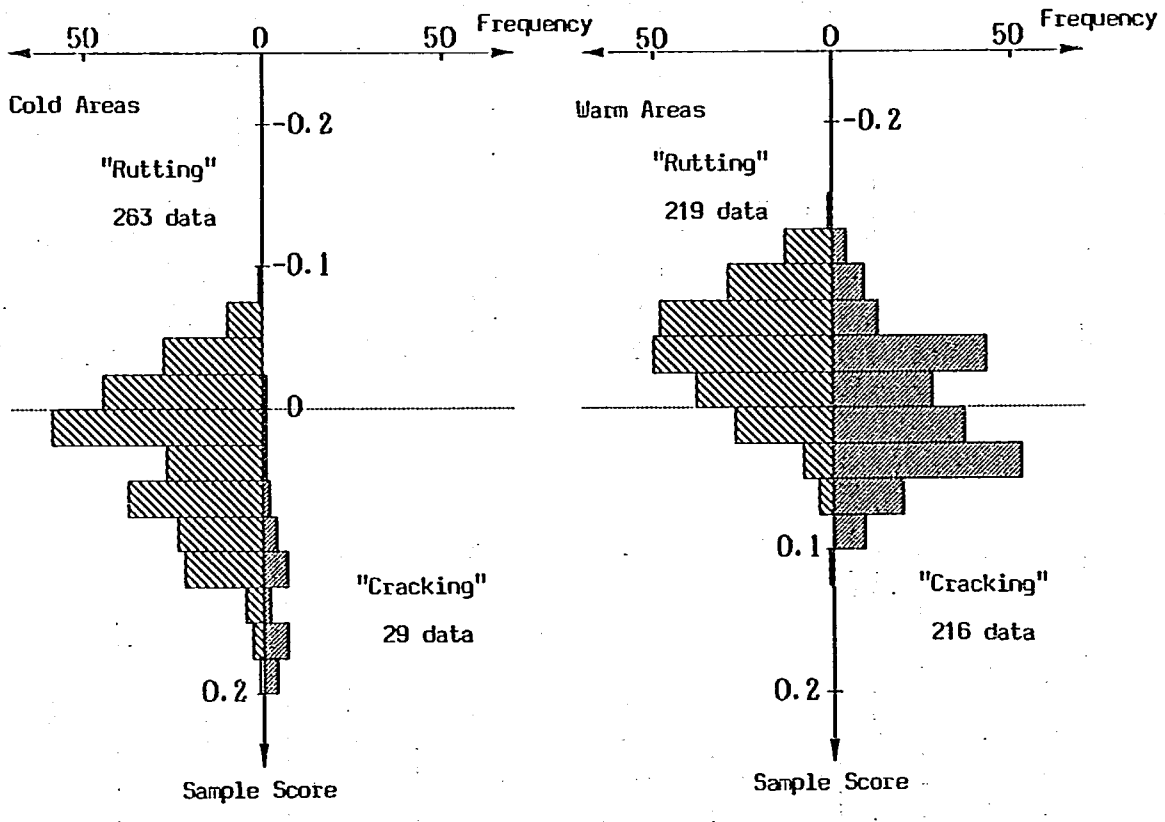


Fig. 3 Frequency Distributions of Sample Scores by Mode of Damage

6/ PREDICTION OF DAMAGE CONDITION

The damage condition of an asphalt pavement which has been designed can be predicted using the results obtained in this study, In short, prediction of the damaged condition of the pavement is made by Quantification Theory II analysis, and next, the amount of the damage is predicted by Quantification Theory I analysis. An example is given below.

The design conditions are identical to those when obtaining Fig. 2.

(1) The damaged condition after opening to traffic will be as shown in Fig. 4.

(2) The time-dependent changes in the amounts of damage seen in Fig. 4 will be as shown in Fig. 2.

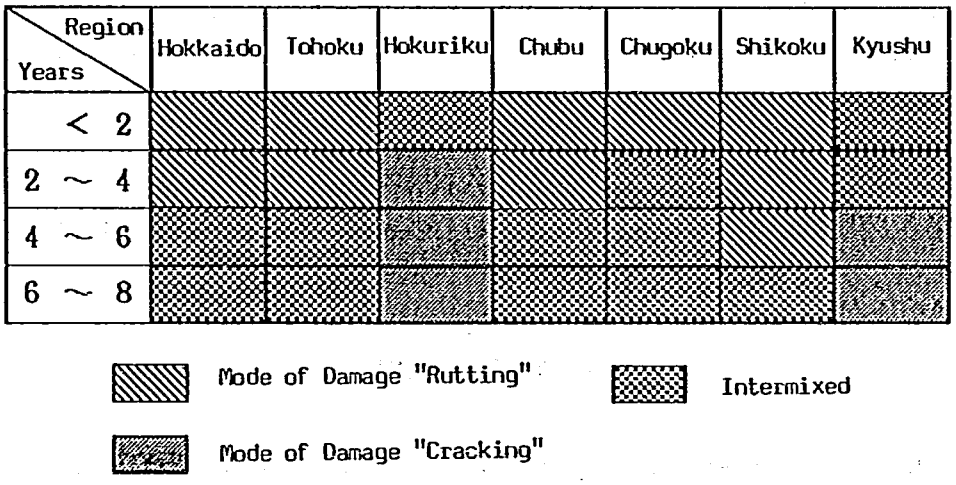


Fig. 4 Estimates of Modes of Damage

7/ CONCLUSIONS

The national territory of Japan is long and narrow in a north-south direction, and consequently, the climatic conditions prevailing on pavements differ greatly depending on the areas. Especially, in cold areas, in addition to the plastic deformation of asphalt in the summertime, rutting of road surfaces is prominent due to wear by studded tires in wintertime traffic. This study was carried out applying Quantification Theory to the results of surveys of 258 stations on national highways throughout the country by the Ministry of Construction to compare and evaluate performances of asphalt pavements in warm and cold areas. The principal conclusions drawn are as listed below.

(1) Asphalt pavement in cold areas are damaged exceedingly by rutting , and therefore, service lives are shorter compared with cases in warm areas.

(2) With asphalt pavements in cold areas, either rutting or cracking is predominant as the mode of damage depending on factors such as area, traffic, and structure, whereas in warm areas, the two modes of damages occur intermixed in many cases.

(3) An example of predicting damage to an asphalt pavement applying the results of the present study has been given.

ACKNOWLEDGMENT

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Design of Asphalt Concrete Mixtures in Cold Climates Using Soft Asphalts

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DESIGN OF ASPHALT CONCRETE MIXTURES IN COLD CLIMATES USING SOFT ASPHALTS

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ABSTRACT

The Province of Alberta is located in western Canada. Southern Alberta borders the northern United States and the province extends about 1200 km to the north. Maximum summer temperatures range from 30 to 35°C and minimum winter temperatures range from minus 40 to minus 45°C across the province. These extreme variations in temperature significantly effect the performance of asphalt concrete pavements and create unique engineering challenges in designing asphalt mixtures and pavements that will provide acceptable performance at both high and low temperatures.

Asphalt specifications in Alberta have evolved to address low temperature transverse cracking performance and preclude the use of highly temperature susceptible asphalts. High viscosity asphalt grades used range in penetration from 150-200 to 300-400.

Two major investigations have been carried out by Alberta Transportation & Utilities (AT&U) which have focused on evaluating the influence of asphalt cement characteristics on both the low temperature performance, ie. transverse cracking and the high temperature performance, ie. rutting of asphalt concrete pavements constructed across Alberta.

High and low temperature performance was evaluated on a wide variety of pavements in terms of the in-place as well as the original, as supplied asphalt characteristics, pavement structure, traffic loadings, climatic effects, etc. Mathematical models were developed that described rutting and low temperature cracking performance in terms of asphalt cement stiffness and other factors.

Recently developed design criteria are presented that will assist AT&U pavement designers in specifying asphalt concrete mixture parameters in order that the high and low temperature performance of asphalt concrete pavements can be optimized. The selection of these parameters, which include asphalt cement grade, aggregate characteristics and other factors, is based upon projected traffic loadings and climatic factors.

1.0 Introduction

In the sixties and early seventies, considerable efforts by researchers and highway engineers were expended to investigate low temperature transverse cracking. This research strongly influenced the development of asphalt cement specifications to address this important performance requirement and, in Alberta, resulted in the use of softer and more viscous asphalt binders and precluded the use of highly temperature susceptible asphalts.

More recently there has been an increasing concern across North America with permanent deformation, or rutting, of asphalt concrete pavements. This concern becomes even more serious as traffic volumes, axle weights and tire pressures continue to increase. This issue is further complicated as current mix design procedures (Marshall and Hveem) do not adequately predict pavement rutting resistance.

These issues provide significant challenges to the highway engineer in designing asphalt concrete pavements and mixes that will provide acceptable performance at both high and low temperatures.

This paper presents a summary of two major investigations carried out by Alberta Transportation and Utilities (AT&U) as part of a cooperative research program with the University of Alberta. These studies have focused primarily on evaluating the influences of asphalt cement characteristics on both the low and high temperature performance of a wide variety of pavements constructed and performing across Alberta.

Also presented are criteria which have developed from the findings of this research which are being used in the process of designing asphalt concrete mixes. These design criteria provide AT&U pavement designers with a more rational approach for selecting asphalt cement grades and aggregate and other mix design parameters based upon anticipated traffic loadings and climatic factors for Alberta conditions.

2.0 BACKGROUND ON ALBERTA CONDITIONS

This chapter presents an overview of climatic, traffic, pavement structures, aggregates and asphalts used in Alberta. Also presented is a brief summary of the evolution of asphalt specifications in Alberta as well as past asphalt mix design practices. This chapter will provide the reader with a background on Alberta conditions, which of course had significant effects on the development of the design guidelines and criteria presented further on in the paper.

2.1 Geography and Climate

Alberta is located in western Canada. Southern Alberta borders the northern United States (latitude 49 degrees North) and the province extends about 1200 km to the north. The area of the province is about 650 000 sq.km. (as compared to Japan's area of about 365 000 sq.km.).

Maximum summer temperatures range from 30 to 35°C and minimum winter temperatures range from minus 40 to minus 45°C across the province. The average Freezing Index ranges from 2800°C-days in the north to 800°C-days in the south. Rainfall varies significantly over the province with the dry southeast corner of the province experiencing an average yearly rainfall of about 300 mm and some areas in north central Alberta experiencing an average annual rainfall of up to 600 mm.

2.2 Network, Traffic and Structural Design

AT&U has been responsible, over the last 35 years or so, for the design and construction of two separate networks of paved roads. There is the Primary Highway network, which at present consists of about 14 000 km of paved road. Additionally there is the Secondary Highway network of which about 5 000 km is paved.

The Primary Highway network is the main intra-provincial transportation system and AADT levels can reach 35 000 on major corridors. The Secondary Highway network provides a more local "farm to market" distribution

system and traffic levels are much lower.

Pavements are designed, from a structural perspective, to accommodate the maximum axle loads allowed by law, at least for the majority of the year. AT&U practice is not to load ban fully paved Primary Highways. However, Secondary Highways are often banned, ie. maximum load limits are reduced, for a period in the spring if strength testing indicates a necessity.

Design and performance experience has resulted in the general practice of "staged base construction" using untreated aggregate base courses and a standard 50 mm asphalt stabilized base course wearing surface. From one to four years after the construction of the base, a final hot mix asphalt concrete pavement is designed and constructed to achieve a 20 year pavement design life.

Almost all pavement structures utilize untreated granular base courses although Portland cement stabilized sand base courses are used in areas of poor gravel supply. Full depth asphalt concrete pavements were used extensively in the seventies but, based upon performance problems and more recently economic circumstances, are not used anymore.

2.3 Construction Practice

All construction is carried out by private contractors. Contracts are tendered publicly and awarded to the lowest bidder. In most cases the Department supplies the asphalt binder and the raw aggregate source free to the Contractor. Projects can range in size from 20 000 t to 200 000 t of asphalt concrete mix. Virtually all the production of hot mix asphalt concrete is carried out by portable drum-dryer asphalt plants. An average of about 3.5 million tonnes of asphalt concrete mixes have been placed per year over the last 10 years.

In the past, all mix designs were carried out by the Department. More recently, through the increased use of end result type specifications, the Contractor has complete responsibility for supply of all materials,

mix design and construction quality control.

2.4 Materials

2.4.1 Asphalt

All asphalt cement used by AT&U is obtained from the distillation of crude oil. All refineries supplying the Department refine crude oil straight to grade.

Currently asphalts are supplied to AT&U by six refineries of which four are located in Alberta, one in British Columbia and one in Montana, USA.

The crude oils utilized by each of these refineries come from a variety of different sources with different characteristics. However, all crude oils used in the Alberta refineries are considered heavy crudes and produce high viscosity, low temperature susceptible asphalts. The crude sources used by the British Columbia refinery and the Montana refinery in the United States produce a medium viscosity, more temperature susceptible asphalt. These asphalts meet AT&U 'B' grade specifications and are therefore used on only a small percentage of paving projects and then only in the lower lifts.

2.4.2 Aggregates

Aggregate sources in Alberta are generally glaciofluvial or fluvial deposits of well graded aggregates. The aggregates are predominately quartz, quartzite or granite rock. The production of paving aggregates from these sources involves crushing to a topsize of 12.5 or 16 mm. The paving aggregates are split into coarse and fine fractions to allow flexibility in designing the final gradation. Often natural blend sands are used to open up well graded aggregate to meet minimum VMA requirements.

2.5 Evolution of AT&U Asphalt Cement Specifications

Asphalt cement specifications used by Alberta Transportation have been developed over the last two decades in an attempt to optimize the low

and high temperature performance of asphalt concrete pavements.

Prior to 1967, asphalt cements were graded only by penetration at 25°C. Two grades were in use: 150-200 pen and 200-300 pen. A minimum quantity of SC-3000 was also used.

Significant changes were made to asphalt cement specifications in 1967 to deal primarily with improving low temperature performance of asphalt concrete pavements. As a result of the preliminary findings of the Alberta Test Road (1) constructed in 1966 and other field surveys (2), a minimum viscosity at 60°C of 275 poise and a minimum penetration at 25°C of 250 dmm were adopted to control both the consistency and temperature susceptibility of the asphalt. This asphalt cement grade was referred to as AC 275 and was the primary asphalt cement grade used through to the mid-seventies.

In 1978, two new asphalt cement designations were introduced to replace AC 275. The two new grades were designated as AC 27.5 and AC 60 and were essentially "high viscosity" 200-300 pen and 150-200 pen asphalt cements respectively. The addition of a 'harder' 150-200 pen asphalt cement provided a more viscous binder that could be used on higher trafficked roadways, especially in warmer climates, where the softer AC 275 was leading to rutting problems.

In 1980, asphalt cement specifications were further revised to include a total of 5 grades - a high viscosity 150-200 pen, a high and medium viscosity 200-300 pen and a high and medium viscosity 300-400 pen asphalt cement (Figure 1). The minimum and maximum viscosity limits paralleled PVN (Penetration-Viscosity Number) lines as defined by McLeod (3). This specification allowed the use of more temperature susceptible asphalt cements ("B" grades) in lower lifts of the pavement structure without compromising low temperature performance. These specifications are still in use.

2.6 Asphalt Mix Design

The Marshall Method of Mix Design has historically been used by AT&U to design asphalt concrete mixtures.

In the sixties and seventies, the objective of mix design was simply to identify, for the gradation of aggregate produced for the project, an optimum asphalt content that would provide adequate stability and durability. During that period very little emphasis was placed on optimizing aggregate or mix characteristics and it was assumed that if Marshall design characteristics met or exceeded Asphalt Institute criteria, an acceptably performing asphalt pavement would result.

In the late seventies and throughout the eighties, the Department has placed an ever greater emphasis on the engineering of aggregate and asphalt mixes. The inadequacies of the Marshall method of mix design are better known, and in the absence of more performance based or rational design procedures, specifications for mix properties and aggregate gradation and other characteristics have evolved based upon historical performance and engineering judgement.

3.0 LOW TEMPERATURE PERFORMANCE OF ACP IN ALBERTA

3.1 Introduction

Low temperature transverse cracking can be defined as regular, "across the pavement" cracks, orientated perpendicular to the longitudinal direction of the asphalt concrete pavement. These cracks result from low temperature stresses, induced in the pavement by the cooling of a longitudinally restrained pavement, exceeding the tensile strength of the pavement.

A significant portion of the paved highway network in Alberta exhibits some degree of transverse cracking. Transverse crack frequencies can be as high as 90 cracks/km or greater.

An extensive, system wide investigation into the performance of asphalt concrete pavements constructed by AT&U was initiated in 1984 with the objectives to identify the major factors that were influencing the low temperature performance of asphalt concrete pavements and to provide more definitive design guidelines to the highway engineer to minimize or negate low temperature transverse cracking.

The results of this investigation are reported in more detail in (4)(5).

3.2 AT&U Investigation

The approach of this investigation involved a large scale transverse cracking survey of 77 highway sections (55 full-depth asphalt pavement and 22 granular base course pavement structures) totalling 991 km of asphalt concrete pavement constructed on the rural highway network in Alberta between 1970 and 1979. Only non-overlayed pavements were included in the investigation.

Information and data pertaining to the pavement structure, its age, as supplied asphalt characteristics, climate, cracking frequency and as-built pavement characteristics for selected sections were gathered and

entered into a data bank for further analysis using multiple regression computer programs. The direct influence of subgrade materials was not included in this part of the investigation.

Asphalt binder stiffness, which is a function of asphalt rheological characteristics, temperature and loading conditions, was determined using McLeod's method (3). The penetration at 25°C and the absolute viscosity at 60°C of the asphalt cement originally supplied to each project was used to calculate PVN.

Temperature is a major input into the determination of asphalt stiffness. The minimum expected ambient temperature for each highway section evaluated was determined from the National Building Code (6) supplemented with local environmental records. To determine the asphalt stiffness for each section, the pavement temperature at a 50 mm depth was determined using a chart based upon the Ste. Anne Test Road (7) that correlates air temperature with pavement temperature at various depths. Following McLeod's method (3), a loading time of 20 000 s (5-6 h), corresponding to a stress being applied as a result of slow chilling, was used.

Altogether, a total of 16 independent variables related to the pavement structure and age, climatic conditions, asphalt cement and asphalt stiffness were considered as potential independent variables for the model.

The final, most statistically significant model developed to explain the observed cracking frequency of the 55 full depth pavement sections was:

$$\text{Frequency (cracks/km)} = 49.40 + 3.09\,(\text{pavement age in years}) + 0.36\,(\text{original asphalt stiffness in kg/cm}^2) - 5.60\,(\text{pavement thickness in mm})^{0.5}$$

The correlation coefficient, r, of 0.80 is significant at the one percent level. The multiple correlation coefficient, r^2, indicates that 64 percent of the total variance is explained by the model. The F test

indicates the model is adequate at the 0.1 percent level.

This model directly identifies the effects of asphalt stiffness and indirectly identifies the effects of asphalt temperature susceptibility, asphalt consistency and service temperature on low temperature pavement performance.

Another model was developed for granular base pavements in which original asphalt stiffness correlated the greatest with transverse cracking frequency. The granular base thickness was not significant.

3.3 Design Approach to Minimize Transverse Cracking

To establish design guidelines to minimize low temperature transverse cracking it was necessary to identify a critical stiffness that separates acceptable from non-acceptable cracking performance. This critical asphalt stiffness, based upon a review of the data available for the 77 sections previously presented and on the case studies presented in Reference (5), was determined to be 30 kg/cm² or 2.9×10^6 Pa. (using original asphalt characteristics and the temperature at a pavement depth of 50 mm.

Knowing the typical characteristics of asphalt cements used by AT&U and using this critical asphalt stiffness value, critical pavement temperatures can be back-calculated using van der Poel's stiffness nomograph as modified by McLeod.

These calculations can be extended to determine critical ambient air temperatures that correspond to an asphalt critical stiffness at either a 50 mm or 100 mm pavement depth. For typical asphalt grades used by AT&U, these critical ambient air temperatures are:

Asphalt Grade	Ambient Air Temperature in °C Resulting in a Critical Asphalt Stiffness of 2.9 x 10^6 at a Pavement Depth of:	
	50 mm	100 mm
150-200A	-37°C	-43°C
200-300B	-38°C	-44°C
200-300A	-43°C	-49°C
300-400A	-45°C	-51°C

These values indicate, that for an expected low ambient temperature of 43°C, that 200-300A or 300-400A would have to be used for the surface pavement course if transverse cracking is to be avoided. This chart also shows that any of the grades could be used for the construction of lower lifts without the asphalt binder critical stiffness being exceeded.

The same map that was used to identify minimum expected ambient temperatures for each pavement section was used to identify climatic regions that would correspond to critical minimum ambient temperatures identified for each asphalt grade. A map was developed to assist in the selection of asphalt grade to optimize low temperature performance based upon climatic region. This map is presented in Figure 2.

3.4 Conclusions

This investigation concluded that the major factors influencing low temperature transverse cracking behavior were pavement thickness, original asphalt stiffness and pavement age.

The results of this research provided more definitive guidelines regarding asphalt grade selection to optimize low temperature pavement performance. However there was still a need to quantify, in a similar way, the influence of asphalt characteristics on high temperature pavement performance and to develop a design method for the selection of asphalt grades to address pavement performance at both in-service temperature extremes.

4.0 HIGH TEMPERATURE PERFORMANCE OF ACP IN ALBERTA

4.1 Introduction

An asphalt pavement distress mode which is gaining increasing attention from highway engineers world wide is permanent deformation in the wheelpaths. This wheelpath rutting is becoming increasingly noticeable in western Canada as traffic volumes and loadings both increase.

An extensive, system wide investigation into the high temperature performance of asphalt concrete pavements constructed by AT&U was initiated in 1986 with the objectives to determine the influence of the binder characteristics and the effects of temperature, specifically with respect to binder stiffness, on the permanent deformation of asphalt concrete pavements (7). The results of this work would provide more definitive input into the development of design guidelines for the selection of asphalt grades to optimize high temperature performance.

This study was conducted in two phases. The first phase examined the performance of pavements throughout the Alberta highway network while the second phase examined the behaviour of mixes in the Laboratory under repeated load testing. The results of the Laboratory phase are reported in (7) and (8).

4.2 Field Phase

4.2.1 Project Selection and Testing

The field portion of this research involved inspections of forty highway sites within the Provincial highways network. Sites were selected that represented high and low traffic volumes, thin and thick pavement structures, hard and soft asphalt binders, and all geographic locations representing the range of climates in the Province. Pavement structure types included asphalt concrete pavements on granular base courses, asphalt concrete pavements on cement stabilized sand base courses and full-depth asphalt concrete placed directly on prepared subgrade.

The sites inspected covered a significant range of the various parameters with cumulative ESALs (Equivalent Single Axle Loads) ranging from about 10 000 to over 3.5 million, asphalt concrete thicknesses between 50 and 300 mm, and asphalt binders with original penetration at 25°C from 160 to 317 (0.1 mm units) although the majority of the projects were constructed with a binder equivalent to a 200-300 penetration grade asphalt cement. Historical construction and mix design data was reviewed to define the characteristics of the as-built pavement structures.

At each site, rut depths were measured using a 1.8 m straight edge.

Of the forty sites inspected, twenty eight sites were cored to obtain information on the current in-place material characteristics. For each core, the density was measured and the asphalt extracted and recovered using the Abson recovery method (ASTM D 1856/79). Rheology testing was conducted on the recovered asphalts and aggregate gradations were measured for each of the tested samples.

The coring of the twenty-eight project sites also allowed for confirmation of the historical structural data.

4.2.2 Analysis and Model Development

The analysis of the data involved correlating the various pavement, materials, traffic and climatic characteristics with the measured pavement rutting. The observed rutting was considered as the dependant variable, with all other parameters considered as independent variables.

The independent variables considered included original Marshall mix design data, field quality control data gathered at the time of the original construction, and materials data collected from the cores taken for this study. As well, measures of asphalt temperature susceptibility, stiffness, and changes in binder rheology over time were calculated and included in the analysis.

Asphalt binder stiffness was determined using McLeod's method (3). PVN was calculated using penetration at 25°C and absolute viscosity at 60°C.

A loading time of 0.5 seconds was used to be consistent with the Laboratory phase of this study. The temperature used was a summer design temperature for the area in which the project was located.

Two models were developed for the field phase of this study. One model, based upon 40 sites, used the rheological characteristics of the original asphalt supplied. The second model was based on the rheological characteristics of the recovered in-place data from the 28 cored sites.

The model developed for the original asphalt supplied data was:

Rut depth (mm) = 2.6186 + 0.0060*(Daily Cumulative ESALs)
- 0.0023*(Daily Cumulative ESALs)*Log(Original Binder Stiffness(KPa))

The correlation coefficient, r, of 0.67 for this model is significant at the 1% level of significance. The r^2 value of 0.45 indicated that 45% of the total variation is explained by the model. The F statistic, calculated using repeat test points to examine lack of fit (LOF) of the model, was 2.97. At the 5% level of significance, this F statistic indicates there is some LOF for this model form; however, from a practical viewpoint the model was considered meaningful.

The second model developed, utilizing the rheological characteristics of the recovered in-place asphalt from the 28 cored sites, was:

Rut depth (mm) = 2.9630 + 0.0076*(Daily Cumulative ESALs)
- 0.0024*(Daily Cumulative ESALs)*Log(Abson Stiffness(KPa))

The correlation coefficient, r, of 0.79 for this model is significant at the 1% level of significance. The r^2 value shows that 62% of the variation is accounted for by the model. The calculated F statistic of 1.58 does not indicate any LOF for this model form (probability ≤ 0.05).

Both models were very similar and showed that asphalt stiffness in conjunction with traffic loadings to be the most significant variables correlated with the measured pavement rutting.

4.3 Conclusions

The models developed for the field phase of this investigation showed that the permanent deformation of asphalt concrete was most significantly correlated to the number of load applications and to the binder stiffness.

Results of the field phase suggested that Alberta highways subjected to loading of less than about 0.5 million ESALs can use a 200-300A asphalt without risking a significant degree of rutting. (In the Edmonton climatic area.)

The field phase also suggested that the pavement structure type did not significantly effect rutting. The field phase did identify that structural thickness was a significant factor; however, because this variable is a function of the ESALs, it was not included in the model.

Neither the field or laboratory phases suggested a correlation of mix design characteristics with permanent deformation. This was most likely influenced by the limited range of parameters and the proximity of Marshall characteristics to the optimal design values.

Based on these results it is apparent that asphalt stiffness and therefore asphalt grade has an effect on the level of rutting that can be expected in an asphalt concrete pavement. Additionally, because the asphalt stiffness is greatly influenced by temperature, consideration must be given to the climatic conditions the pavement will experience in-service. The recognition of these interactions has contributed to the development of specific guidelines presented in the next Chapter.

5.0 DEVELOPMENT OF DESIGN GUIDELINES

5.1 Introduction

The results of the two major research programs carried out by the Department clearly identified the effects of asphalt binder stiffness on both high and low temperature performance of asphalt concrete pavements in Alberta.

This research identified that the transverse cracking of asphalt concrete pavements across the province is due to the use of highly temperature susceptible asphalts in the past and more recently to the use of less temperature susceptible but "harder" asphalts, ie. stiffer asphalts, in an attempt to minimize rutting potential.

This research also identified that the most serious rutting of the network is confined to a few highly trafficked highways eg. Hwy 1, Calgary to Medicine Hat; Hwy 16, Edmonton vicinity; Hwy 2, south of Calgary to north of Edmonton. A significant proportion of this rutting on the provincial road network has been influenced by the extensive use of AC 275 (high viscosity, 200-300 pen) prior to 1978 and the use of mixtures of inadequate rutting resistance for the traffic loadings and climatic conditions.

However, it is anticipated that future increased traffic loadings will significantly increase the extent and severity of rutting on the network in spite of the use of stiffer asphalt cements. In fact, based on current traffic data it is estimated that traffic loadings, in terms of cumulative ESALs, over the next twenty year period will be more than double those of the previous twenty year period.

Until acceptable performance based mix design methods and criteria are developed, a need was identified to expand current guidelines and rules-of-thumb and to provide more definitive and rational design criteria to be used in the design of asphalt concrete mixes.

The design criteria and guidelines developed addressed:

* selection of asphalt grade
* selection of aggregate characteristics
  - topsize
  - manufactured and natural fines (minus 5000 μm sizes)
  - % fractures
* Marshall mix design factors
  - number of blows
  - minimum Marshall Stability, Marshall flow
  - VMA and air voids

These criteria and guidelines were based upon traffic loadings as defined by the projected cumulative ESALs over the design life of the pavement, and the maximum anticipated summer air temperatures the pavement would be subjected to. The minimum winter air temperatures the pavement would be exposed to were indirectly built into the guidelines.

5.2 Asphalt Grade Selection

A design map was developed to allow a designer to select an appropriate asphalt grade for the anticipated design ESALs. The development was based upon the rut prediction models described earlier which take into account more directly the effects of asphalt stiffness, traffic loadings and summer operating temperatures.

In order to separate the effects of temperature on binder stiffness, a number of climatic zones based upon maximum summer temperatures were identified for the Province. These zones are presented in Figure 4.

Using the ten year average summer maximum temperature for each zone, binder stiffnesses were calculated following McLeod's method and using typical rheological properties for 150-200A and 200-300A and a loading time of 0.5 s.

Using the rut prediction models, curves for rutting levels of 5 and 10

mm were established over the range of binder stiffnesses calculated and ESALs anticipated.

For each zone, the ESALs levels required to reach 5 and 10 mm ruts for a binder stiffness equivalent to 150-200A were determined. These levels identified the range of ESALs where 150-200A could be used. Below this range, 200-300A would be specified. This is illustrated in Figure 5. ESALs above this range would identify a need for enhancements to the quality of the aggregates being used in order to provide improved rutting resistance. This need for aggregate enhancements is justified as the development of the rut models was based upon the typical aggregate and mix characteristics used in the past.

The criteria developed for asphalt grade selection is based upon using the 'softest' grade of asphalt cement that will provide acceptable low temperature transverse cracking behavior and still provide acceptable rutting resistance.

This map demonstrates that a pavement constructed near Edmonton, in central Alberta, using 150-200A could accept about double the total ESALs over its design life as compared to a pavement constructed near Lethbridge, in southern Alberta, with the same asphalt grade and still provide similar rutting performance.

This work led to the development of the "ESAL Criteria for Selection of Asphalt Mix Type" shown in Table 1.

5.3 Asphalt Concrete Mixes

It was necessary to categorize and identify mix types to aid the designer in specifying an appropriate mix type for a specific project. Eight asphalt mix type designations were developed and defined. Seven of the mix types were currently being used although without any formal designation.

Within each mix type, in addition to asphalt grade selection, criteria

were developed for each parameter identified. This is presented in Table 2. The basis for the criteria developed follows.

5.3.1 Aggregate Characteristics

Department aggregate specifications include requirements for gradation, % fractures, LA abrasion and plasticity. Historically, both 12.5 mm and 16 mm topsize aggregates have been used in Alberta. Selection in the past was generally governed by achievement of acceptable % fractures and more recently has considered both the increased rutting resistance that may be afforded by using larger topsize aggregates as well as the increased segregation potential of larger topsize aggregates.

An extensive analysis of past mix designs performed by the Department was carried out and the design characteristics (eg. Marshall stability, Marshall air voids and VMA, aggregate gradation, % of manufactured fines in the -5000 portion of the aggregate) for both 12.5 mm and 16 mm mixes were analyzed and compared.

This analysis helped define limits for the percentage of manufactured fines in the minus 5000 µm portion of the combined aggregate, defined as '% MF, -5000', for each mix type.

Type 1 mix, for which ESAL criteria has not been developed yet, would require 16 mm topsize aggregate, 100% MF, -5000 and 100% fractures. This mix would be used on very high trafficked highways.

Type 2 mix would be used on high trafficked highways and would have a requirement for a minimum of 70% MF, -5000 and 70% fractures.

Types 3 and 4 mixes are considered equivalent and their selection would be based on past experience or project specific design or materials constraints. The % MF, -5000 criteria is based upon the designated aggregate topsize. These criteria of 40 and 50% are somewhat restrictive compared to what has been experienced in the past. It is estimated that about half of the aggregate deposits used in the past would not be able to meet this requirement without the addition of manufactured fines.

Types 5, 6 and 7 mixes would be used on lower trafficked highways, secondary roads and community airports and do not have a separate requirement for manufactured fines. Conventionally crushed aggregates meeting the specified gradation band and % fractures requirements would be acceptable.

The use of 12.5 mm mixes on lower trafficked roads where risk of pavement rutting is very low affords reduced segregation potential of the asphalt mix during construction which can reduce maintenance costs over the life of the pavement.

5.3.2 Marshall Design Parameters

Marshall design characteristics are presented in Table 3 and generally conform to Asphalt Institute criteria (10). The minimum Marshall stability has been increased from AI criteria for Types 1 and 2 mixes. Although it is recognized that high Marshall stability doesn't guarantee high rutting resistance, low Marshall stabilities tend to be associated with aggregate characteristics that are not desireable in a highly rut resistant mix.

The number of blows to be used in the mix design is also based upon summer operating temperatures and ESAL levels and recognizes that the risk of rutting or bleeding of mixes designed with 50 blows increases with increased summer temperature.

5.4 Summary

The design map and ESAL criteria developed will allow a more rational approach for specifying asphalt grade, aggregate characteristics and Marshall mix design parameters.

For very low volume airports, mixes designed with 50 blows, 300-400A asphalt cement and 12.5 mm topsize aggregate will provide acceptable performance.

As ESAL levels increase, stiffer asphalts, larger topsize aggregates,

increased proportions of manufactured fines and 75 blows can be specified based upon maximum summer temperatures to ensure acceptable high temperature pavement performance.

6.0 SUMMARY AND CONCLUSIONS

Research undertaken by Alberta Transportation and Utilities has identified that the asphalt binder plays a significant role in the performance of asphalt concrete pavements at both high and low service temperatures. The results of this research have been applied to the development of guidelines for the selection of various mix types. These criteria address:

* selection of asphalt grade
* selection of aggregate characteristics
* Marshall mix design parameters

The selection criteria accounts for climatic variations across the province and the effect of climate on binder stiffness. The asphalt grade selection is biased towards providing acceptable high temperature performance; a softer asphalt grade is specified to address low temperature performance concerns when loading conditions permit.

Aggregate characteristics are specified in terms of topsize, proportion of manufactured fines and % fractures to enhance high temperature performance when required by loading conditions.

The number of blows required for the Marshall mix design are specified to reflect the concern for high temperature performance for the various mix types.

The development of the Design Guidelines for Asphalt Mix Types has resulted in a more rational approach for the selection of mix parameters to optimize pavement performance throughout the province in the absence of performance based mix design methods and criteria.

The approach taken to develop these guidelines is based upon sound engineering judgement, real life observations of the performance of many kilometres of asphalt concrete pavements in Alberta and significant research efforts. Although the criteria developed are applicable only

to Alberta conditions and reflect significantly past design practice, it is felt that the approach presented is applicable to any other jurisdiction which experiences variations in climate and traffic loadings.

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Table 1 ESAL CRITERIA FOR SELECTION OF ASPHALT MIX TYPE

ZONE	ASPHALT MIX TYPE					
	1[1]	2	3 or 4	5	6	7[2]
A	-	> 1.0	0.5-1.0	0.3-0.5	< 0.3	-
B	-	> 1.5	0.7-1.5	0.4-0.7	< 0.4	-
C	-	> 2.0	1.0-2.0	0.5-1.0	< 0.5	-
D	-	> 2.5	1.5-2.5	0.8-1.5	< 0.8	-

1 Criteria To Be Developed
2 Community Airports

ESAL criteria given in Table 1 are the total ESAL's (X 10^6) per direction that will be applied to the asphalt pavement over its design life.

Table 2 ASPHALT CONCRETE MIX TYPES AND CHARACTERISTICS

Asphalt Mix Type	Aggregate Top Size (mm)	% MF,[1] -5000[2]	Fractures +5000 (2 faces) (Min)	Asphalt[2] Cement Type and Grade	Marshall[3] Stability N (Min)	No. of[3] Blows
1	16	100	100	150-200A 120-150A PMA	12000	75
2	16	70	70	150-200A	12000	75
3	16	40	60	150-200A	6700	75
4	12.5	50	60	150-200A	6700	75
5	12.5	-	60	200-300A	6700	75
6	12.5	-	60	200-300A	4500	50
7[4]	12.5	-	60	300-400A	4500	50
8[5]	10	-	60	Note 1		Note 2

1 % Manufactured Fines In The -5000 Portion Of The Combined Aggregate
2 For Surface Courses And For Virgin Mixes Only
3 For Marshall Mix Design
4 Community Airports
5 Levelling course

June'90

Note 1 - Use the same asphalt grade as for the surface course.
Note 2 - Use the same number of blows as for the surface course.

Table 3 MARSHALL CHARACTERISTICS FOR ASPHALT MIXES

CHARACTERISTICS	CLASS 10	CLASS 12.5	CLASS 16
Marshall Stability, N	Refer to "Asphalt Concrete Mix Types and Characteristics" Table for minimum requirements.		
Air Voids %	3 to 5	3 to 5	3 to 5
V.M.A %	15.5 to 17.0	15.0 to 16.5	14.5 to 16.0
Flow mm	2 to 5	2 to 5	2 to 5
Retained Stability %	70	70	70

(i) Retained stability after 24 hour soaking at 60°C to be run at the recommended design asphalt content.

(ii) The minimum V.M.A. criteria may be reduced by up to 1% if experience indicates that the mixture will perform satisfactorily and all other criteria are met.

(iii) All asphalt mixtures designed for airport paving will have an air void requirement of 2 to 4 regardless of aggregate class.

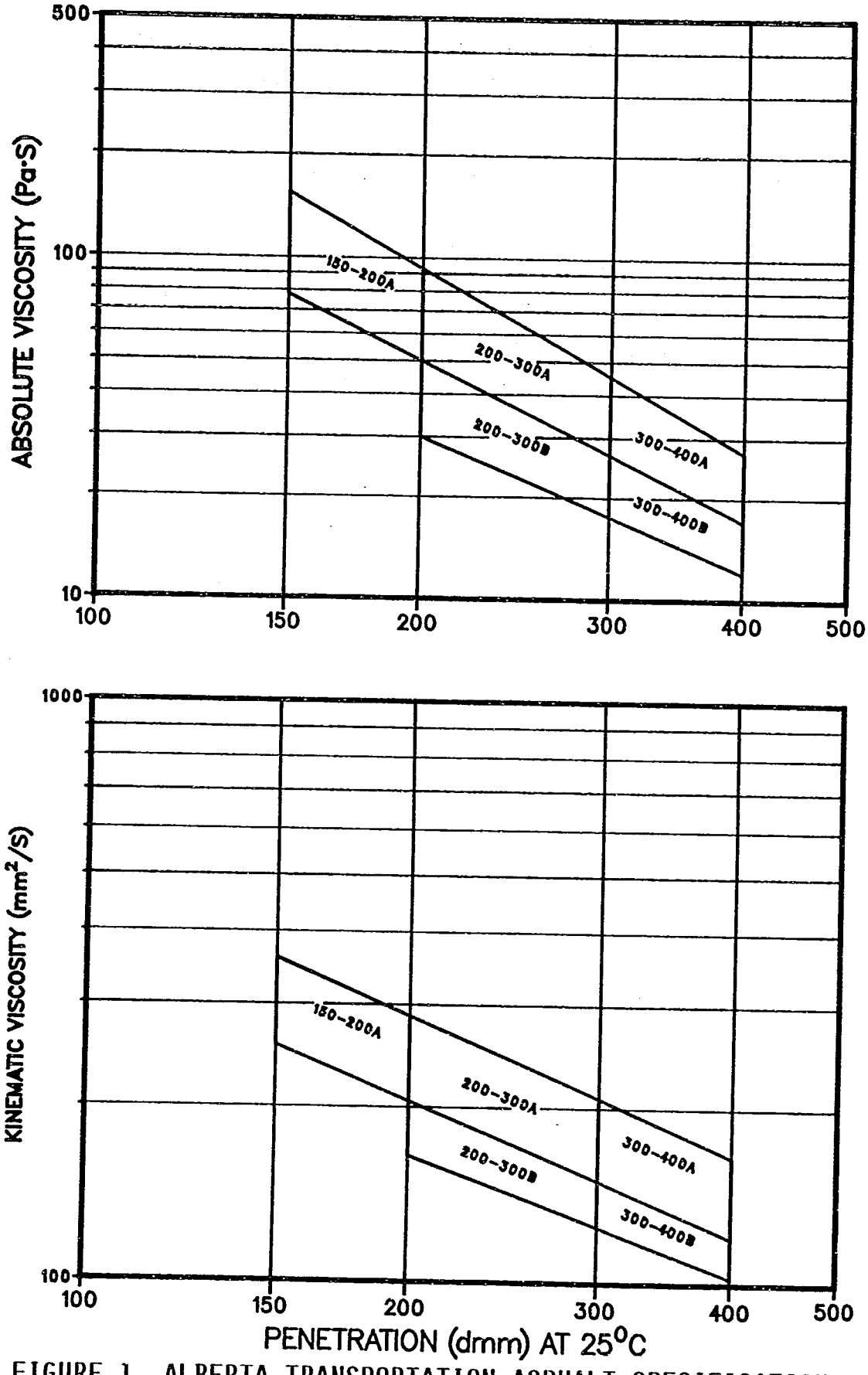
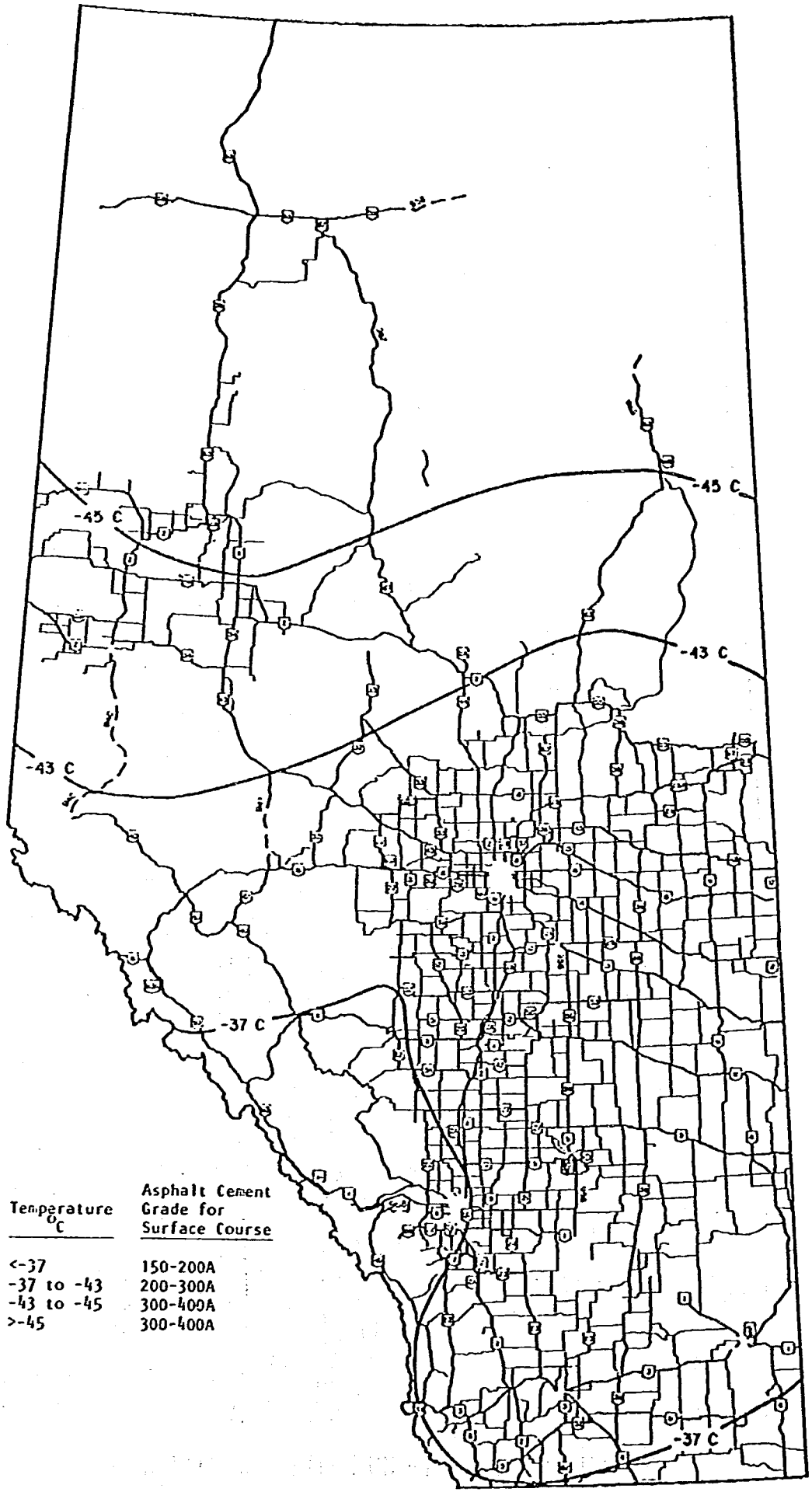


FIGURE 1 ALBERTA TRANSPORTATION ASPHALT SPECIFICATIONS



Temperature °C	Asphalt Cement Grade for Surface Course
<-37	150-200A
-37 to -43	200-300A
-43 to -45	300-400A
>-45	300-400A

FIGURE 2 LOW TEMPERATURE DESIGN MAP

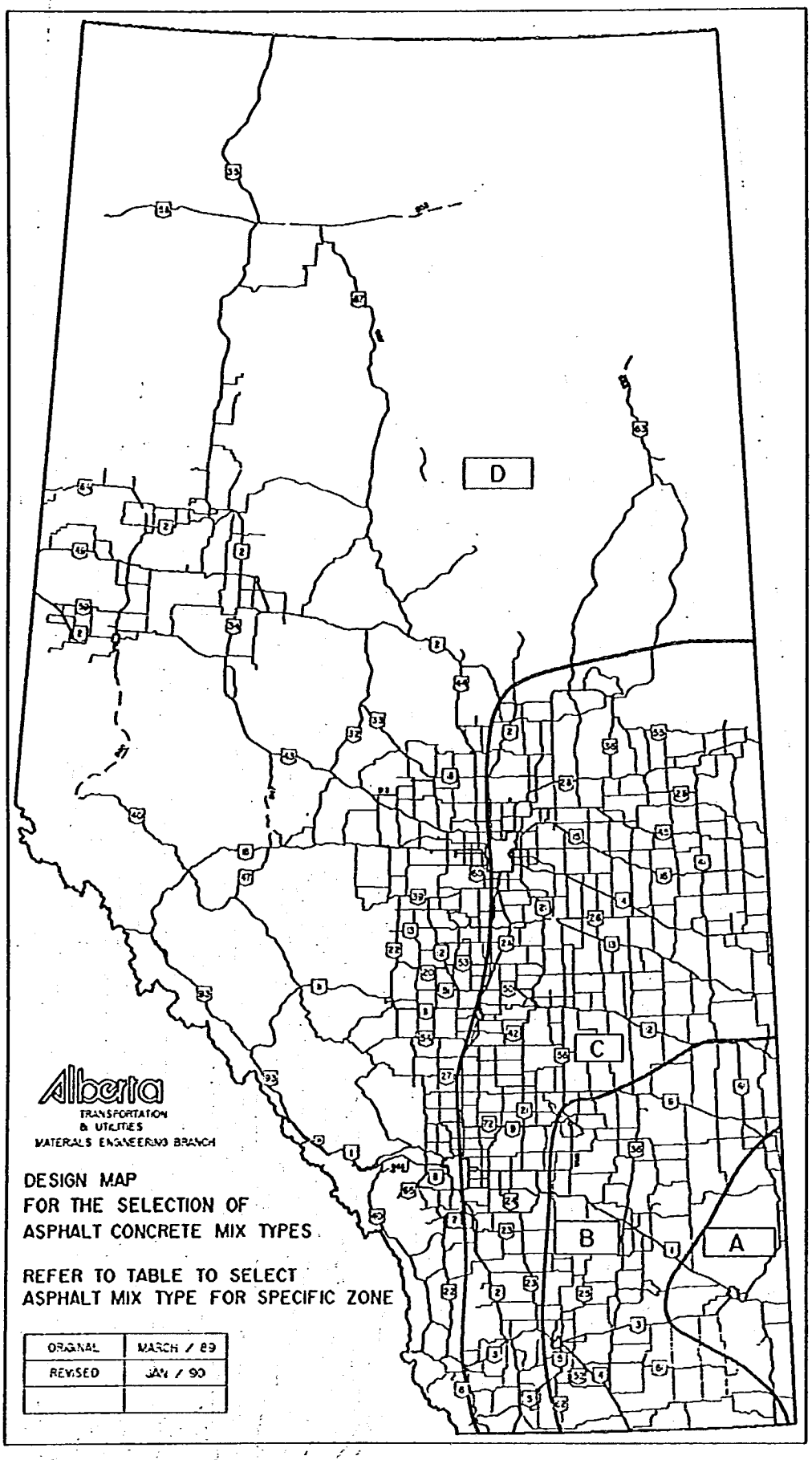


FIGURE 3 HIGH TEMPERATURE DESIGN MAP

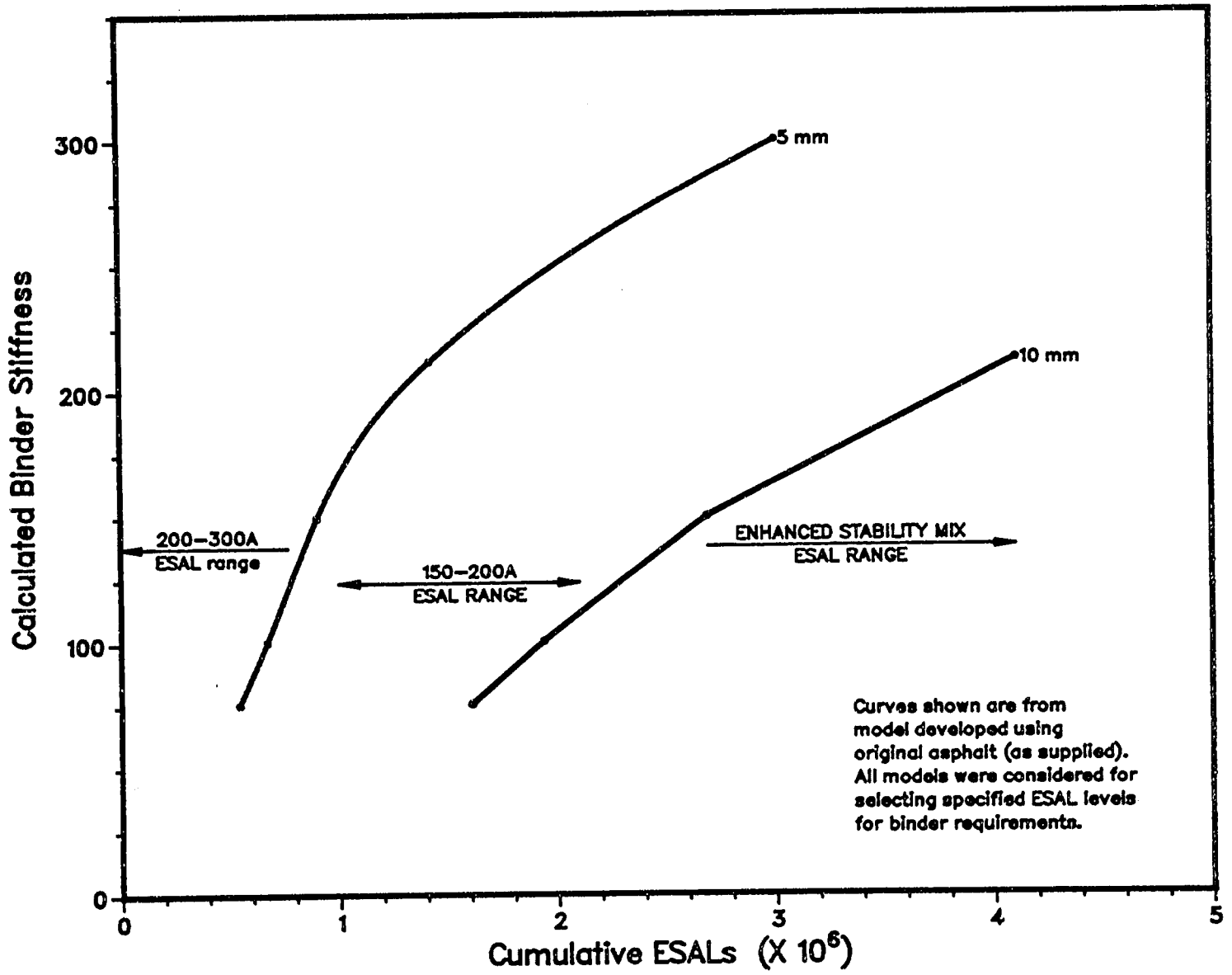


FIGURE 4 CURVES OF CONSTANT RUT DEPTHS

Damage to Expressway Pavement in Hokkaido and Solutions Required

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Japan Highway Public Corporation

DAMAGE TO EXPRESSWAY PAVEMENT IN HOKKAIDO AND SOLUTIONS REQUIRED

by

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ABSTRACT

Pavements in the Hokkaido region are exposed to extremely rigorous climatic environment as compared to the main island of Honshu because of a number of adverse factors. That is, road temperature falls below 20°C in winter, "winter days" when the minimum temperature falls below 0°C exceed 150 days per year, and "snow days" exceed 100 days a year.

In Hokkaido, expressways have been commissioned to provide service withstanding such rigorous climatic conditions since the first expressway was constructed 18 years ago.

During this period, however, pavements have been subjected to diverse types of damages, such as rutting due to wear, transverse cracking, etc.

Thus, the Sapporo Bureau of Construction of the Japan Highway Public Corporation (JHPC) is committed to the construction and management of highways in Hokkaido and has been engaged in continuous research on pavement issues related to snow-covered, frigid regions.

This paper presents the findings of a series of investigations and studies conducted by the Sapporo Bureau of Construction and the Japan Highway Public Corporation on pavement damage and countermeasures requried.

DAMAGE TO EXPRESSWAY PAVEMENT IN HOKKAIDO AND SOLUTIONS REQUIRED

by

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PREFACE

Public expressways in Hokkaido Insland extend for a total of 233 km, including the Central Hokkaido Expressway which was initially commissioned in December 1971 Chitose Interchange (IC) - Kita-hiroshima IC and the Sasson Expressway Otaru IC - Sapporo-Nishi IC which in all extends 47 km.

Since the Hokkaido region is exposed to more rigorous climatic environment than the main island of Honshu in terms of snowfall and temperature, a variety of unique contrivances for instance are required in the design, construction and maintenance of expressways.

Studies have long been made by many organizations on the issues of road freezing and road wear in Hokkaido, because of the natural phenomena unique to this region.

Relative to this situation, transverse cracking in pavements have become pronounced over part of these expressways, and accordingly, there is apprehension concerning their negative impact on pavement integrity. These issues have been exposed to extensive studies and investigations in terms of both road structures and materials used. However, at present, we have failed to reach any tangible solution regarding this matter.

Our opinion is that a timely solution to this issue is imperative, especially in view of the fact that Hokkaido is expected to extend its expressways for another 1,000 km (600 miles) and that pavement maintenance and repair will be required for roads already commissioned to service.

Thus, the Sapporo Bureau of Construction of the Japan Highway Public Corporation (JHPC) is committed to the construction and management of highways in Hokkaido and has been engaged in continuous research on pavement issues related to snow-covered, frigid regions.

This paper presents a report on the results of studies and investigations into existing and potential pavement damage and proposes necessary countermeasures.

1. CURRENT SITUATION OF EXPRESSWAYS IN HOKKAIDO

1.1 EXPRESSWAYS

The expressways in Hokkaido consists of two major routes including the north-to-south Trans Hokkaido Expressway and the east-to-west Trans Hokkaido Expressway and plans are underway to extend the total length to approx. 1,363 km.

Of these, the 209 km long Central Expressway (Noboribetsu Muroran IC - Fukagawa IC) and the 24 km long Sasson Expressway (Otaru IC - Sapporo-Nishi IC) have been commissioned to service for a total distance of approx. 233 km.

Also, an additional 516 km of expressways are under investigation and under construction (Fig. 1).

Current traffic volume stands as of March 1990, for instance, at 20,000 cars daily on the expressway between Eniwa IC and North Hiroshima IC, and 2,000 cars daily between Kitago IC and Sapporo IC.

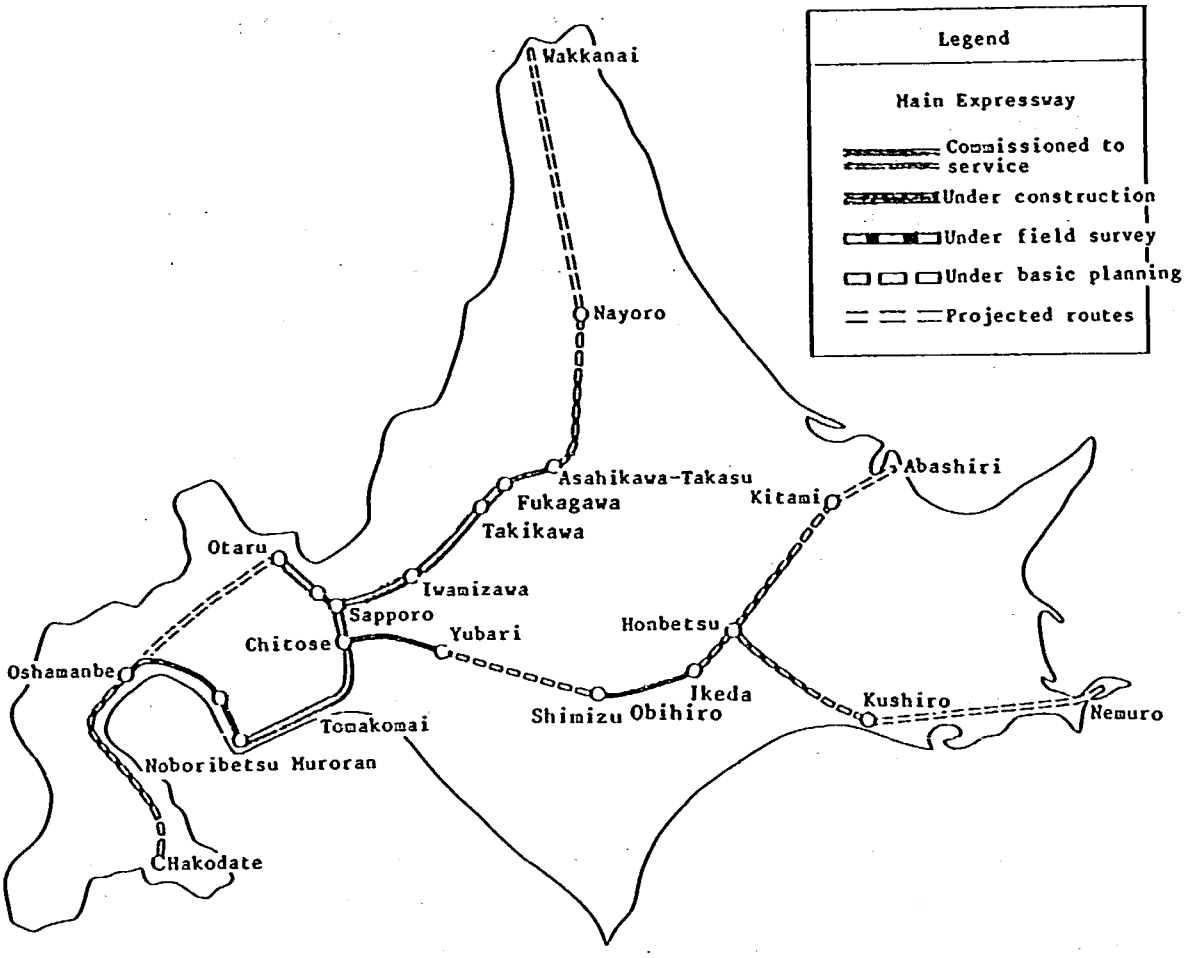


Fig-1 Expressways across Hokkaido

1.2 PREVAILING NATURAL ENVIRONMENT OF EXPRESSWAYS

As shown in Table 1, far more rigorous climate prevails in Hokkaido than in Tokyo. Specifically, Hokkaido is lower than Tokyo in annual average temperature by 7°C and minimum temperatures by 10-20°C. Accordingly, the former is exposed to problems of frost heaving, freezing, and thawing transverse cracking.

Table-1 Climate in Hokkaido[1)]

City	Summer days (>25°C) (days)	Winter days (<0°C) (days)	Annual average temp. (°C)	Min. temp. (°C)	Max. temp. (°C)	Sunny hours (hr)	Rain fall (mm/year)	Freezing index (°C days)
Asahikawa	55	161	6.3	-41.0	35.9	1734	1158	920
Sapporo	46	139	8.0	-23.9	35.8	1962	1158	510
Obihiro	41	166	6.1	-34.9	37.8	2130	952	950
Tokyo	106	28	15.3	- 9.2	38.4	1942	1460	-

1.3 EXPRESSWAY PAVEMENT STRUCTURES

In Hokkaido, most expressways are asphalt-paved, and the pavement structure is designed according to the CBR-TA Process[2)]. Figure 2 presents a typical pavement structure.

In particular, in order to prevent the rupture of pavement due to subgrade frost heaving and freezing, conventional subgrade soil has been replaced with non-frost heavable material for threatened layers.

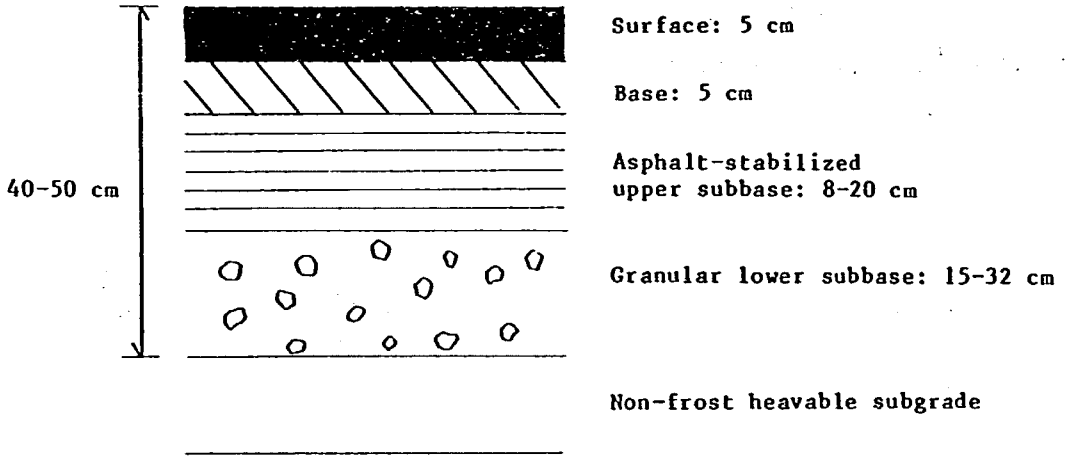


Fig-2 Standard pavement structures

2. EXPRESSWAYS PAVEMENT DAMAGE IN HOKKAIDO

2.1 TRANSVERSE CRACKING

Photo 1 shows a typical transverse cracking prevailing on expressways across Hokkaido. It is characterized by 1) extending nearly perpendicular to the length of a highway and 2) starts on the shoulder and gradually proceeds toward the center. Transverse cracking is increasing in density and in location, as noted in subsequent paragraphs since the first cracking was discovered on the Central Hokkaido Expressway which was laid across seedling plantations in the spring of 1981. Field investigations into transverse cracking have been carried out nearly every year since 1981, the year the initial cracks were found.

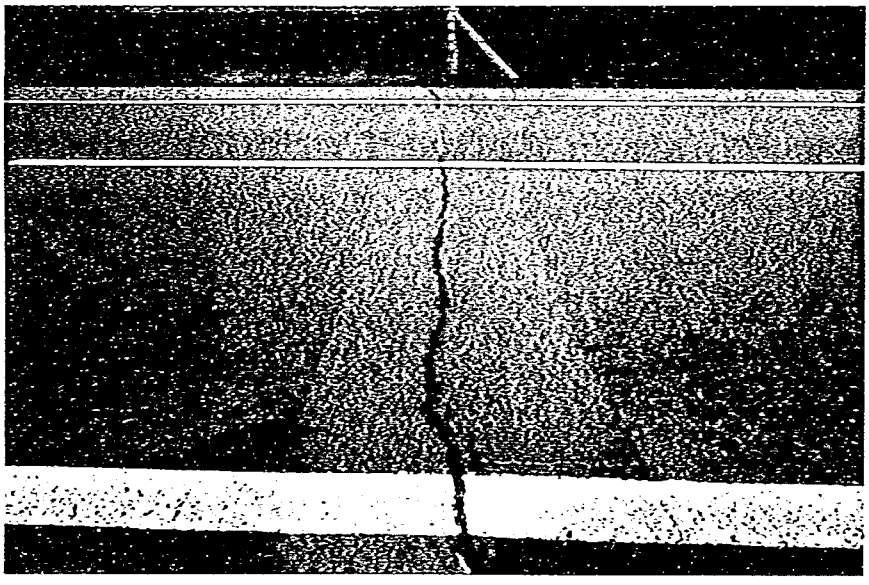


Photo-1 Transverse cracks

2.1.1 CRACKING

Figure 3 summarizes the findings of the investigation, breaking the cracking down by specific pavement unit according to which initial pavement was effected.

With respect to the cracks classified as "ordinary cracks," transverse cracking index (number of cracks per direction/km) was determined on the basis of the number of cracks per unit road extension classified as Rank A or higher (2 m long or more cracks) on the account of reasons given below, as shown in Fig. 4.

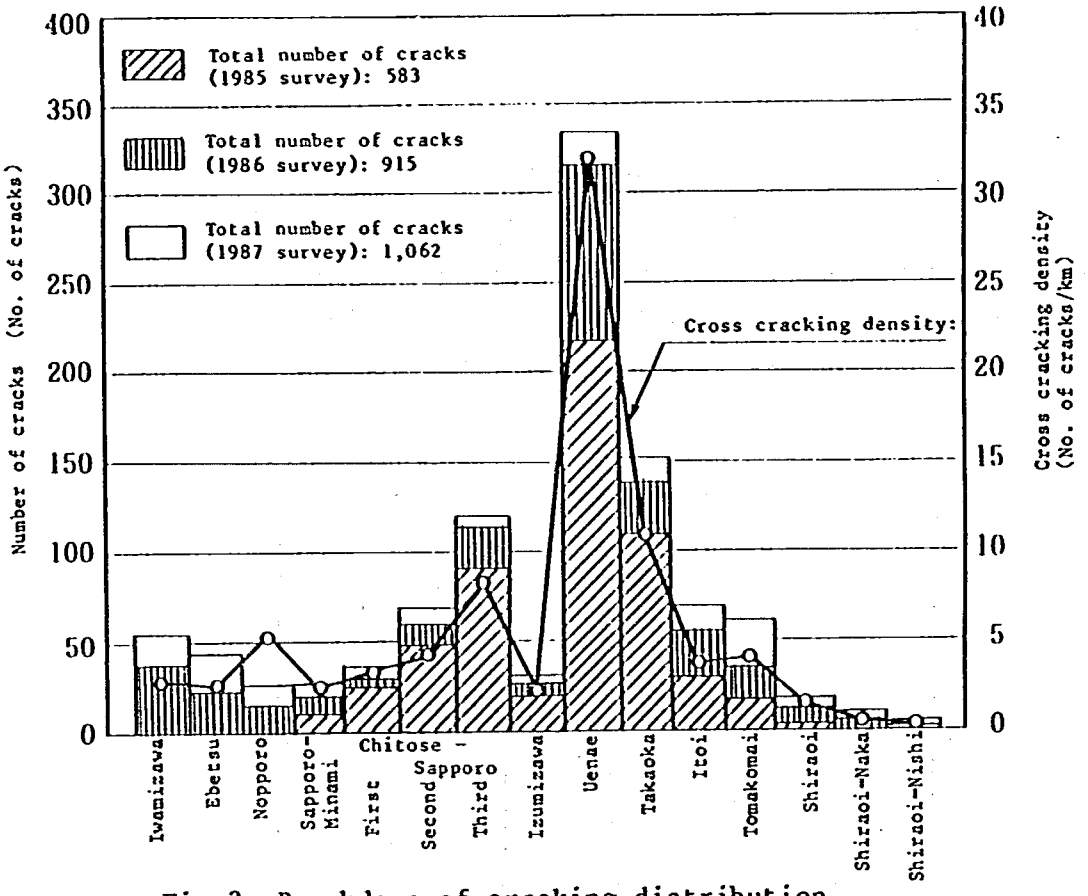
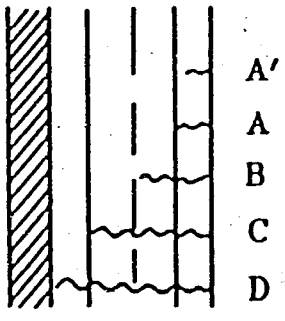


Fig-3 Breakdown of cracking distribution by pavement zone



* The figure represents roadside cracks extending inward to the center; cracks initiating from the center are also classified as cross cracks.

Fig-4 Classification of cracks

. Most of the cracks extend in a direction perpendicular to road length.
. In terms of cracking index, the ratio stands at 0.9%, but even in the locations most severely cracked, there is a possibility that cracking characteristics may be overlooked.

In particular, we investigated cracks extending longer than 2 m from the roadside based on assumption that cracks under 2 m would have less effect on vehicle traffic.

These figures indicate that the largest cracking index occur on the pavement laid across seedling plantations, followed by those laid on heights.

2.1.2 YEAR-TO-YEAR COMPARISON OF CRACKING

Figure 5 gives a year-to-year comparison of the number of cross cracks --- classified as Rank A or higher --- produced on the Central Hokkaido Expressway (Sapporo-Minami IC - Tomakomai-Nishi IC). The figure reveals that the total number of cracks more than quardrupled during 1981 through 1986.

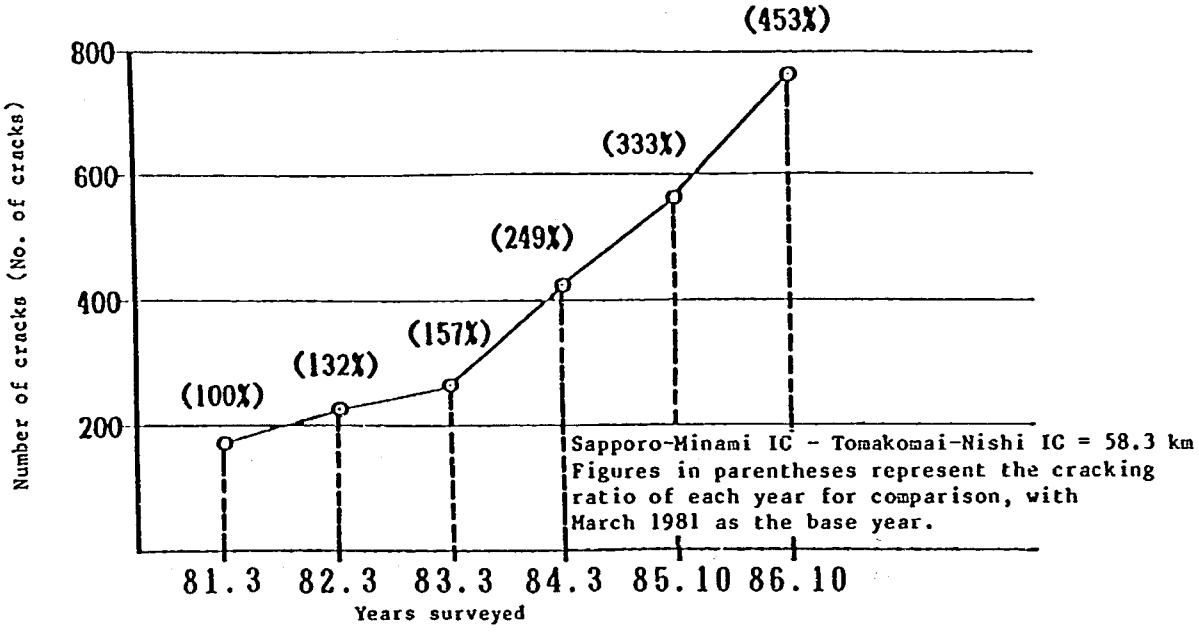


Fig-5 Year-to-year comparison in cracking

Figure 6 shows the year-to-year comparison by number and type of cracks on a 73 km zone (Sapporo-Minami IC - Shiraoi IC) of the Central Hokkaido Expressway.

According to the figure, both minor cracking within a single lane (A) and large cracking spanning multiple lanes (D) are increasing. Recently, the former shows a notable increase.

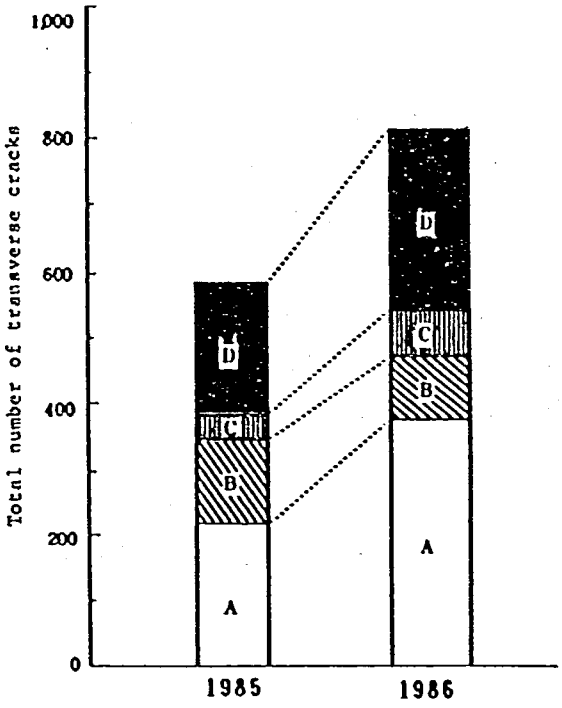


Fig-6 Year-to-year comparison by type of cracking
(Sapporo-Minami- Shiraoi 73 km)

Figure 7 Associates cracks produced with the lateral pavement structures. According to the figure, it is obvious that 75% of these cracks stem from some sort of defective paving work.

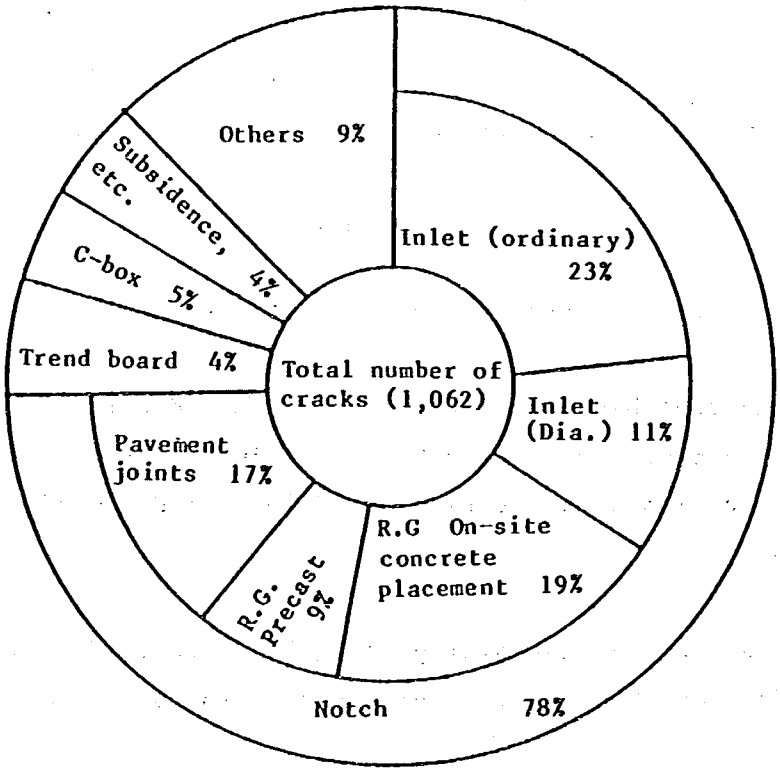


Fig-7 Classification of causes of cracking

2.2 RUTTING

Figure 8 shows the aging of roads after repair, taking typical inter-IC zones on the main national expressways as an example. The cumulative renovation ratio (along the vertical axis) represents all the renovations made to the inter-IC zones in terms of total extension (lane/km), and divides this distance by a total road distance (lane/km) between these interchanges. Accordingly, a 100% cumulative renovation ratio means that renovation was made once, while a 300% cumulative renovation ratio means that renovation was repeated three times during the given period. The figure indicates that the Central Hokkaido Expressway has been renovated at extremely short cycles compared to other highways.

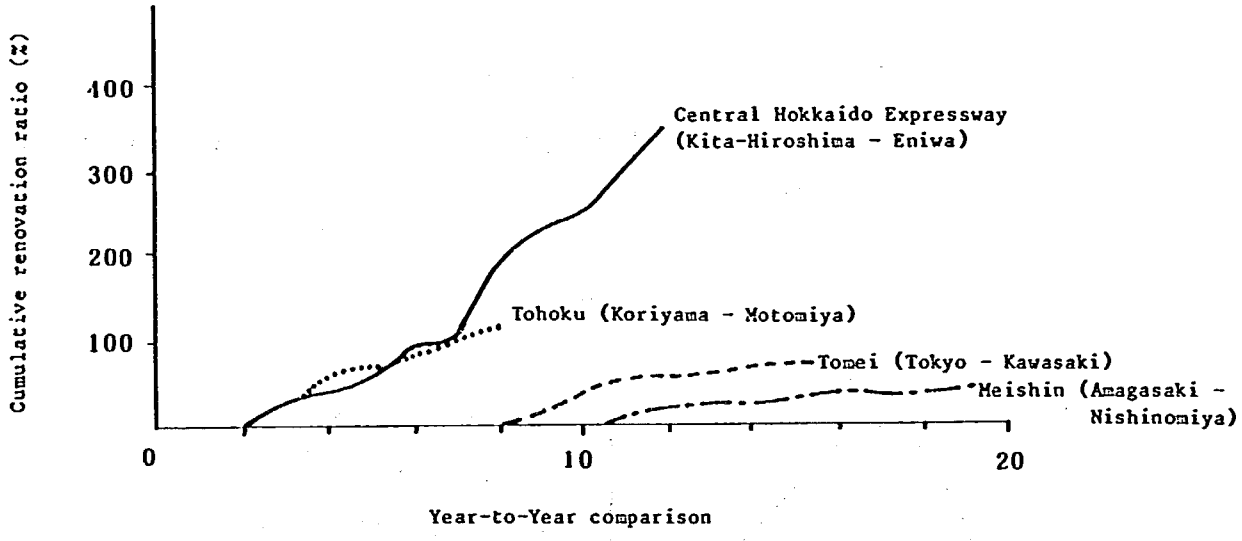


Fig-8 Cumulative renovation ratios for typical expressways

Especially, the zone between Sapporo-Minami IC and Chitose IC of the Central Hokkaido Expressway where traffic volume is the heaviest is subjected to annual maximum rutting of 13-20 mm and demands renovation every 2-3 years.

Also, annual maximum rutting over passing lanes range from 1 to 5 mm in all zones. Consequently, they require renovation once every 5-6 years.

3. PREVENTION OF PAVEMENT DAMAGE

Regarding damage to pavement as referred to in the preceding paragraph, research and investigations into probable causes and remedies have also been carried out by other organizations from an early period.

This paragraph presents a summary of the series of research, investigations and remedies implemented by JHPC.

3.1 TRANSVERSE CRACKING

Regarding the question of transverse cracking, JHPC has been carrying out intensive research in two ways; one is renovation, the other, prevention, as summarized below.

3.1.1 RENOVATION OF TRANSVERSE CRACKS

Since transverse cracking does not require the road surface to be graded, it has less affect on traffic or road serviceability.

Infiltration of rainwater has greater effect on the integrity of pavements.

The means to renovate transverse cracking, such as implemented for the Central Hokkaido Expressway, include total surface renovation by means of ground overlay or local renovation by injection of bituminous binder, resin, etc.

In the case of ovrelay-applied renovation of cracking throughout every layer, a road thus repaired is subjected to reflection crack during winter since overlay pavement is only 3-4 cm thick.

According to the investigation[3] of the Hokkaido Bureau of Investigation, cracking does not occur for a period of 2-3 years where pavement is 8 cm thick, or for 4 to 5 years where pavement is 12-13 cm thick.

In view of this fact, 3-4 cm deep overlay renovation cannot be expected to be very effective.

In contrast, injection-applied renovation commonly uses straight asphalt, but it cannot retain lasting effect due to a number of adverse factors, i.e., contraction of cracks, scattering of asphalt, etc., thereby necessitating renewal of injection every year.

Since 1986, trial construction has been undertaken using a variety of renovative measures. However, no economical and effective technique has yet been established on a commercial scale. Of these, a relatively effective method is given in Fig. 9.

This method places a 200 mm wide sheet net on the crack to support the pavement and seals the cracks with rubber asphalt over a width of 30-40 mm.

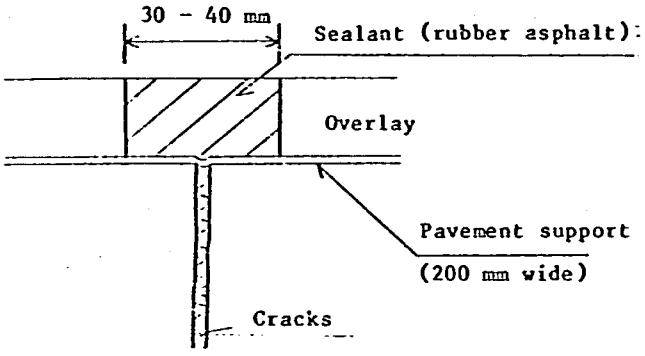


Fig-9 Renovation of transverse cracks

3.1.2 PREVENTION OF TRANSVERSE CRACKING

1) Mechanism and major causes of transverse cracking

Transverse cracking is ascribed to pavement contraction due to temperature drop or cyclic application of temperature stress as a result of significant fluctuations in ambient temperature.

An asphalt mixture has a strong capacity to mitigate stress and can compliantly transform itself. Commonly, it does not undergo rupture due to internal stress at normal temperature. However, an asphalt mixture loses it power to mitigate stress with a decrease in temperature and tends to show specific properties as if it were an elastic body.

It has been held that transverse cracking occurs at the point where stress due to pavement temperature fluctuations has grown to beyond the yield point of pavement.

If fact, cracks are caused by diverse factors or their multiplying effects, as shown in Fig. 10. Therefore, it is impossibly difficult to precisely define a mechanism for initiation of transverse cracks.

This paragraph summarizes factors which may affect the initiation of transverse cracks based on the results of testing.

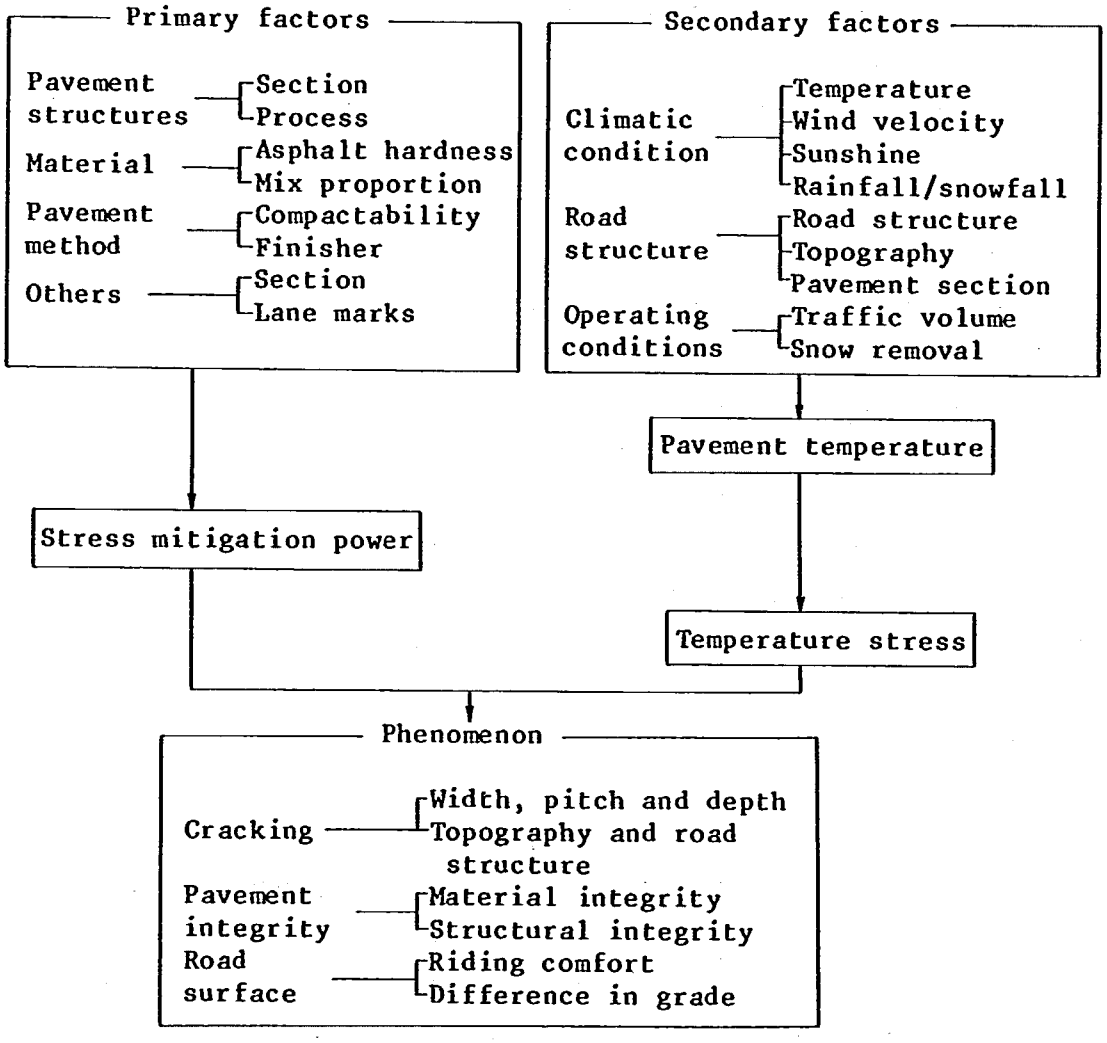


Fig-10 Mechanism of transverse cracking

a. Factors of stress concentration

Transverse cracking on the Central Hokkaido Expressway is for the most part (about three-fourth) caused by drainage facilities cold joints, etc. as mentioned in the preceding paragraphs. It is ascribed to the convergence of internal pavement stress on structurally weak parts.

The presence of less cracking on the Izumizawa Expressway which is equipped with fewer drainage facilities supports the idea that stress concentration can cause transverse cracking.

b. Low temperature properties of asphalt mixtures

We investigated asphalt and its mixture for their low temperature properties. In the course of the investigation, we collected samples from an expressway between Kita-Hiroshima IC and Tomakomai IC and then tested those mixes for temperatures stress and the recovered asphalt for Fraass breaking point.

The following gives the relative findings.

- Temperature stress test on roads constructed across seedlings plantations and subjected to intense cracking indicated the presence of a higher transition point temperature and a higher rupture temperature than those laid in other regions.
- The Fraass breaking point of recovered asphalt is interrelated with the rupture temperature of the temperature stress test, which implies that there is a possibility that asphalt's low temperature properties can also initiate cracking of asphalt mixtures.

(2) Prevention of transverse cracking

As mentioned in the preceding paragraphs, there is the possibility that lateral cracking is caused by the low temperature properties of asphalt. Accordingly, it is desirable to design the pavement structures of expressways in Hokkaido, using critical regard for their characteristic properties and the quality standard of the asphalt.
Accordingly, JHPC has applied the provisional standards sepcified in Table 2 to trial construction of new expressways.

Table-2 Provisional standards for asphalt and reason

Standard	Reason
Fraass breaking point (less than $-10^{\circ}C$) Note: Film heating (after TFOT)	. Consideration is given to the rupture temperature (below $-30^{\circ}C$) of asphalt mixture (matching rupture temperature to breaking point based on test results) . Formerly, the standard value adopted by the Hokkaido Bureau of Development was $-12^{\circ}C$ (at PI=100-120) as the pre-TFOT value. . In Western Europe, many countries prescribed the Fraass breaking point and the North European Commission has specified the brittle point of $-10^{\circ}C$ (at PI=70-100) as the post-TFOT value and $-12^{\circ}C$ (at PI=100-145) as the post-TFOT value, etc.
Softening point (over $40^{\circ}C$)	. Consideration is given to rutting produced by pavement softening in the summer season. . $40^{\circ}C$ is defined as the low limit (at PI=100-120)
Penetration index (PI) (over -0.7)	. Determined in consideration of changes in temperature from the stage of construction to operation (the higher the PI, the lower the heat sensitivity) . The investigation found that the smaller the PI, the greater the cracks (Values: at the stage of blending design)

Trial construction

. Location: Central Hokkaido Expressway
(Takikawa IC - Fukagawa IC: 10,020 m)

. Ashalt used: Straight asphalt (80-100) combined with soft asphalt
(total qty used: 2,400 tons)

. Pavement techniques used: Total layer was surface layer work, foundation layer work and asphalt stabilized subbase (Fig. 11)

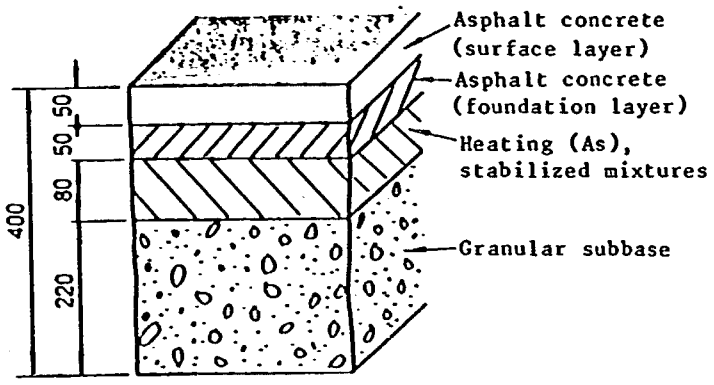


Fig-11 Pavement section (Unit: mm)

Table 3 shows the properties of asphalt used, while Table 4 presents the results of the Marshal test on mixes used.

Table-3 Properties of asphalt used

Product name	Asphalt A		Ashalt B	
Classification	Straight asphalt (80-100)		Soft asphalt	
Process used	Direct distillation		Direct distillation	
Properties	Standard value	Manufactured in July 1988	Standard value	Manufactured in July 1988
Penctration [25℃] (1/10 mm)	80-100	87	-	140
Softening point (°C)	42-50	45.5	Min. 40	43.0
Penetration index (PI)	-	-1.0	Min. -0.7	-0.3
Specific gravity	Min. 1.0	1.0316	Min. 1.0	1.0209
Fraass breaking point (°C) [after TFOT]	-	-14	Max. -1.0	-15
Moriyoshi breaking point (°C) [after TFOT]	-	-23	Reference value (max. -26)	-26
Optimal mixing temperature (°C)	-	154	-	145
Optimal compaction temperature (°C)	-	142	-	133

Table-4 Plant Marshal test results on mixtures

Item	Unit	Aspahlt A			Asphalt B		
Working techniques		Asphalt base	Binder	Sur-face	Asphalt base	Binder	Sur-face
Asphalt (qty)	%	4.2	5.7	6.6	4.2	5.7	6.6
Density	g/cm^2	2.338	2.381	2.359	2.336	2.380	2.358
Stability	kg	767	958	1.197	710	794	1.121
Flow value	1/10mm	31	29	30	33	29	33
Void	%	6.5	4.4	3.6	6.5	4.5	3.8
Aggregate voids filled with asphalt	%	59.0	75.0	80.7	59.6	74.0	80.0
Residual stability	%	82.8	85.6	85.0	83.5	85.3	83.8

In Table 3, two types of asphalt are similar in respect to the Fraass breaking point, but from the nomogram in Fig.12, a difference of 6-7°C in expected crack temperature is noted. This experimental zoen has been commissioned to service since September 1989 without any sign of cracking to date, but JHPC will engage in continuous investigation.

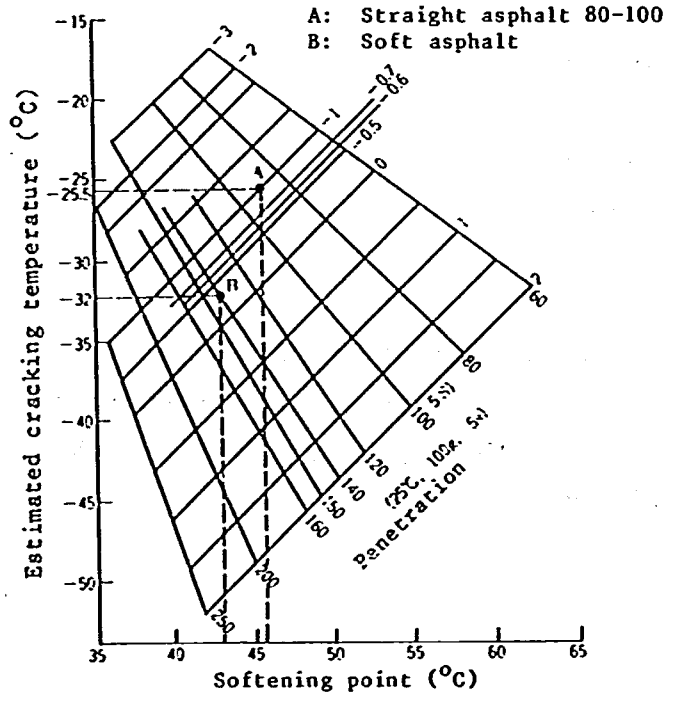


Fig-12 Estimated nomogram for cracking temperature based on asphalt softening point penetration and PI[6)]

3.2 PROTECTION AGAINST RUTTING

Studies on wear resistance of asphalt mixtures have been conducted by any organizations[4].

JHPC Laboratory has also been engaged in testing antiwear mixtures using a revolving pavement tester[5].

JHPC conducted the following studies on workability and wear resistance through trial pavement of commerical highways based on the research results.

The findings are given below:

- 20 mm sized coarse aggregate resists wear more than 13 mm sized aggregate.
- There is no benefit in adding rubber to asphalt.
- Gap grade is unrelated to wear.
- The smaller the Los Angeles abrasion weight reduction of coarse aggregate, the greater the resistance against wear.

Taking these findings into account, JHPC is employing 20 mm (max.) coarse aggregate for pavement of new roads where surface layer thickness can be optionally determined.

Also, JHPC uses converter slag with minor loss of Los Angeles abrasion loss on the occasion of renovation to improve wear resistance of thse mixtures.

Also, in addition to the sophistication of the materials used, JHPC provides surface layer regeneration with resources and energy-savings and cost efficiency through the repaving process for about 10 percent of the total renovation works.

4. CONCLUSION

This paper discussed the question of damage inflicted on expressways in Hokkaido and proposed solutions in preceding paragraphs. In this connection, no basic solution has yet been reached. With respect to transverse cracking, JHPC is planning to continue investigations in the trially paved (soft asphalt) Central Hokkaido Expressway (Takigawa IC - Fukagawa IC) for the next several years. We assume it will take sometime before we can ascertain it seffectiveness. With regard to wear-inflicted rutting on highways, a virtual limit has been reached in terms of our protective capacity. Accordingly, more effective operation of our limited budgets is important.

In this sense, our new approach toward greater sophistication of enginering process will become vitally important.

There is the prospect, as a matter for the near future, that the use of spike tires will be totally banned. In such a case, new countermeasures will have to be employed to upgrade road serviceability and assure traffic safety in winter. Critical observation should continue to be fixed on potential moves in this direction.

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Asphalt Technology for Hot In-Place Surface Recycling

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ASPHALT TECHNOLOGY FOR HOT IN-PLACE SURFACE RECYCLING

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ABSTRACT

The Heat Reforming Process for in-place, hot rehabilitation of deteriorated asphalt pavements consists of a heating system (up to three LPG infrared preheaters) which can effectively heat to a depth of 50 mm, and a reforming system (Reform, Remix and/or Repave options) which can readily improve the pavement quality through rejuvenation and/or place a new surface course, from very thin to 50 mm in thickness, in one pass. The main asphalt technology aspects of the Heat Reforming Process can be summarized as: determining the overall suitability of the aged, cracked and/or rutted pavement for processing (must be structurally adequate for instance); testing the existing pavement to determine the necessary rejuvenator application rate (asphalt cement content, recovered penetration/viscosity, etc.); and monitoring the quality of the in-place, hot recycled mix and any new mix during and after processing, including surface tolerance. No problems have been encountered in meeting 'softening' specifications for aged pavements. The Heat Reforming Process has also been extended to effectively treat asphalt pavements with problems such as severe flushing and/or low insitu air voids. The key requirements for cost-effective, technically sound, in-place, hot rehabilitation are described through Heat Reforming Process project experience and the associated asphalt technology.

ACKNOWLEDGEMENT

This paper is based on North American experience gained from 1987 to 1989 with the Heat Reforming Process equipment and system developed by the Taisei Road Construction Co., Ltd., Tokyo, Japan. The Paving in Cold Areas (PICA) workshops have been an important element in this technology transfer. As part of the Heat Reforming Process technology transfer, similar technical presentations have been made to the Roads and Transportation Association of Canada (1988), Canadian Technical Asphalt Association (1989) and Transportation Research Board (1990). Specific equipment and materials are indicated throughout by: HRP - Taisei Heat Reforming Process; RR - Shell RJO Rejuvenator; RC - Witco Cyclogen® L Rejuvenator; RS - Sunoco Sundex 790 Rejuvenator; and OR-60 - Osaka Cement Company Limited (Taisei application in HRP) OR-60 Artificial Aggregate. The opinions and findings expressed or implied in this paper are those of the authors, and not necessarily their organizations. Comments would be most appreciated.

1. INTRODUCTION

The objective of in-place, hot asphalt pavement surface recycling is to cost-effectively restore the existing aged, cracked, worn and/or rutted surface course to the same quality as a new hot-mix overlay [1-4]. Pavements generally suitable for such hot surface recycling have adequate structural performance (no structural defects beyond localized areas that can be repaired), without prior treatments (surface treatment, rubberized asphalt, epoxy patching, etc.) that may preclude recycling, unless removed first (by milling for instance). Over the past ten years, mainly as a result of surface rehabilitation equipment and technology developments in Europe and Japan, the process has evolved from simple heater/scarification to hot surface recycling with rejuvenation, and new hot-mix overlay placement in one pass, capabilities. The focus of this paper is the Heat Reforming Process (HRP-5 and HRP-6) for in-place, hot, asphalt pavement surface rehabilitation (Reform, Remix and/or Repave Options), based on experience gained with North American highway and airport projects since 1987 [5]. The key requirements for successful in-place, hot rehabilitation, including remediation of problems such as severe flushing, are described through this project experience and the associated asphalt technology.

2. HEAT REFORMING PROCESS

The Heat Reforming Process (HRP-5 and HRP-6, noting that there are several other comparable systems), as shown in operation in Figure 1 (HRP-5) and schematically in Figure 2, typically consists of: a LPG infrared heating system (up to three preheaters with up to 5.58 million kcal/hour total heating capacity) which can effectively heat to a depth of 50 mm; and a reforming system that can apply additional heat (up to 616,000 kcal/hour), add rejuvenator, mix rejuvenator into the old mix, add new mix, and/or place a very thin (~10 mm) to 50 mm in thickness, new hot-mix overlay in one pass. This overlay is well bonded to, and acts monolithically with, the rejuvenated old mix, as compared to conventional overlays. These components, and related features, are also shown in Figures 3 and 4. There are four basic Heat Reforming Process

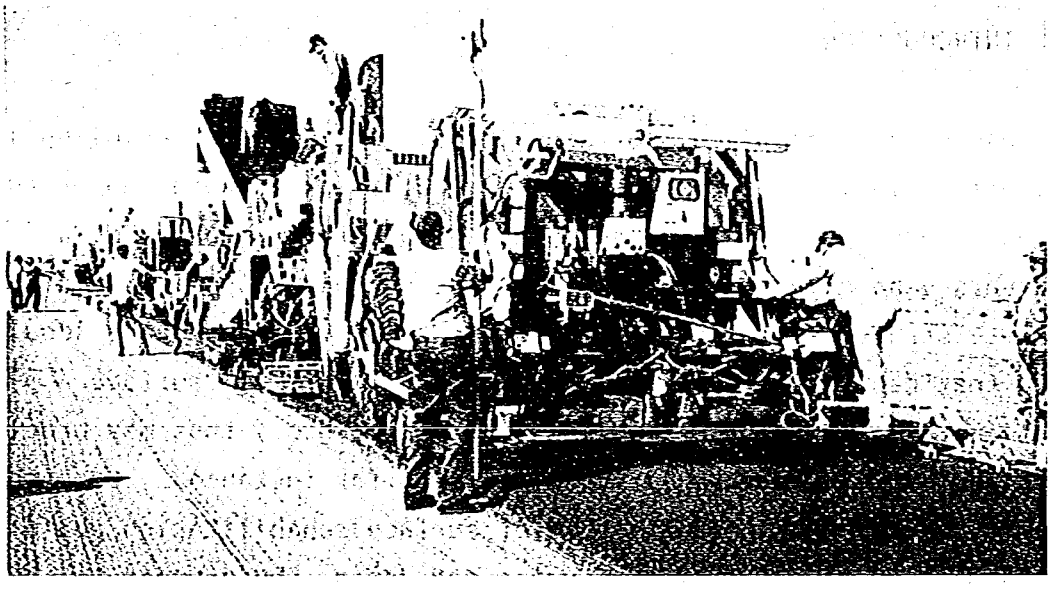
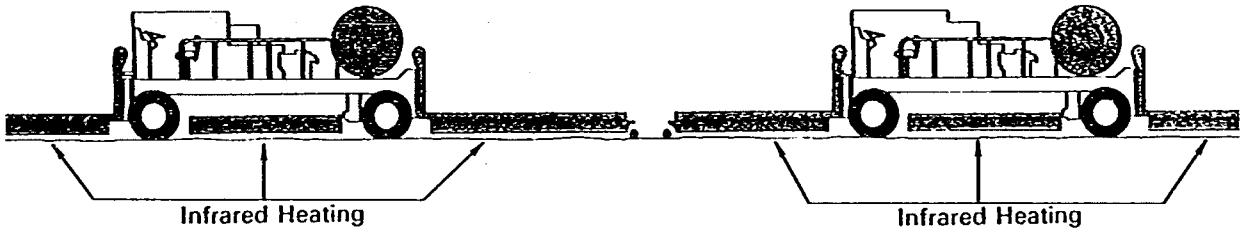
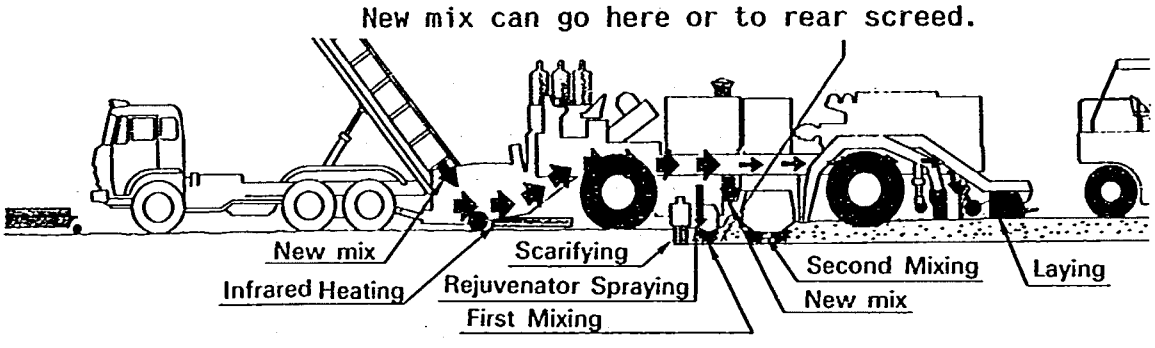


FIGURE 1 GENERAL VIEW OF HEAT REFORMING PROCESS (HRP-5) AT SPOKANE INTERNATIONAL AIRPORT SHOWING THE TWO PREHEATERS AND REFORMER. The pre-milled (75 mm) keel section involved heater/ scarification/rejuvenation (50 mm) with new surface course placement (50 mm) in one pass (Remix-Repave Option) [6].



LPG Infrared Heating System

The LPG infrared heating system can heat the existing aged asphalt pavement up to 50 mm in depth to a specified temperature without degrading the asphalt.



Reform, Remix and Repave System

The reforming system can apply additional heat, scarify to required depth, add selected quantity of rejuvenator, mix rejuvenator into old mix (first mixer), add new hot mix to rejuvenated old mix (second mixer) and/or place an overlay from very thin (~10 mm) up to 50 mm.

FIGURE 2 EQUIPMENT INVOLVED IN HEAT REFORMING PROCESS (HRP-5) FOR ONE PASS, IN-PLACE, HOT, ASPHALT PAVEMENT SURFACE REHABILITATION [2].

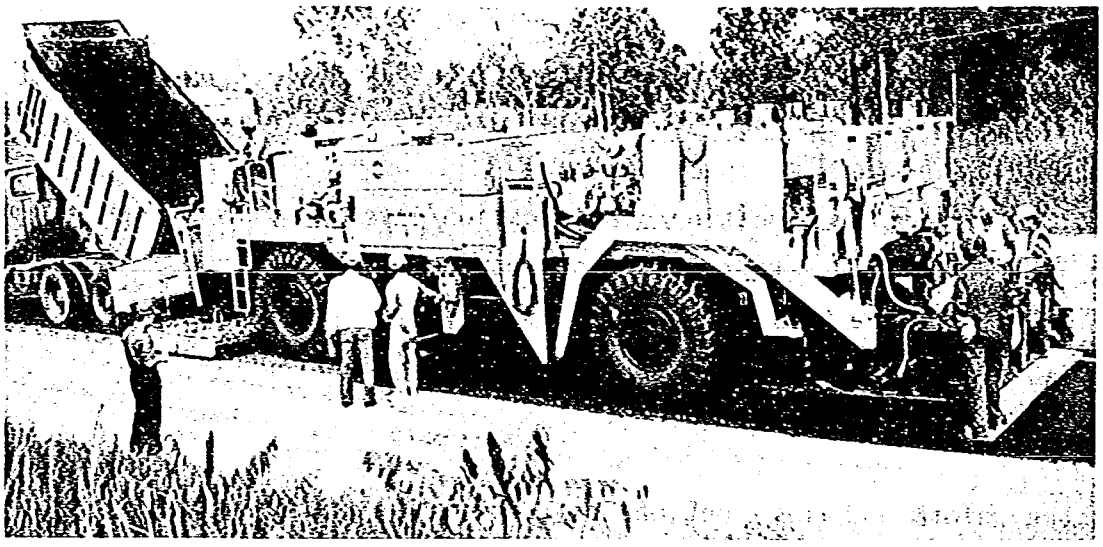


FIGURE 3 HEAT REFORMING PROCESS (HRP-5) ON ONTARIO HIGHWAY 5/24, SHOWING REFORMER (40 mm WITH REJUVENATION, REMIX OPTION) [7].

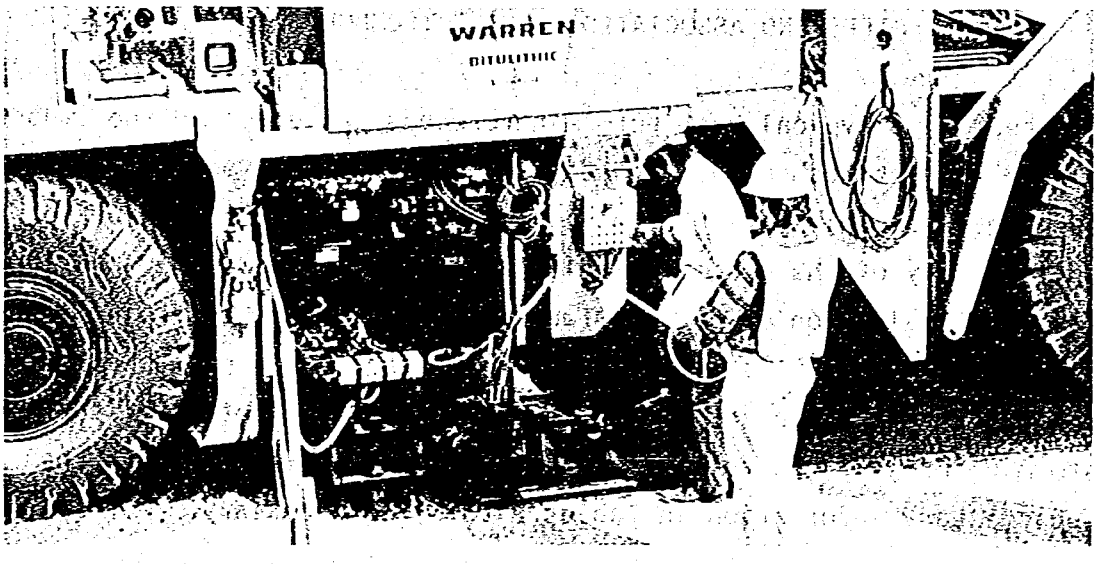


FIGURE 4 CLOSE-UP OF REFORMER WITH REJUVENATOR TANK AND METERING (TOP); AND SCARIFIER, REJUVENATOR SPRAYS, FIRST MIXER, CONTROLS AND SECOND MIXER (FROM LEFT TO RIGHT BETWEEN THE WHEELS). The first mixer blends the rejuvenator into the old mix and can also 'mill' to ensure the required depth of scarification.

options, as described in more detail in following sections:

Reform Option - heating/scarification/levelling/reprofiling/compaction;

Remix Option - heating/scarification/rejuvenator/mixing (and/or new hot mix/mixing)/levelling/reprofiling/compaction;

Repave Option - heating/scarification/levelling/laying new hot mix/reprofiling/compaction; and

Remix-Repave Option - heating/scarification/rejuvenator/mixing/levelling/laying new hot mix/reprofiling/compaction.

As the reform and remix options do not involve the one pass laying of a new hot-mix overlay, a 'simpler' Heat Reforming Process (HRP-6) without overlay capability has also been developed, as shown on highway and airport projects in Figures 5 and 6. To minimize the number of stops for LPG tank refilling, preheaters with a full shift LPG tank capacity are now being used, such as shown for the preheaters in Figure 5.

3. PROJECT STEPS AND ASSOCIATED ASPHALT TECHNOLOGY

The steps in a typical heat reforming process project, and the important related asphalt technology aspects, are outlined in a series of tables covering: general steps (Table 1); preliminary pavement evaluation and applicability of the process (Table 2); detailed pavement evaluation (Table 3); selection of option (Table 4); and quality control (Table 5) [1-4]. It is important that the procedures given in Table 1, from preliminary pavement evaluation through to completion of the Heat Reforming Project, be followed step-by-step, with reference to the supplementary information in Tables 2 to 5, as necessary. For instance, the preliminary pavement evaluation (Step 1, Table 1) must be of sufficient scope (Table 2) for the agency to be able to make a decision on the applicability of the Heat Reforming Process (Step 2, Table 1) on the basis of pavement structure adequacy, any potential problems with prior treatments and cost relative to other possible pavement

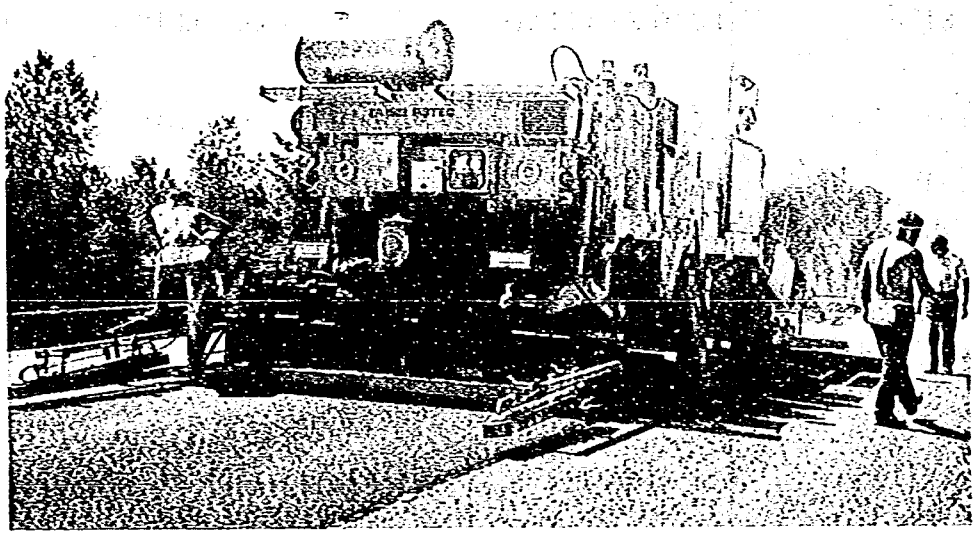


FIGURE 5 HEAT REFORMING PROCESS (HRP-6) ON BRITISH COLUMBIA HIGHWAY 1 NEAR LANGLEY (40 mm WITHOUT REJUVENATION, REFORM OPTION) [8].

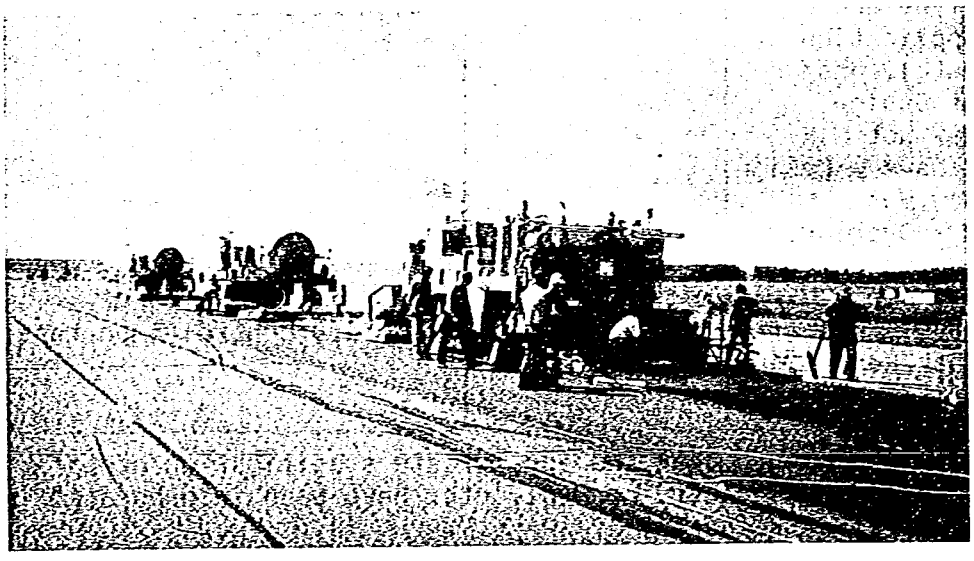


FIGURE 6 HEAT REFORMING PROCESS (HRP-6) ON RUNWAY 15-33 AT PRINCE GEORGE AIRPORT (50 mm WITH REJUVENATION, REMIX OPTION). The HRP-6 is a smaller, 'simpler' version of the HRP-5 intended for the Reform and Remix Options, rather than Repave Option [9,10].

TABLE 1 GENERAL STEPS IN A HEAT REFORMING PROCESS PROJECT [2]

STEP	FURTHER DETAILS	COMMENTS
1. Preliminary Pavement Evaluation	Table 2	Mainly to determine if pavement structure is adequate.
2. Applicability Yes ↓	Table 2	If Heat Reforming Process not applicable, develop alternative rehabilitation/ reconstruction method(s).
3. Detailed Pavement Evaluation	Table 3	Mainly quality and properties of existing surface course.
4. Selection of Heat Reforming Process Option	Table 4	Reform, Remix, Repave or Remix-Repave.
5.a. Remix - Select Rejuvenator (Type and Application Rate), and/ or Design New Hot Mix. b. Repave - Design New Hot Mix Overlay.	Figure 7	Major asphalt technology aspects in conjunction with Step 3.
6. Completion of Heat Reforming Process Project	Table 5	Quality control important.

Note: It is assumed that the appropriate specification preparations, tendering, quality control, etc. have been incorporated in the steps, as required.

TABLE 2 PRELIMINARY PAVEMENT EVALUATION [1,2]

Mainly Adequacy of Existing Pavement Structure

ITEM	DETAILS	REASON
Inventory Information	·class of pavement ·pavement structure[a]. ·pavement history ·traffic volume	·work schedule ·applicability of Heat Reforming Process ·supplements detailed evaluation ·work schedule
Pavement Structure	·structural defects (types and extent)[c]. ·non-structural defects (types and extent) ·localized structural defects	·applicability of Heat Reforming Process ·selection of Heat Reforming Process Option ·need for preliminary localized repairs
Prior Treatments (See Inventory Also)	·any special treatments or materials (surface treatment, rubberized asphalt, road markings, fabrics, epoxy patching, etc.)	·need for removal (cold milling for instance), if possible, before Heat Reforming Process
Geometry and Profile	·width, alignment and gradient[a]. ·surface profile (extensive rutting and wear)[b].	·applicability of Heat Reforming Process ·need for preliminary treatment (cold milling for instance) if possible, before Heat Reforming Process
Miscellaneous	·manholes, catch-basins, utility covers, etc. ·adjacent (close) plants, trees, flammables, etc.	·work schedule, protection and potential flammable gas countermeasures ·work schedule and protective action as necessary

Notes: a. In general, a pavement with structural defects (i.e. lack of structural capacity and/or inadequate base, beyond localized defects that can be readily repaired) will not be a suitable candidate for the Heat Reforming Process. Pavements with non-structural surface defects (rutting, wearing, cracking, aging, poor frictional characteristics, etc.) are suitable candidates for the Heat Reforming Process.

b. Pavement width, alignment and/or gradient improvement requirements, or excessive rutting and wear (greater than about 50 mm), may preclude the Heat Reforming Process.

TABLE 3 DETAILED PAVEMENT EVALUATION [1,2]

Mainly Quality and Properties of Existing Surface Course

ITEM	DETAILS	TYPE OF SURFACE DEFECT[a]. WEAR	RUTTING	CRACKING	FRICTION
Surface Condition	·cracks (types and extent)	N	N	M	N
	·transverse profile	M	M	N	R
	·longitudinal profile	R	R	N	N
Existing Asphalt Concrete[b]. (Usually surface course, but must be at least to proposed scarification depth.)	·thickness	M	M	M	M
	·asphalt cement content (for scarification depth)	M	M	M	M
	·gradation (for scarification depth)	M	M	M	M
	·density	M	M	M	M
	·air voids	M	M	M	M
	·penetration/viscosity of recovered asphalt cement (for scarification depth)	M	R	M	N

M - Mandatory R - Recommended N - Not Necessary

Notes: a. Information to be representative of the pavement section involved, with special areas (spray patching for instance) and localized structural distress areas noted.

b. Typically based on a coring program. Cores to be representative of pavement section involved, with additional cores taken as necessary for special areas.

TABLE 4 HEAT REFORMING PROCESS (HRP) OPTIONS [2,3]

PURPOSE[a]	OPTION	PROCESS
To improve the profile of surface course deformed by rutting or wearing, but in comparatively unaged condition with minor cracking (no rejuvenator required)[b].	Reform HRP-5 or HRP-6	Heating - Scarification - Levelling - Reprofiling[c] - Compaction[d].
To improve the quality of old, cracked, aged surface course by the addition of rejuvenator and/or new hot mix[e].	Remix HRP-5 or HRP-6	Heating - Scarification - Rejuvenator - Mixing - and/or New Hot Mix - Mixing - Levelling - Reprofiling - Compaction
To improve the profile of surface course severely deformed by rutting or wearing, with new hot-mix overlay placed in one pass. To improve frictional characteristics. To provide some pavement strengthening.	Repave HRP-5	Heating - Scarification - Levelling - Laying New Hot Mix[f] - Reprofiling - Compaction
Combination of Remix and Repave purposes.	Remix-Repave HRP-5	Heating - Scarification - Rejuvenator - Mixing - Levelling - Laying New Hot Mix - Reprofiling - Compaction

Notes: a. Prime purpose given in each case.

b. Often used prior to hot-mix resurfacing (heater/scarification).

c. Standard screed and screed controls.

d. Standard compaction equipment and procedures.

e. The composition, gradation and/or asphalt cement content of the new hot mix can be adjusted to improve the quality of the old mix.

f. Standard augers and auger controls.

TABLE 5 QUALITY CONTROL FOR HEAT REFORMING PROCESS [1,2][a].

ITEM	RECOMMENDED METHOD[b].
Width	•as usual
Depth of Scarification	•for HRP (depth set) - measure depth from existing surface adjacent to second mixer •for others - circular ring method
Rejuvenator Application Rate (if any)	•calculate from quantity used (litres)
Rejuvenator Quality (if any)	•as usual (specifications and ASTM D 4552 [11])
New Mix Addition Rate (if any)	•calculate from quantity used (tonnes)
Thickness of New Hot Mix Overlay (if any)	•calculate from quantity used (tonnes)
Temperature at Breakdown Rolling	•monitor at mid-point of re-profiled depth
Temperature of New Hot Mix (if any)	•as usual
Asphalt Cement Content, Gradation and Marshall Compliance of New Mixes (if any)	•as usual
Compaction	•as usual (nuclear density for establishing rolling pattern, cores for acceptance) - important to compare to relevant re-compacted density
Surface Tolerance	•as usual
Penetration/Viscosity of Recycled Mix	•as usual

Notes: a. As the Heat Reforming Process (HRP) is largely based on conventional hot-mix paving technology, it is only necessary to supplement the usual quality control requirements. The quality control items and frequency of testing should be established at the level necessary to ensure specification compliance.

b. All testing should be done on random, representative samples by qualified technicians in a certified laboratory.

rehabilitation strategies (milling and hot-mix plant recycling for instance). Any special features, such as severe flushing, can also be considered at this time. If the Heat Reforming Process is applicable, then the detailed pavement evaluation information (Step 3, Table 1), developed as outlined in Table 3, is required for the selection of the Heat Reforming Process option (Step 4, Table 1), and so on. Where there appear to be other appropriate rehabilitation strategies available, agencies sometimes tender the Heat Reforming Process option selected along with alternatives such as milling/hot-mix plant recycling/ conventional overlay in order to obtain the most cost-effective approach.

In addition to the asphalt technology aspects determined by the detailed pavement evaluation (Table 3) and quality control requirements (Table 5), it is necessary to establish the rejuvenator type and application rate for the project. This is generally made the responsibility of the contractor, along with any necessary hot-mix designs, if end result specifications have been adopted. It is also necessary to establish the desired recycled mix average temperature and minimum temperature required at the bottom of the scarified depth.

Laboratory evaluations have been completed on RR, RC and RS rejuvenators [11,12] that have shown RR to meet standard recycling agent requirements [13 for instance] and to be cost-effective. Supportive field experience with the RR rejuvenator was obtained in 1987 in Ontario [5,7]. Before turning to this information, it should be pointed out that the Abson method for recovery of asphalt cement [ASTM D 4552, 14] requires considerable technician skill and experience to obtain reliable recovered penetration/viscosity data. For this reason, the proposed ASTM rotavapor approach [15] is being evaluated as it can be 'automated' and has shown reasonable laboratory and inter-laboratory accuracy. The selection of RR rejuvenator quantity for softening aged asphalt cements, based on laboratory testing of both recovered and artificially aged asphalt cements [11,12], is summarized on Figure 7 along with an example of its use in terms of penetration improvement. Using Figure 7, the RR rejuvenator application rate can be developed for any penetration

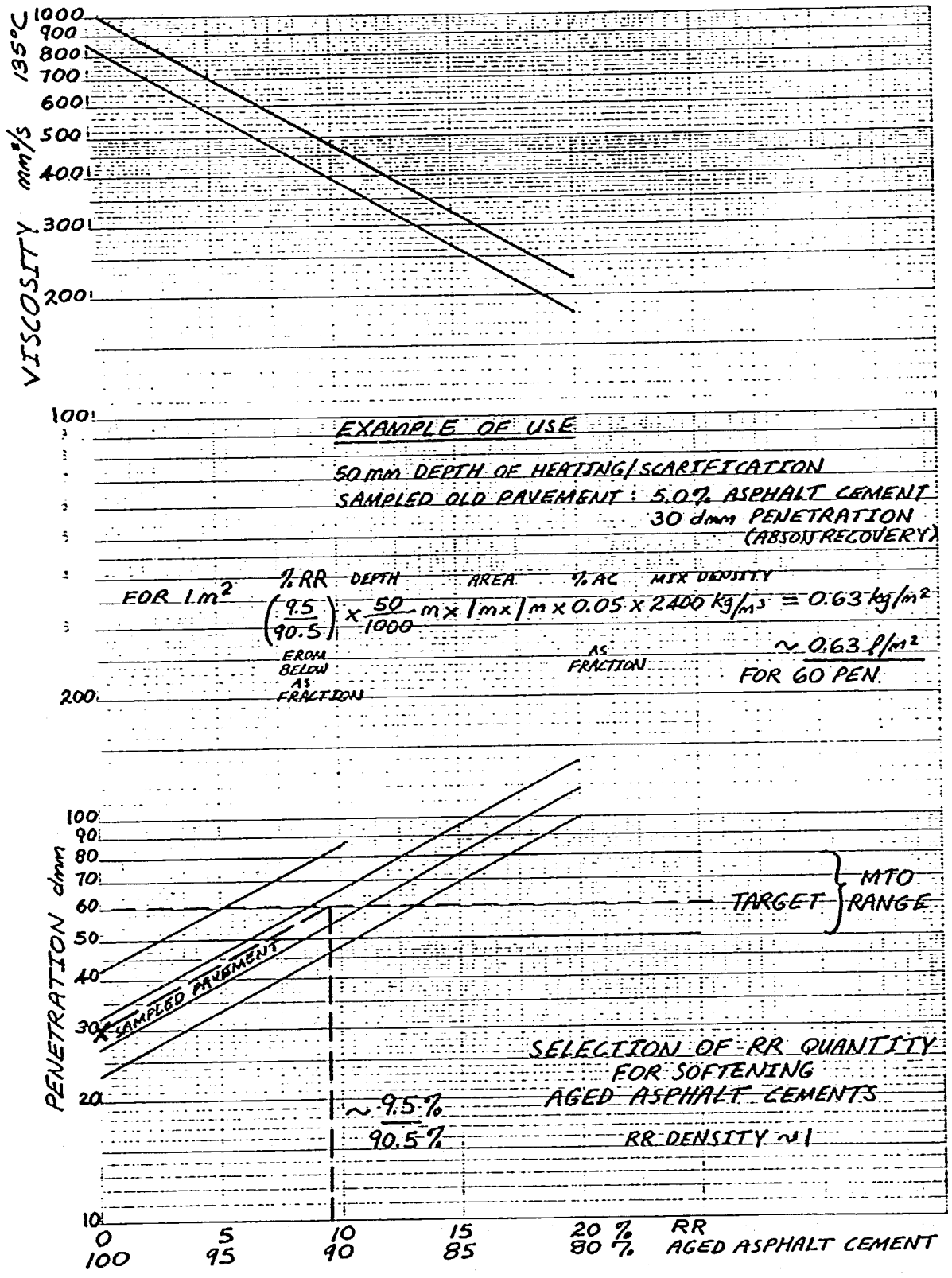


FIGURE 7 SELECTION OF RR REJUVENATOR FOR SOFTENING AGED ASPHALT CEMENTS [11,12].

and/or kinematic viscosity requirements. A similar approach can, of course, be used for the RC and RS rejuvenators. It should also be noted that RR rejuvenator application temperature data (desirable viscosity) have also been developed. It is generally advised that the lowest suitable RR rejuvenator application temperature be used, typically about 70 to 80°C, to avoid any misting during incorporation and mixing.

The average recycled mix temperature required for satisfactory compaction is in the 105 to 115°C range at the breakdown roller, depending on specific site and ambient conditions. Generally, no problems have been experienced with achieving specified compaction levels, particularly when the Repave Option is involved. In order for the scarification of the old asphalt concrete to be effective and efficient, experience has shown it is desirable for the minimum 'bottom' temperature to be the softening point temperature for the project recovered asphalt cement before rejuvenation [16]. Approximate penetration, softening point and absolute viscosity data are given in Table 6 that can be used to select this desirable minimum temperature.

4. PERFORMANCE

Some specific project experience can be used to outline how well 'projected' softening of aged asphalt, from the Figure 7 data, was achieved in the field. It is also of interest to know how much 'damage' LPG infrared heating does to an existing pavement, which of course requires data from projects where rejuvenation was not involved. For a British Columbia Reform Option project [8], the before Heat Reforming Process (HRP-6) data were an average penetration of 63 dmm, average kinematic viscosity (135°C) of 328 mm^2/s and average absolute viscosity (60°C) of 212 Pa.s, compared with after reforming data of an average penetration of 59 dmm, average kinematic viscosity (135°C) of 299 mm^2/s and average absolute viscosity (60°C) of 247 Pa.s, i.e. within the accuracy of testing, there was little 'damage' to the pavement [17].

TABLE 6 APPROXIMATE PENETRATION, SOFTENING POINT AND ABSOLUTE VISCOSITY RELATIONSHIPS [16][a.]

PENETRATION[b.] dmm	SOFTENING POINT[d.] °C	ABSOLUTE VISCOSITY[c.] Pa·s
5	84	690,000
10	74	140,000
15	69	57,000
20	64	29,000
25	61	18,000
30	59	12,000
35	57	8,400
40	55	6,100
45	53	4,800
50	51	3,700
55	50	3,000
60	49	2,400

Notes: a. From empirical relationships developed from extensive test data on recovered asphalt cements.

b. The softening point-penetration relationship is approximate.

c. The softening point-absolute viscosity relationship is fairly 'accurate'.

d. The minimum 'bottom' temperature for scarification should be the softening point temperature for the penetration/absolute viscosity of the aged pavement's recovered asphalt cement before rejuvenation. Testing should be on samples representative of the proposed scarification depth.

For a 1987 Heat Reforming Process (HRP-5) project in Ontario (Figure 3) with the Reform Option and significant quality control testing, the RR Rejuvenator application rate was set (0.40 l/m^2) to achieve a recovered average penetration of about 55 dmm, somewhat above the low end of the Ministry (MTC, now MTO) specified range of 50 to 80 dmm. The aged asphalt cement had an average initial penetration before the Reform Option of 34 dmm, and the average recovered penetration after the Reform Option was 54 dmm, close to the 'softening' level anticipated [7]. (The parallel Ministry testing indicated a somewhat lower 'softening' level of 44 dmm was achieved, but their data (not ASTM D 1856 method) includes tests on control sections without rejuvenator application [18].

The compaction level achieved (>98 percent) was satisfactory and the Marshall compliance testing was favourable, with the exception of somewhat low air voids. When a rejuvenating agent is added to an old mix, the effective binder content in the recycled mix does increase (by about 7 to 10 percent), resulting in lower air voids. It is necessary that this impact on field air voids be checked, and options such as the addition of an under-asphalted new hot mix or hot sand be considered, as proven effective on a number of Remix Option projects. Regardless, from both an economic and overall recycled mix quality viewpoint, the rejuvenator addition rate should be set as low as possible.

An obvious question at this point is the relative cost of the Heat Reforming Process options compared to conventional hot-mix paving technology. Data on alternative bids, <u>where equivalent quality and pavement modifications have been required</u>, show the Heat Reforming Process to be 10 to 20 percent lower in cost. As the actual beneficial depth of heater/scarification extends below the specified nominal depth (i.e. have elevated temperature, with some 'closing' of cracks below scarification depth, for instance), the actual cost-effectiveness is probably better than this level. In addition, specific site conditions, and particularly ambient conditions (prefer hot, calm days for hot in-place recycling), influence the project costs. For instance, under good operating conditions, the HRP-5 in the Remix(50 mm)-Repave(50 mm) Option can achieve up to 1000 m^2/h. With street work that involves utility

covers and/or poor ambient conditions, this production rate can drop to less than 500 m^2/h. Regardless, in comparing the costs of alternatives, it is important that the quality and pavement modification requirements (i.e. end result) be equivalent. In addition, the Heat Reforming Process has the public attractions of much reduced traffic impact during construction and the recycling of pavement materials.

5. PROBLEM PAVEMENTS

Projects recently completed demonstrated methods by which the Heat Reforming Process technology can be used to address localized asphalt pavement problems such as flushing/bleeding, low insitu air voids, etc.

5.1 Severe Flushing/Bleeding

Variable, moderate to severe flushing/bleeding of wheelpath areas (and associated wheelpath rutting) developed on several relatively high traffic rural roads in Southwestern Ontario shortly after MC800 hot mix binder course had been placed on the existing roadways [19]. This binder course was scheduled to be overlaid with standard hot-mix surface course within a few weeks of the MC800 hot mix placement. The nature and severity of the typical flushing/bleeding distress involved is indicated in Figure 8.

The flushing/bleeding problems were attributed to moisture in the mix that caused both poor coating of the coarse aggregate and excessive fluids (moisture and/or somewhat high MC800 - asphalt cement/cutback - content). The flushing/bleeding was not continuous and appeared to be only in the top 19 to 25 mm of the MC800 binder course. The 'normal' remedial action would have been complete removal of the flushing/ bleeding material and replacement with suitable material meeting the agency specification requirements. However, site evaluation and laboratory testing of core samples suggested that the Heat Reforming Process could be used to treat the excessive flushing/bleeding without removal [19].

The recommended remedial action consisted of spreading 12 to 19 mm of hot ($\sim 180^0$C) asphalt sand on top of the bleeding areas (Figure 9). This operation was followed by thorough in-place remixing of this hot asphalt sand with the top 25 mm or so of MC800 binder course, then relaying (HRP) and compacting to give a satisfactory improved MC800 binder course driving surface for subsequent hot-mix surface course placement.

The hot asphalt sand was placed directly over the flushing/bleeding areas immediately ahead of the heat reforming unit. Due to the shallow treatment depth involved, 'soft' nature of MC800 mix and heat provided by the hot sand application, it was not necessary to use any preheater units. Additionally, with the excess MC800 present, preheater use was avoided due to the potential for 'blue smoke' generation and possible ignition of volatiles from the MC800. The softened MC800 mix/hot asphalt sand mixture was then scarified and thoroughly remixed and relaid to grade using the heat reforming unit (HRP) (Figure 10), followed by conventional compaction.

It was intended that the rehabilitated MC800 binder course surface be left to 'cure' for about two weeks and then a new hot mix surface course placed, after selective removal of any small areas of flushing that might still subsequently develop. However, the rehabilitated MC800 binder course was actually left under traffic for about six weeks, with no evidence of the previous flushing, even in localized areas where the previous bleeding had been severe. The hot mix was placed conventionally, without any problems, but was designed to have air voids near the agency upper limit of five percent to reduce any future flushing potential (Figure 11).

5.2 Low Insitu Air Voids

Highway 140 near Welland, Ontario is a two lane secondary highway of moderate traffic volume with an existing surface approximately 30 years old [20]. This project was selected by the Ontario Ministry of Transportation (MTO) in 1989 as a 'demonstration project' to assess the suitability and performance of the Heat Reforming Process equipment

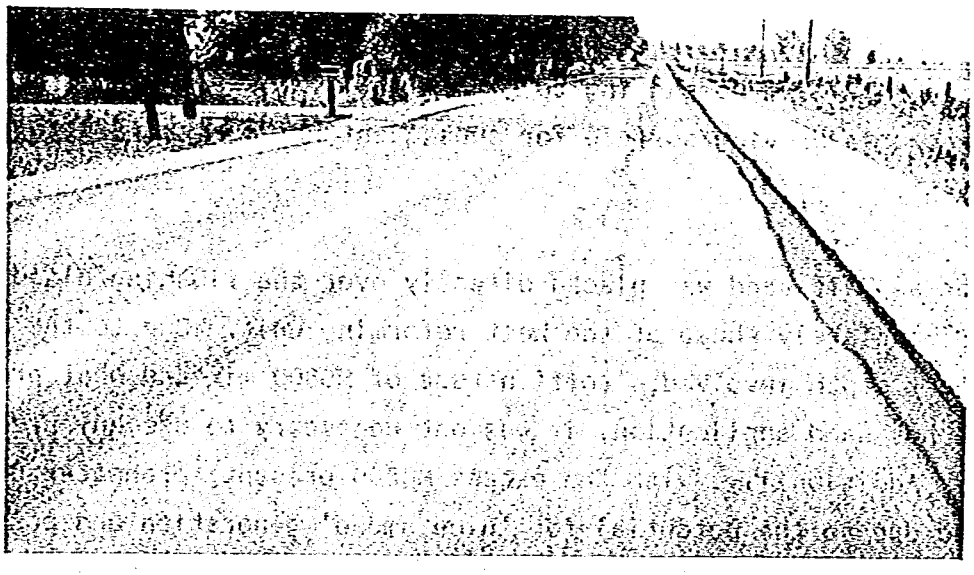


FIGURE 8 SEVERE FLUSHING OF MC800 MIX BINDER COURSE [19].

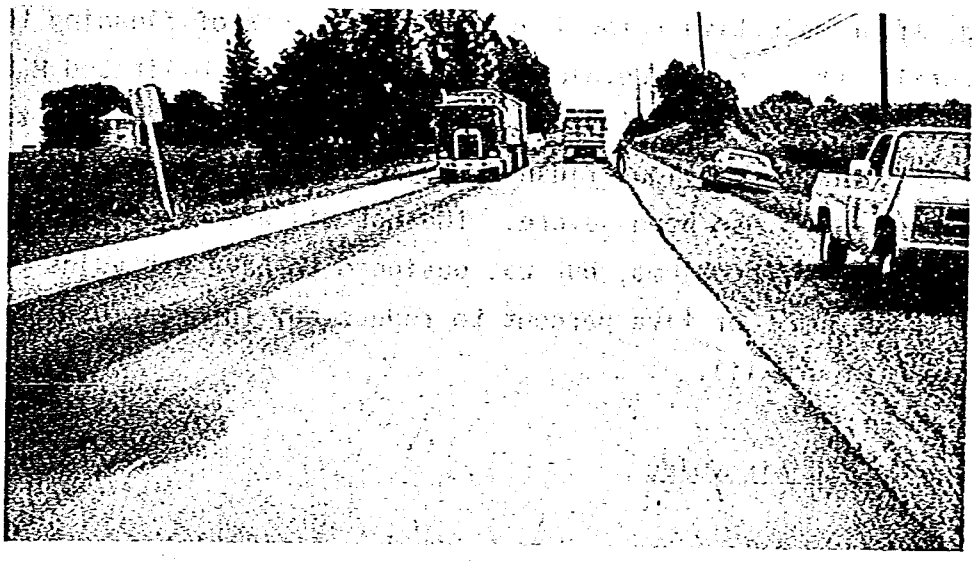


FIGURE 9 HOT SAND PLACED APPROXIMATELY 12 TO 19 mm THICK IMMEDIATELY AHEAD OF HEAT REFORMER [19].

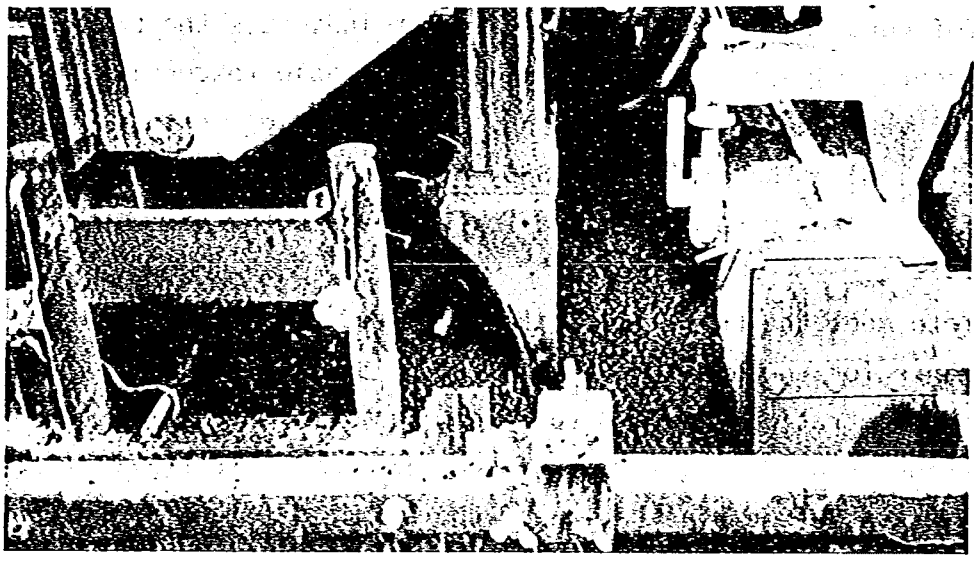


FIGURE 10 MC800 MATERIAL AND HOT ASPHALT SAND AFTER MIXING IN THE REFORMER (HRP) JUST AHEAD OF THE SCREED (ON LEFT) [19].



FIGURE 11 FINISHED ROAD SURFACE AFTER PLACING HOT-MIX SURFACE COURSE (ABOUT SIX WEEKS AFTER HRP TREATMENT) [19].

(HRP) and technology for standard pavement rehabilitation. The project was tendered with hot in-place recycling (and rejuvenation) to a depth of 40 mm with integral application of a 25 mm thick hot mix overlay. Testing of cores obtained from the pavement indicates the existing surface course asphalt concrete was quite variable (extensive hot mix patches of varying age and thickness, original 'old' surface course and spray patching). The air voids (SSD) of recompacted mix from the upper 40 mm of the cores were determined to be only about 2.5%.

In order to meet the project specification recovered penetration requirements for the new mix (between 50 to 80 dmm, with the existing recovered penetration about 29 to 35 dmm), it was determined that RR rejuvenating agent dosage levels of 0.46 l/m^2 and 0.31 l/m^2 were required to raise the penetration of the original pavement and patch areas, respectively, to the specified minimum of 50 dmm. At these RR dosage levels and the relatively low insitu air voids for the existing pavement, there was a strong potential for flushing after in-place hot recycling, with a possible negative impact on the hot-mix integral overlay.

To mitigate potential problems with flushing due to excessive 'liquid' content (existing asphalt cement plus rejuvenator) and low insitu and mix air voids, a thin (5 mm) layer of hot ($\simeq$ 180^{0}C) manufactured sand was applied using a chip spreader between the first and second preheaters (Figure 12). This manufactured sand was thoroughly mixed by the reformer with 40 mm of hot recycled mix (plus added rejuvenator), relaid and compacted. The manufactured sand opened the recycled mix sufficiently that the rejuvenating agent could be added at the prescribed dosage (Figure 13) and no flushing was observed in the new mat (Figure 14). In addition, the thin layer of hot sand acted as an 'ablation' layer absorbing any excess asphalt cement at the surface of the existing mix and preventing direct application of heat to any spray patch areas, thus reducing possible 'blue smoke' problems.

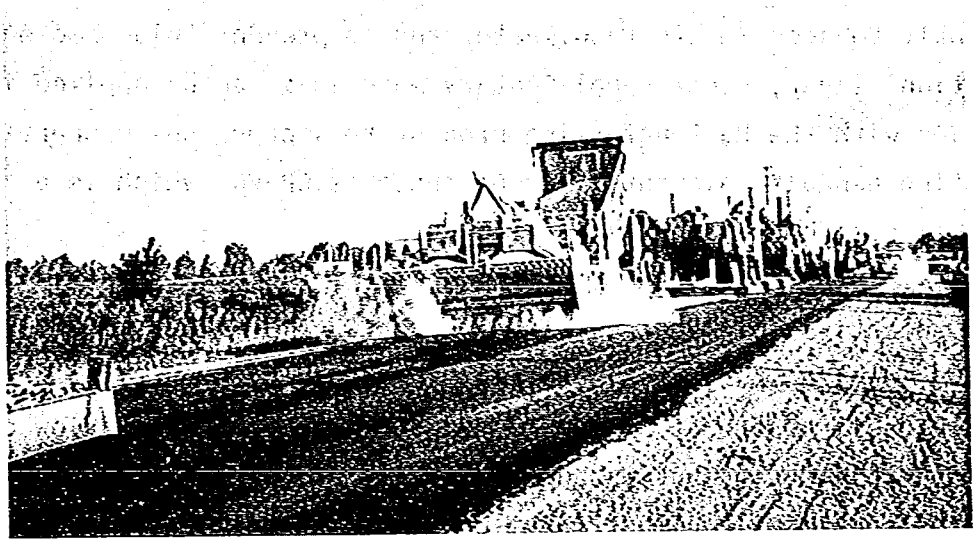


FIGURE 12 HOT MANUFACTURED SAND BEING APPLIED APPROXIMATELY 5 mm THICK USING A CHIP SPREADER BETWEEN THE FIRST AND SECOND PREHEATER [20].



FIGURE 13 HOT MANUFACTURED SAND AND REJUVENATED EXISTING ASPHALT CONCRETE AFTER MIXING BY HEAT REFORMER [20].

6. USE OF SUPPLEMENTARY MATERIALS

In addition to the use of hot sands to open up 'tight' existing mixes sufficiently to accommodate rejuvenator and to prevent 'blue smoke' as an 'ablation' layer, other supplementary materials can be applied in conjunction with the Heat Reforming Process to improve the properties of the existing asphalt pavements. Among these is OR-60, which is a special oil absorbent 'aggregate' [21,22].

OR-60 is an artificial 'aggregate' graded between 5 mm/0.5 mm, that, when added to mixes susceptible to plastic deformation rutting, acts as an inorganic oil absorbent hardener. OR-60 prevents plastic flow of asphalt concrete during hot weather by absorbing the lighter oil fraction of the asphalt cement. The OR-60 can be uniformly spread in a thin layer directly on the asphalt pavement to be heat reformed or directly incorporated through the HRP. At a typical application rate of six percent by mass of asphalt pavement to be treated, the OR-60 causes about 0.3 percent 'reduction' in the asphalt cement content of the mix. It is anticipated that OR-60 will be used during 1990 for HRP projects in the Toronto area.

7. CONCLUDING COMMENTS

In order to take advantage of the demonstrated quality (Figure 15) achieved with the Heat Reforming Process (HRP-5, HRP-6 and similar systems), it is necessary for project specifications to reflect the equipment requirements in terms of: LPG infrared preheaters (to minimize 'damage' of existing pavement); scarifying unit (to ensure specified scarification depth achieved); rejuvenator addition (to have application rate properly inter-locked to rate of reformer travel); first mixer to incorporate rejuvenator into scarified old mix (to both ensure thorough mixing and also establish the depth of processing, typically through incorporating milling teeth); second mixer (to further mix the rejuvenated material and to allow new hot mix addition to correct deficiencies in the old pavement such as air voids and stability); proper first and main screeds (to ensure the necessary

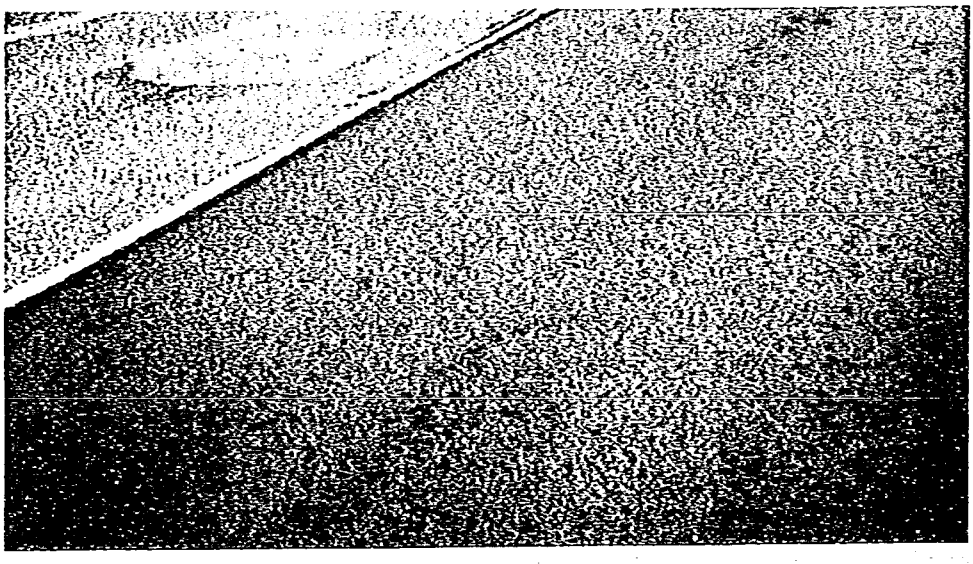


FIGURE 14 ASPHALT MAT BEHIND HRP SCREED PRIOR TO COMPACTING [20].



FIGURE 15 TYPICAL RURAL HEAT REFORMING PROCESS (HRP-5) PAVEMENT SURFACE (JEGEL).

distribution of material and laying quality, main screed typically state-of-the-art with automatic grade and slope controls); and proper machine and mix instrumentation (to expedite quality control checks - infrared thermometers, depth sensors, rejuvenator temperature control, rejuvenator application meter, etc.) As there are several suppliers of suitable Heat Reforming Process equipment, and qualified contractors available, there should be no problem for North American agencies to obtain quality in-place, hot asphalt pavement rehabilitation.

Heat Reforming Process asphalt technology extensions include the addition of asphalt absorbing aggregate (OR-60) and the use of hot sand to remediate severe flushing/bleeding problems. It is anticipated that the growth of the Heat Reforming Process in North America will result in future reports on both asphalt technology improvements and continuing equipment developments for cost-effective, technically-sound, in-place, hot asphalt pavement rehabilitation and remediation.

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Influence of Field Compaction on the Resistance to Stripping of Asphalt Pavements in Cold Areas

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G. Kennepohl, Ministry of Transportation, Ontario

INFLUENCE OF FIELD COMPACTION ON THE RESISTANCE TO STRIPPING OF ASPHALT PAVEMENT IN COLD AREAS

by

A. O. Abd El Halim[1], H. M. El Hussein[2] and Gerhard Kennepohl[3]

ABSTRACT

Extensive research work has been carried out in the past to investigate the effect of compaction on stripping of asphalt pavements specially under cold climate. However, due to the fact that since 1900 only rigid cylindrical rollers were employed, compaction was not considered to have any significant influence on the problem of stripping.

The availability of a field prototype of the new Asphalt Multi-Integrated Roller, AMIR, provides an opportunity to examine whether compaction can improve the mix resistance to stripping. Accordingly, three field tests were carried out in Toronto and Ottawa. Three different asphalt mixes were employed. In each test section standard compactors were used to compact one half of the test section while the AMIR roller compacted the other half.

Asphalt cores and slabs recovered from the field were tested in the laboratory. Test results showed that the new compaction technique improved the overall mechanical properties of the compacted mix. In particular, the results of indirect tensile strength showed more than 10 % improvement in the stripping resistance of the mix . This improvement has been attributed to the elimination of surface induced cracks that occur under current standard compactors. Therefore, the new compaction method is expected to provide crack free pavement structure with higher resistance to the effects of colder weather. This paper reports on the field trials, the testing program and test results.

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INTRODUCTION

Early failure of asphalt pavements has been attributed in many cases to moisture induced damage known as asphalt stripping. Increased maintenance and rehabilitation costs coupled with limited funds to preserve the pavements serviceability, motivated many highway and airfield agencies in Canada to investigate the phenomenon of stripping (1,2). The survey conducted in 1964 by MTO revealed that stripping was evident throughout Ontario. Similar findings were found on most canadian airfields.

The problem of asphalt stripping is a very complex phenomenon influenced by many variables. It is defined as the physical separation of asphalt cement and aggregate produced by either adhesion failure or cohesion deficiency. There is a wide agreement in the literature that water is the main cause of the loss of adhesion and consequently a bond failure in the asphaltic mixture. However, while there are number of theories and mechanisms to explain the stripping phenomenon the problem is yet to be overcome.

Many of the studies performed relate the ease of water movement inside the pavement to initial high air void contents. In Ontario, and in an effort to avoid water movement through pores, asphalt concrete mixes are designed with a void content as low as 2% in the finished compacted mix. This has been expected to make the pavement almost impervious to surface water. According to Haas (2), water still inters the pavement. Jemeneze (3) also reported that asphalt stripping resulted in complete failure of some pavements within two weeks after construction. These observations would suggest that water was trapped in the mix during construction or surface water worked its way through the asphalt surface very short time after construction. In either case, surface cracks had to be present during construction.

Clearly, construction has to be considered when problems associated with asphalt pavements are investigated. However, of all the factors contributing to stripping, influence of construction has been the least investigated (3,4).

IMPORTANCE OF COMPACTION

In a recent study carried out for the Ministry of Transportation of Ontario (5), a number of deficiencies were identified in the current research approaches. First, there is a lack of confidence in the various laboratory test methods developed to predict the susceptibility of asphalt pavements in the field to stripping. Reported test data showed that asphalt cores conditioned to simulate moisture damage have actually resulted in higher strength

values than cores tested dry. Second, laboratory test results of asphalt cores recovered from the field and tested for moisture damage demonstrated high values of variation within the tested cores. This variation can not be explained by statistical analysis. Third, in spite of the reported observations of early failure due to stripping, construction of asphalt pavements was not considered in investigating the stripping phenomenon.
While these three deficiencies appear to be unrelated, the following analysis suggest that considering the influence of compaction on the characteristics of the finished pavement can help answering some of the questions related to stripping.

1. The direction of rolling while compacting the asphalt mat has a significant impact on the tensile strength as well as the stiffness of the compacted mix. For example, an asphalt core recovered from the field can have two significantly different tensile strength values. This will depend on the loading direction during the test relative to the direction of rolling during compaction.

2. The characteristics of a laboratory prepared asphalt samples can be significantly different when compared to similar samples recovered in the field. While both samples may have the same mix proportions and density, differences exist in surface conditions, particle orientation and strength.

3. Existing rollers have a number of geometric and material limitations in their design that can result in hair line cracking and non uniform densities across the width of the compacted pavement.

It is interesting to note that compaction technology did not change significantly since the first static steel roller was used to compact asphalt mixes at the beginning of the 20th Century. Most of the modifications in the current technology were made to improve productivity or obtaining an optimum density with the lowest number of passes. The introduction of vibratory rollers, rubber coated drums and the use of pneumatic rubber wheels achieved improvements in the technology. However, all these rollers have a serious geometric limitation with their cylindrical drums or wheels. Results of research work by Halim et al (6,7) showed that current rollers induce surface cracking during construction. These surface cracks will remain in the pavement allowing surface water and moisture to penetrate the asphalt surface initiating the first phase of the stripping mechanism. Also, in cold areas such as in Canada or Japan, the freezing of water in the cracks will result in additional stresses acting on the faces of the cracked area causing the cracks to propagate.
Clearly, in order to objectively evaluate the true effect of current compaction methods on the problem at hand, a

comparison has to be made with crack free pavement. A new roller, Asphalt Multi-Integrated roller (AMIR), Figure 1, was designed and built to prevent construction induced cracks. Details of the AMIR roller are given in a number of reports (7 to 9) and also presented in a paper by Selvadurai and Svec in this workshop.

This study will be the first attempt to investigate asphalt stripping in the context of compaction comparing between the products of both conventional rollers and the new compactor, AMIR.

DEVELOPMENT OF A FRAMEWORK FOR EVALUATION OF STRIPPING

The development of the AMIR compactor based on sound theoretical analysis made the opportunity to examine the deficiency of the current compactors a viable option. Also as it was pointed out earlier, compaction was considered to have no significant effect on the problem of asphalt stripping. This observation stands to reason since the pavement researchers have only one alternative i.e., current drum rollers. Therefore , if asphalt cores are recovered from the field to evaluate the effect of compaction on a given problem such as stripping, the finished asphalt pavement will always have the same cracking characteristics. Since these cracks can not be eliminated with current equipment, their influence will be common across most of the investigated projects and as a result will mask the actual influence of the cracks. On the other hand, when samples are prepared in the laboratories to simulate field conditions, the construction induced cracks observed in the field may not be present. Clearly, one should expect poor correlation between test results performed on laboratory prepared samples and the ones recovered from the field.

In order to evaluate the influence of the compaction method on the resistance of a given mix to moisture induced damage, the following approach was adopted :

1. Develop an **"unbiased test method"** that can induce moisture damage to the mix,

2. Develop an **"unbiased criterion for comparison"** to evaluate the effect of compaction on the amount of damage induced in 1 above, and

3. Define an **"acceptable minimum level of significance"** to assess the achieved improvement.

The development of this approach is presented in the following sections.

Development of Unbiased Test Method

In a recent study carried out to evaluate currently employed laboratory test methods to assess stripping potential of asphalt mixes (10), the authors of the study concluded the following :
" Stress pedestal, boil and indirect tensile tests were evaluated for assessing the stripping potential of asphalt concrete mixtures. The tests were applied to surface and base-binder mixtures that included five aggregate combinations, asphalt cement from two sources, and three anti stripping agents. The boil and indirect tensile tests were most promising although neither they nor the stress pedestal test accurately predict the expected performance of all mixes. To this they added that " A pass - fail criterion, according to which all reported moisture - susceptible mixtures fail and all reported moisture resistant mixtures pass, could not be developed for any of the tests evaluated".

It is obvious from these reported remarks that a great deal of uncertainty exists as to which test method should be adopted. Also, this difficulty is coupled with the problem of correlating the results of laboratory prepared test samples with results obtained from testing field samples. For example, it is common to find the results of the laboratory samples suggesting a none stripping mix while the field observations suggest that the same mix was experiencing stripping in the field.

Therefore, in order to evaluate the effect of compaction on the resistance of a given mix to stripping, it is important to adopt a test method that will induce moisture damage regardless of whether or not stripping will occur in the field. To achieve this objective a parametric laboratory investigation was carried out as follows :

1. Identify the most significant parameters causing **"Laboratory Induced Moisture Damage"**. The parameters included in this step were degree of vacuum saturation (14 Hg vs 24 Hg), freeze thaw cycles (three vs none), thawing temperature (68 F.vs 140 F.) and control conditions i.e. tested under room temperature without any conditioning. Thus, five different set of test conditions were investigated as shown in Table 1.

The indirect tensile strength test method was adopted to measure the effect of each tested parameter.

2. Evaluate the effect of each tested parameter and establish an **"Influence Index"** representing the contribution of the rated parameter to the measured stripping. The result of this step should provide a relationship in the form of a "curve" . The characteristics of the resulting "curve" should provide valuable

information concerning the resistance of a given mix to moisture damage.

3. When objectives 1 and 2 above are achieved, the influence of compaction method can be evaluated by comparing between the "Performance Curves" produced by different compactors.

To explain the significance of this approach on the problem at hand, let us assume that a given asphalt mix showed perfect resistance to moisture induced damage. The "performance curve" for such a mix will be similar to the curve shown in Figure 2.

It should be noted that for actual asphalt mixes there will be some reduction in the measured strength due to the effect of the tested parameters. Accordingly, the "Performance Curve" shown in Figure 2 should be considered the maximum or upper envelop to all possible asphalt mixtures. Clearly, a more realistic asphalt mix will have a performance curve similar to the curves of mix "a" or mix "b" as given in Figure 2 . However, while both mixes would be considered as susceptible to stripping according to the ratio between the indirect tensile strength of the conditioned sample to the strength of the control sample, in both cases less than 60%, this conclusion is misleading. The performance curve for mix "a" in figure 2 suggests a mix with normal behavior with regard to the effect of the tested parameters . On the other hand, the performance curve of mix "b" indicated faster deterioration in the relative strength of the mix as a result of the applied test parameters. The deterioration observed on the lower part of the first mix can be explained as "test characteristics" while the one noted on the second mix is a "mix characteristics". Therefore, one should select mix "a" over mix "b" when both are evaluated for stripping potential. A positive shift in a given curve to the right or in the vertical direction would indicate an improved mix.

The analysis discussed above suggests that the current test methods that are based on the ratio between the strengths of the conditioned sample and control dry sample can not be considered reliable. Therefore, part of the data generated for this research was utilized to examine this approach.

Development of Unbiased Criterion for Comparison

While the proposed test method presented above can be used for the purpose of comparison as well as evaluation, the authors adopted a comparison criterion developed by Lottman (11). A set of ratios were used to compare between the stripping potential of two asphalt mixes, a control and a treated mix. It was suggested that in case of comparing between two or more mixes, then the treated mix must be

evaluated by the following method :

CTSR = Treated Wet ITS / Control Dry ITS,

M.E. = Treated Dry ITS / Control Dry ITS, and

RTSR = Control Wet ITS / Control Dry ITS

where,

CTSR: Combined tensile strength ratio,
M.E.: Modifier effect on the tensile strength,
TSR : Tensile Strength Ratio, and
ITS : Indirect Tensile Strength
RTSR: Retained Tensile Strength of Control Cores

The first two ratios were then compared to the retained strength of the control specimens (RTSR) and unity respectively. Thus, for accepting the treated mix and rejecting the control mix the following conditions must apply :

CTSR > RTSR , and
M.E. > 1

In the case of comparing the effect of compaction method on the stripping potential the following criterion is adopted in this report :

1. Consider the "Control Mix" is the one compacted by Current compaction methods.
2. A "Reference" strength for all comparisons will be the dry strength of the control compaction method.
3. Consider the "Treated Mix" is the mix compacted by the AMIR method.
4. Add a third ratio to the two introduced by Lottman as follows :

S.I. = Treated Wet ITS / Control Wet ITS

where,

S.I.: Stripping Improvement.

Thus in order to accept the AMIR compacted mix as a better choice than the Control compacted mix in terms of higher resistance to moisture induced damage, the following three conditions must be satisfied :

1. CTSR > RTSR,

2. M.E. > 1 , and

3. S.I. > 1

Failure to meet any one or more of the above three conditions will result in rejecting the new compaction method as an improvement over the control i.e. current steel compaction techniques.

Acceptable Minimum Level of Significance

The difficulty with determining an acceptable level of significance arises from the fact that there is always a cost element associated with improving a given situation. For example if an additive is claimed to improve the resistance of pavements to stripping by 10 %, the significance of this level of improvement can only be established when the cost of the additive is shown to be smaller than the benefits obtained by its use. In the case of the AMIR roller this problem does not exist. The improvement obtained through higher densities, higher tensile strength and expected longer service life is in fact achieved at lower cost than the cost of using the existing equipment. These analysis are based on the assumption of the availability of a commercial AMIR compactor. Thus, any improvement provided by the new compaction method would be rated as significant since the new compactor would not result in any additional cost to existing cost of construction. For more practical purposes, any improvement in the range of 10% or more will be considered as significant .

OUTLINE OF FIELD TESTS

Early laboratory test results (12) suggested improvement in stripping resistance when AMIR roller was used. These results proved the need for a full scale experimental investigation using samples from field compacted pavements.

To outline the research investigation methodology and to design field trials necessary for the study,the variables involved will be defined first. Many of the factors contributing to stripping have been investigated and their contribution to stripping is relatively known with the exception of construction. Haas (2) considered construction methods by which water is trapped within the pavement as one of the unforeseen circumstances that contribute to stripping. To clarify some of these unforeseen factors,this study will limit the number of variables to those associated with construction techniques in isolation of other factors such as those associated with mix characteristics. The use of different mixtures in different locations is expected to assist future studies to incorporate the outcome of this study in an overall assessment of the problem. The study is expected to yield general and consistent results applicable to all locations and for different mixtures. Therefore,field trials were designed to investigate the following factors:-

1- Number of roller passes
2- Type of asphalt mixture
3- Case of overlay versus asphalt on top of base coarse
4- Case of one lift versus two lifts

5- Use of static steel or Vibratory roller versus AMIR

To investigate the above mentioned factors,two field trials were constructed as outlined below:

Toronto Test:

The first field test was carried out in Toronto last June. The test was designed to compare between the quality of compaction of the AMIR compactor and the conventional static steel roller. The total weight of AMIR was slightly less than that of the steel roller. Thus, the pressure under AMIR was much less than under the steel roller. The section was selected on the unpaved parking lot at the site of the Lovat Tunneling property. An MTO standard HL-3 asphalt mix was employed in this test. Each roller made three passes on its designated section. After completion of the test photos and slides were taken for the finished surface. One week latter, a total of 104 asphalt cores were recovered from the test sections for further laboratory evaluation.

Ottawa Test:

The success of the Toronto field test lead to the planning and construction of four more field tests in Ottawa. For the purpose of this paper, asphalt cores recovered from the first of the four field trials are reported . The first Ottawa test was carried out to compare the quality of AMIR compaction to the one obtained by the combination of a steel vibratory and rubber wheeled rollers. The test section was an old paved access road on the NRC campus in Ottawa. A standard MTO HL-4 asphalt mix was employed in the test. The main objective of this test trial was to evaluate the ability of each compaction method in achieving the minimum density as defined by the MTO specifications. The AMIR compactor started its operation fully loaded. As a result some steering problems were encountered in the early stages of the trial. This difficulty resulted in dividing the finished section as follows:

1. Block 0, represents the part of compacted asphalt when the AMIR roller was fully loaded.

2. Block 1 , represents the part of compacted asphalt which remained uncompacted by AMIR for sometime due to longer waiting time needed to empty the AMIR compactor.

3. Block 2, represents the portion of the compacted asphalt that was laid immediately following Block 1.

4. Block 3, represents the last truck load compacted by the AMIR compactor at the final stage of the field test.

In this research asphalt cores taken from Blocks 0, 2 and 3 were tested. The first Ottawa test was carried out August 1989 and actual laboratory testing started on cores taken from Block 3 followed by cores from Block 2 while cores from Block 0 were tested last. Therefore, the curing time effect should be considered in terms of number of Block as follows:

1. Block 3 : after 4 weeks for dry and conditioned tests.
2. Block 2 : after 6 weeks for dry tests.
3. Block 0 : after 8 weeks for dry tests.

Laboratory Testing and Conditioning Procedure

a) Bulk Specific Gravity
Asphalt cores recovered from the two field tests were transported to the Civil Engineering Laboratory at Carleton University. Bulk specific gravity tests were performed according to **ASTM D2726-73(79)** using Standard Surface -Dry specimens.

b) Vacuum Saturation and Permeable Voids
A test method developed by Lottman (13) and adopted by Transport Canada was used in this investigation. The test method is given below:

1. Selected dry asphalt cores were placed in a deep water vacuum saturation jar. The jar was then filled with distilled water to a hight of one inch above the top of the core.

2. Vacuum saturation was achieved by the application of the desired pressure, 14 or 24 inch of mercury vacuum , to the cores submerged in the distilled water at 75 F for a duration of 30 minutes.

3. After the 30 minutes, the vacuum was removed and the inside of the jar was allowed to reach ambient atmospheric pressure. The cores were kept submerged in the jar under atmospheric pressure for another 30 minutes.

The permeable voids for the cores were calculated only when 24 " Hg was applied, using the following formula:

$$\text{Permeable Void } \% = 100\{(w'_L - w_L) + (w'_a - w_a)\}/(w'_a - w_L)$$

where,

w_L : submerged weight of initially desiccated dry core section immersed in distilled water at 75 F for 3 minutes,

w_a : weight in air of desiccated (dry) core section

w'_L : submerged weight of vacuum saturated core section in distilled water at 75 F, and

w'_a : weight in air (saturated surface dry) of core section after obtaining weight w_L

c) Degree of Saturation

A review of the literature indicated that vacuum saturation of asphalt samples is a rather simple and effective means of filling the voids with water. Subsequently, air void content has been used to determine the potential intensity of exposure to water.

Presently, the level of overall saturation of a pavement mix in its later life can not be determined. However, in this study, asphalt cores from different compaction methods were exposed to the same level of vacuum saturation as outlined above. Thus, the objectives stated in the NCHRP 192 and 246 can be realized. It has been suggested in these reports that the objective of the vacuum saturation of asphalt cores is not only to put water into the specimen voids, but also to recognize that it is a damage event. Therefore, in order to avoid any excessive damage to the samples, the degree of saturation adopted in this investigation was limited to a range of 50 to 80%.

d) Indirect Tensile Strength Test

In order to assess the influence of compaction method on the stripping resistance of the asphalt mix the standard indirect tensile strength test method was adopted. The method of calculating the strength is the same used by Transport Canada.

It should be noted that all the asphalt cores taken from the field were marked according to the direction of rolling. The indirect tensile strength tests reported in this investigation were performed with the load applied perpendicular to the rolling direction.

e) Core Moisture Conditioning

The first step in the moisture conditioning of the core samples is vacuum saturation which was discussed before. However, there is no agreement in the literature on the additional conditioning required to develop stripping (14). As presented earlier, to overcome this problem, the conditioning of the asphalt cores was considered a variable in this investigation. Five test conditions were adopted as follows :

1) Vacuum saturated at 14 inch Hg, 3 Freeze/ Thaw cycles soaking. Temperature in the water bath was 140 F ,
2) Same as condition 1 except vacuum saturation used was 24 inch Hg,

3) Vacuum saturation followed by soaking in a 140 F water bath,
4) Same as condition 1 except temperature was 68 F,
5) Same as condition 4 except vacuum used was 24 inch Hg.

In addition to the above five different set of test conditions, indirect tensile strength test was performed on dry cores to establish the "Reference" strength. All wet indirect tensile strength tests were performed at 75 F after two hours of soaking at the test temperature.

TEST RESULTS

Influence of Conditioning

Eighteen asphalt cores from Block 3 of the control section of NRC first field test were used to establish the influence of each of the conditioning cases on the measured tensile strength. The results for this step are given in TABLE 2 and Figure 3.

The results given in the table support the following analysis :

1- The "Reference" strength is the one given by the results of the dry test performed on the control compacted asphalt cores. The measured strength was 94.4 psi.

2. While four of the five conditioning cases included 3 freeze/thaw cycles, the results indicated that significant damage was caused when the asphalt cores were exposed to the higher range of temperature i.e. 140 F. This result is further supported by the results obtained from the case of exposing the cores to the 140 F without any freeze/thaw cycles. This would suggest that while severe stripping was taking place at 140 F, under the Canadian field conditions occurrence of such severe stripping of the same mix is unlikely.

3. Freeze/thaw cycles in the absence of the higher temperature seems to have no significant effect on the problem of stripping as shown by the retained strength of case 2 in table 2 , 97% . In fact some of the test results suggest that improvement may be achieved under the freeze/thaw cycles.

4. The effect of increasing the pressure during the vacuum saturation process was less evident when severe damage was observed i.e. under 140 F conditioning (3% difference due to change of the pressure). On the other hand, the effect of the application of higher pressure on the tensile strength of the tested cores is more evident at the lower temperature level i.e. 68 F (a difference of 20%). Clearly, the influence of soaking at 140 F on the measured

strength dominated the effect contributed to the overall damage induced by the other tested variables .

The normalized damage values shown in the last column of table 2 were obtained by dividing the loss in the measured tensile strength due to a given conditioning by the maximum loss obtained i.e. the loss experienced by cores tested under 24 Hg,3 F/T and 140 F. The calculated values were rounded off to the closest 5%. These damage indicators will be used for further analysis of AMIR as well as control samples. Figure 3 shows the relationship between the strength ratios and the calculated normalized damage given in Table 2 . The performance curve shown in the figure suggests a mix with high degree of sensitivity to stripping. The curve does not show "Test Characteristics" at the high level of damage.

The developed values of expected damage (normalized values of damage) of the control section were used to establish the effect of compaction on the resistance of the mix to stripping. Two more sets (one set from AMIR compacted section while the other set from the control section) of 18 asphalt cores taken from Block 2 of the first Ottawa field test were tested. The results of this test are given in Table 3 and Figure 4 . These results suggest the following:

1. The second set of asphalt cores taken from Block 2 of the control test section confirmed the results obtained from Block 3. The performance curve suggests a sensitive mix to stripping. Again it should be noted that this mix may not show the same degree of stripping in the field under Canadian climate. Also, it was observed that the conditioning case of 14 Hg, 3 F/T cycles and 68 F resulted in an increase in the relative tensile strength of the tested mix. This result would suggest that freeze / thaw cycles may result in an improved bond of the asphalt cores tested in the Laboratory .

2. The performance curve of AMIR compacted asphalt cores suggests a different mix with improved stripping resistance. The effect of the different conditioning cases on the stripping resistance of the AMIR samples indicates a "Test Characteristics" i.e. forced stripping due to the effect of water soaking at the 140 F.

3. Regardless of whether stripping will occur in the field or not, the results of this test showed that stripping is induced in the laboratory under the 140 F moisture conditioning.

4. Clearly the improvement shown by the AMIR compacted method can be explained by the elimination of the surface cracks. The elimination of the construction induced cracks resulted in an increase of the overall resistance of the

same asphalt mix by more than 30 %.

In summary, the results presented in this section indicated that in order to predict the sensitivity of a given mix to stripping one has to establish a "performance curve" which describes the effect of the different parameters on the relative strength of the mix.

Evaluation of Compaction Method

Toronto Test Results

Indirect tensile strength tests were carried out on asphalt cores taken from the Toronto field test to examine the following:

1. Effect of curing time on the tensile strength of cores compacted by AMIR and Conventional methods.

2. Effect of compaction method on the stripping resistance of a standard HL-3 asphalt mix.

Table 4 gives the test results for evaluation of the curing time. The results in the table suggest the following observations:

1. In general, the indirect tensile strength of newly compacted asphalt pavements would increase with time until it reaches a maximum value. As can be noted from the tables, the control compacted asphalt cores gained 20% and 30% increase after 3 weeks and 3 months from the date of compaction. The same phenomenon is noted in the case of the AMIR compacted cores. These cores showed an increase of 11% and 23% after 3 weeks and 3 months respectively.

These test results would suggest that while both asphalt cores will gain more strength with time, the same results would also indicate that the AMIR compacted asphalt mix is more stable in comparison to the control compacted cores. The measured change in strength over time is clearly less sensitive in the case of the new compaction method. This observation is coupled with the fact that the new compaction method provides higher tensile strength values at all times as can be seen in the tables.

2. The AMIR compacted asphalt cores were 31%, 21% and 23% higher than the control compacted cores after 1, 3 weeks and 3 months respectively. This increase in strength was achieved at no extra cost nor any modification to the asphalt mix.

3. The advantage of having a more stable material over the early period of time following construction is very significant. It has been reported by many researchers that

many of the problems of new asphalt overlays were observed at relatively early stage of the pavement life. Thus, one should expect that with higher strength and more stability, the new compaction method can provide the needed protection when the pavement is opened to traffic.

The influence of compaction on the resistance of the mix to stripping was evaluated using the developed criterion as follows:

RTSR	=	35.4/ 70.6	=	50 %		
CTSR	=	43.8/ 70.6	=	62 %	>	50 %
M.E.	=	86.3/ 70.6	=	122%	>	100%
S.I.	=	43.8/ 35.4	=	124%	>	100%

Since the new compacted samples passed the three conditions, one can conclude that the mix compacted by the AMIR roller would have higher resistance to moisture induced damage than the control compacted mix. Also, while the retained tensile strength of the steel compacted method is only 50 % suggesting severe damage, the relative strength of the AMIR compacted cores suggests no serious damage as indicated by its 62 % retained strength.

Ottawa Test Results

While the Toronto test was carried out to compare between the AMIR and the standard static steel roller under the same number of passes, the Ottawa field test was performed to compare between AMIR and the Vibratory/rubber wheeled rollers. Also, the Toronto test was carried out on HL-3 mix while the Ottawa test was performed on HL-4 asphalt mix. The test results of the Ottawa field test are given in Table 5. These results support the following analysis:

1. Similar to the results obtained from the Toronto test, the AMIR compacted method provides pavements with higher strength. The measured tensile strength for AMIR compacted samples were 9 %, 11 % and 16 % higher for Blocks 0, 2 and 3 respectively.

2. The variation in strength between the Blocks can be explained by two factors. First, at Block 0 the AMIR roller was fully loaded and difficult to steer. This difficulty resulted in the AMIR roller making few passes over the control section. Second, the tests were performed on cores from Block 3 then Block 2 and finally Block 0. Thus, the variation between Blocks can be attributed to these two parameters.

Table 6 gives the average specific gravity and strength values for each block according to the order of testing.

The analysis of the results given in the table agree with the observations reported on the results from the Toronto

tests. The application of the stripping criterion gives the following results :

RTSR = 44.3 / 88.9 = 49.8 %
CTSR = 54.9 / 88.9 = 61.8 % > 49.8%
M.E. = 103.5 / 88.9 = 116.5% > 100%
S.I. = 54.9 / 44.3 = 124.0% > 100%

The evaluation criterion suggests that the Vibratory/rubber compacted HL-4 mix would have lost more than 50% of its tensile strength due to stripping, the AMIR compacted method resulted in improving the mix strength by more than 20 % . Also, similar to the results obtained from the Toronto tests the AMIR compaction method would result in improving the strength of the dry cores by more than 15 % .

The results of the Toronto and Ottawa field tests suggest the following :

1. The AMIR compaction method would provide better mechanical properties than either a static steel or the combination of vibratory and rubber wheeled rollers.

2. The AMIR compactor produced asphalt pavements with higher and more uniform densities than any existing compaction method. The failure of the existing roller to provide similar results is due to the design limitation associated with the geometry of the loading device i.e. the cylindrical shape.

3. The results obtained from two different asphalt mixes at two different cities suggests that the obtained improvements are due to the compaction method. In these two field tests, other factors such as temperature of the mix, operator's experience, subgrade conditions , etc. should be considered as factors favoring the conventional equipment.

3. Since the main difference between the AMIR compacted pavements and the Control compaction method is the construction induced cracks (in the case of the Conventional compactors), one can conclude that the presence of these surface cracks plays a major role in allowing water to penetrate through the asphalt surface . This water penetration would result in the initiation of the stripping process. Also, under cold climate the presence of the water in the construction cracks can cause stress concentration at the tip of the cracks. These local stresses when combined with both the thermal and traffic induced stresses can lead to the usually unexplained fast deterioration of new asphalt overlays.

CONCLUSIONS AND RECOMMENDATIONS

The results presented in this report provide a rare opportunity to evaluate the effect of different compaction methods on the overall tensile strength of the finished product. It has been shown that the use of the AMIR compactor on the same asphalt mix would result in increasing the strength of the dry core samples as well the conditioned cores. The effect of stripping was shown to be significantly reduced when the surface cracks were minimized or eliminated. In summary the results support the following conclusions:

1. The use of the steel drum compactors would induce surface cracks at time of compaction. These cracks would result in allowing the water to penetrate through the asphalt pavement resulting in the known phenomenon of asphalt stripping.

2. In the absence of high temperature the effect of freeze/thaw cycles on the problem of stripping is not conclusive.

3. The development of the "Performance Curve" represents an important tool to fully understand the interaction between the properties of the compacted mix and the different conditioning parameters.

4. More research work is needed to develop a reliable mechanistic approach to predict stripping of asphalt pavement in the field. The recommended approach will require further testing of laboratory prepared asphalt cores as will as cores recovered from the field.

Finally, the main objective of this study was achieved. Compaction method influence both physical and mechanical properties of compacted mixes. It has been shown that the new AMIR compactor can indeed provide asphalt pavements with superior mechanical properties than any existing compaction method. Also, the test results presented in this report showed that the resistance of a given mix to moisture damage is very much dependent on the compaction method. Furthermore, the elimination of surface cracks during construction can result in significant improvement of the resistance of the asphalt to moisture induced damage.

ACKNOWLEDGMENT

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FIGURE 1 : ASPHALT MMULTI-INTEGRATED ROLLER (AMIR)



FIGURE 2: EXPECTED PERFORMANCE CURVES OF ASPHALT MIXES

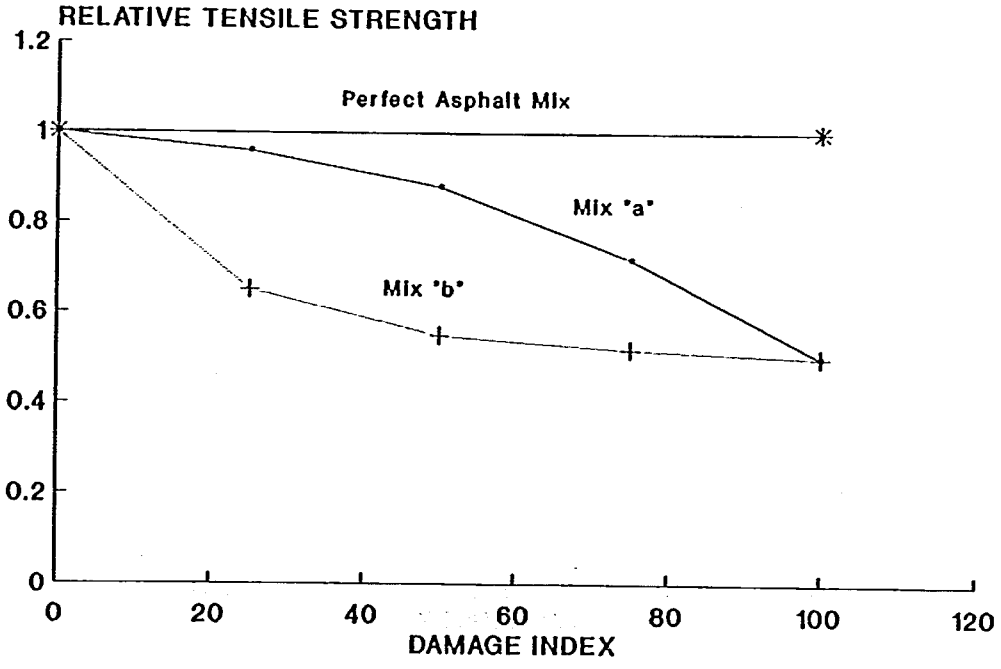


Figure 3: ESTABLISHED PERFORMANCE CURVE FOR OTTAWA TEST SECTION

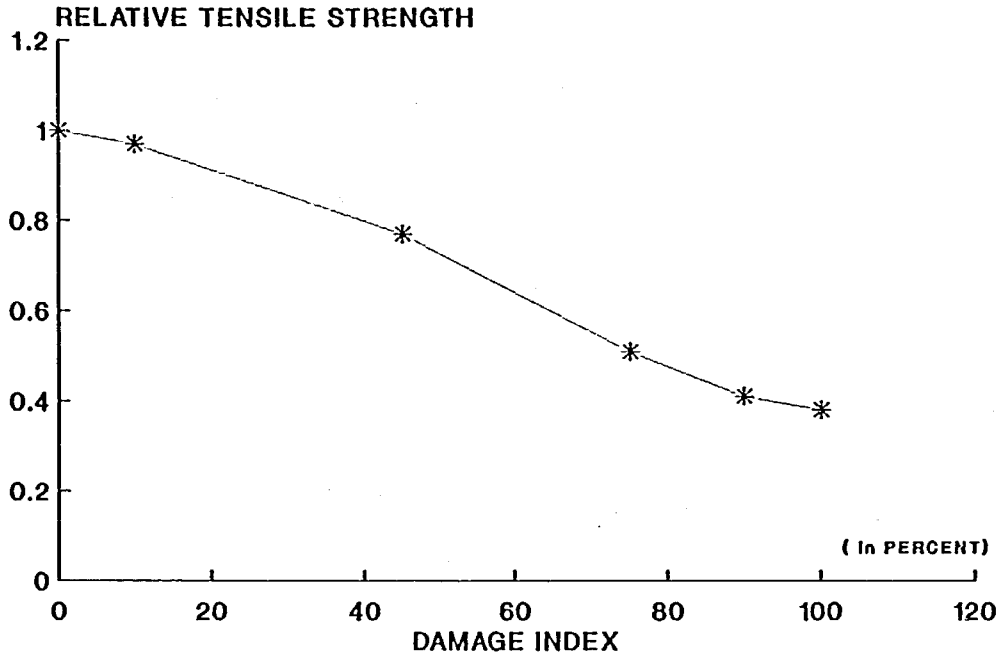


Figure 4: EFFECT OF COMPACTION ON RESISTANCE TO STRIPPING

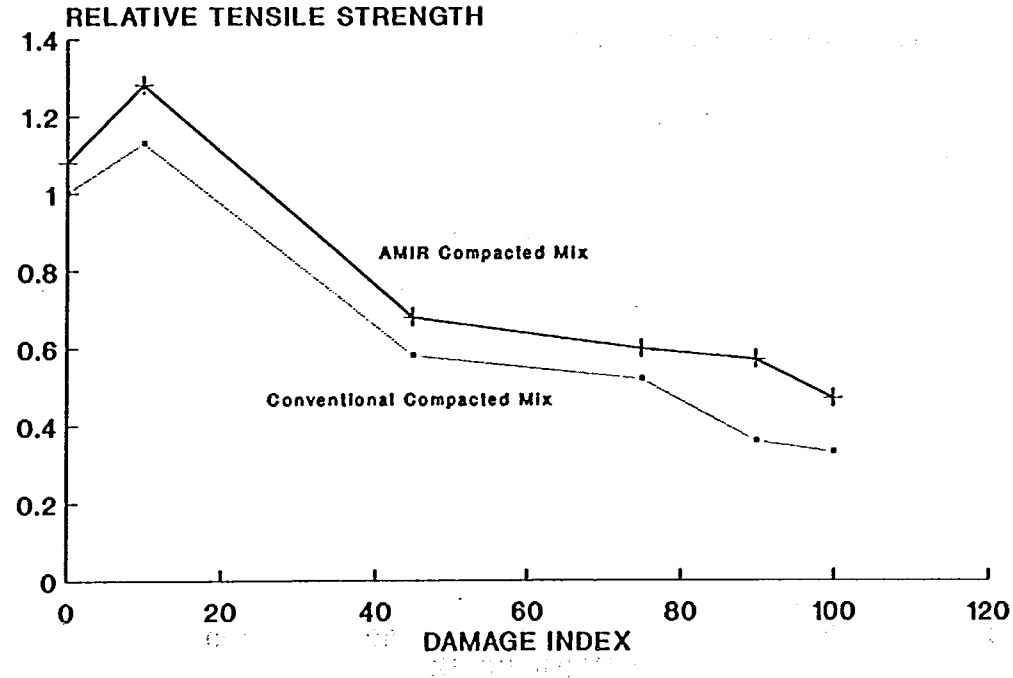


TABLE 1: Details of Test Conditioning

Condition #	Vacuum Saturation Pressure	Three Freeze Thaw Cycles	Thawing Temperature
1	14" Hg	yes	140 F
2	24" Hg	yes	140 F
3	24" Hg	No	140 F
4	14" Hg	yes	68 F
5	24" Hg	yes	68 F

TABLE 2 : Influence of Conditioning Procedures on Moisture Induced Damage

(Average of 3 cores)

Conditioning	Measured Strength in (psi)	Ratio	Normalized Damage
Reference	94.4	100	0
14 Hg, F/T, 75 F	91.4	97	10
24 Hg, F/T, 75 F	72.4	77	45
24 Hg, N/A, 140 F	48.6	51	75
14 Hg, F/T, 140 F	38.9	41	90
24 Hg, F/T, 140 F	35.7	38	100

TABLE 3 : Effect of Compaction on Stripping
(Average of 3 cores)

Conditioning	Measured Strength (in psi)		Ratio %	
	Control	AMIR	Control	AMIR
Reference	121.7	132.0	100	108
14 Hg, F/T, 68 F	137.0	155.6	113	128
14 Hg, F/T, 68 F	71.1	83.0	58	68
24 Hg, N/A, 140 F	62.7	73.6	52	60
14 Hg, F/T, 140 F	44.1	69.6	36	57
24 Hg, F/T, 140 F	40.3	57.1	33	47

TABLE 4: Test Results of Asphalt Cores from Toronto Field Trial
Average of 6 core samples

Time	Specific Gravity		Indirect Tensile Strength	
	AMIR	Steel	AMIR	Steel
One Week/Dry	2.351	2.331	70.6	54.1
St. Dv.	0.015	0.025	3.8	4.4
Marshall	96.2 %	95.4 %		
Three Week	2.344	2.324	78.5	65.9
St. Dv.	0.022	0.037	3.4	5.8
Marshall	95.9 %	95.1 %		
Three Month	2.352	2.346	86.4	70.6
St. Dv.	0.011	0.036	2.8	6.4
Marshall	96.2 %	96.0 %		
Conditioned	2.327	2.311	43.8	35.4
St. Dv.	0.025	0.042	2.9	4.6
Marshall	95.2 %	94.6 %		

TABLE 5 : Test Results According to Curing Time

Curing Time	Tensile Strength (*) AMIR	Control	Ratio
2 Months(Block 3)	103.5	88.9	116 %
3 Months(Block 2)	113.8	102.8	111 %
4 Months(Block 0)	149.1	136.9	109 %

(*) Control : Combination of Vibratory / Rubber wheel rollers

TABLE 6: Test Results of Asphalt Cores from Ottawa Field Trial
Average of 6 core samples

Block #	Specific Gravity		Indirect Tensile Strength	
	AMIR	Control	AMIR	Control
Block 3	2.426	2.415	103.5	88.9
St. Dv.	0.010	0.024	8.8	9.7
Marshall	96.5 %	96.1 %		
Block 2	2.446	2.422	113.8	102.5
St. Dv.	0.018	0.018	2.9	11.4
Marshall	97.3 %	96.4 %		
Block 0	2.440	2.404	149.1	136.9
St. Dv.	0.086	0.013	9.3	18.8
Marshall	97.1 %	95.7 %		
Conditioned	2.431	2.397	54.9	44.3
St. Dv.	0.007	0.030	5.4	3.5
Marshall	96.7 %	95.4 %		

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Applicability of RCCP in Cold Areas

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Applicability of RCCP in Cold Areas

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ABSTRACT

As abrasion of pavement surfaces in cold areas due to the use of tire chains and studded tires in winter time is considerable, improvement of pavement durability is an important role of road maintenance management. The Hokuriku Regional Construction Bureau has given special attention to cement concrete pavements of high abrasion-resistance. And, Roller Compacted Concrete Pavement (RCCP) was selected for trials in 1988 and 1989 to study its applicability in cold regions, because of its higher daily production and earlier opening to traffic than conventional concrete pavement. The main purposes of these trials were to evaluate RCCP as an abrasion-resistant pavement in cold areas and to evaluate the durability of RCCP under heavy traffic.

The items studied were 1) mix design of RCC, 2) mix design and freezing and thawing, 3) mix design and surface characteristics, 4) slab thickness and density, 5) slab thickness and surface evenness, 6) slab thickness and joint spacing, 7) density and flexural strength, 8) construction time of contraction joint and 9) time period before opening to traffic.

This paper was prepared as basic data to find the indices of applicability of RCCP in cold areas, and as a basic document to establish Technical Guidelines for RCCP.

Applicability of RCCP in Cold Areas

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1/ INTRODUCTION

The annual abrasion value of asphalt pavement according to the Hokuriku Regional Construction Bureau, which is located in cold area, is as much as 5 mm due to the use of tire chains and studded tires. Consequently maintenance costs including overlays are increasing annually.
In this Region, almost all paving consists of asphalt pavement, but cement concrete pavement has been applied in tunnels, under snow sheds and in limited bypass sections. Generally conventional cement concrete pavement is difficult to apply on existing roads already open to traffic because of high initial construction costs and the long curing time required before opening the road to traffic. Therefore, Roller Compacted Concrete Pavement (RCCP), which recently has been attracting attention, was selected for trials to study its applicability in cold areas.

Though the draft of Technical Guidelines for RCCP, which will be issued by the Japan Road Association, specifies that the limitation of RCCP slab thickness shall be 25 cm with high uniform density and acceptable surface evenness, our trials were planned for 28 or 30 cm based on the slab thickness standards shown in the Manual for Cement Concrete Pavement by the Japan Road Association to serve to heavy traffic. To confirm the applicability of RCCP,

the first trial was carried out in 1988 by constructing a 28 cm single-layer slab on National Highway Route 17, and the second trial by constructing a 30 cm a single-layer slab on the Joshin Bypass of National Highway Route 18 in 1989.

This paper describes these trials and the results of evaluation to the present.

2/ DESCRIPTION OF TRIAL SECTIONS

National Highway Route 17 connects Tokyo Metropolitan and Niigata City and National Highway Route 18 connects Takasaki City in Gunma Prefecture and Joetsu City in Niigata Prefecture. Both arterial routes traverse the center of Japan, connecting Pacific Ocean and Japan Sea Coasts.

The trial sections on Route 17 and Route 18 are located in hilly areas and about 90 km apart from Niigata City and about 20 km apart from Joetsu City respectively.

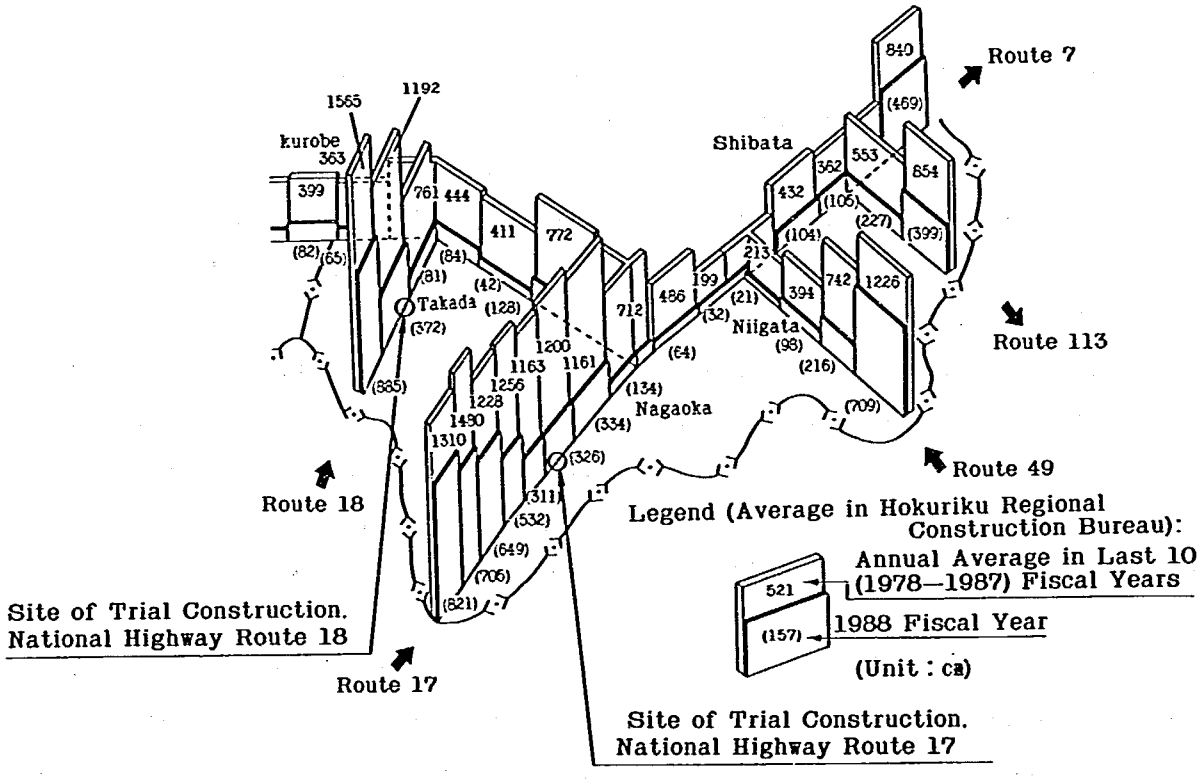


Figure 1 Comparison of the Total Snowfall in 1988 with the Average Annual Snowfall in Last 10 Years

The elevations of these routes are relatively low, 100 m and 170 m respectively. However, both routes are located in extremely heavy snowfall zones of Japan.

The average cumulative snow fall in the last decade was 1,200 cm on Route 17 and 1,198 cm on Route 18 with less snow falling in recent years (Figure 1). The lowest average daily temperatures in the last 4 years (1987-1990) were -2.7 °C and -0.4°C, respectively.

Average traffic volume is 7,800 vehicles/12h including 20.8 % heavy vehicles on Route 17 (as of 1987 end) and 5,180 vehicles/12h including 27 % heavy vehicles on Route 18 (as of March 1990).

3/ TRIAL ON NATIONAL HIGHWAY ROUTE 17

In a project to rehabilitate a 620 m long section of cement concrete pavement, 120 m section of one lane was selected for the RCCP trial.
In spite of the advantages of RCCP, problems remaining to be solved are improvement of surface evenness, obtaining of uniform high density of concrete slab and improving surface textures.
As the originally designed slab thickness of one lane was 28 cm, the same thickness was applied in the RCCP trial. After the careful review of our past records, no examples of 28 cm thickness were found. We selected the single layer construction method to avoid adhesion problems between the two layers when the two layer construction method is used.

3.1/ TRIAL

An outline is shown in Table 1. Plan and typical cross section are shown in Figures 2 and 3, respectively.

3.2/ MIX DESIGN

The fundamental process of mix design was carried out conforming to the Manual for RCCP Trial by the Pavement Committee, Japan Road Association; mix design was established based on the mean value of the maximum densities

Table 1 Outline of the Project

Description of the Construction and Date Constructed	
Length	120m
Width	3.75m
Area	450m²
Construction Thickness	28cm
Date of Trial Construction	Oct. 18, 1988
Date Opened to Traffic	Oct. 24, 1988

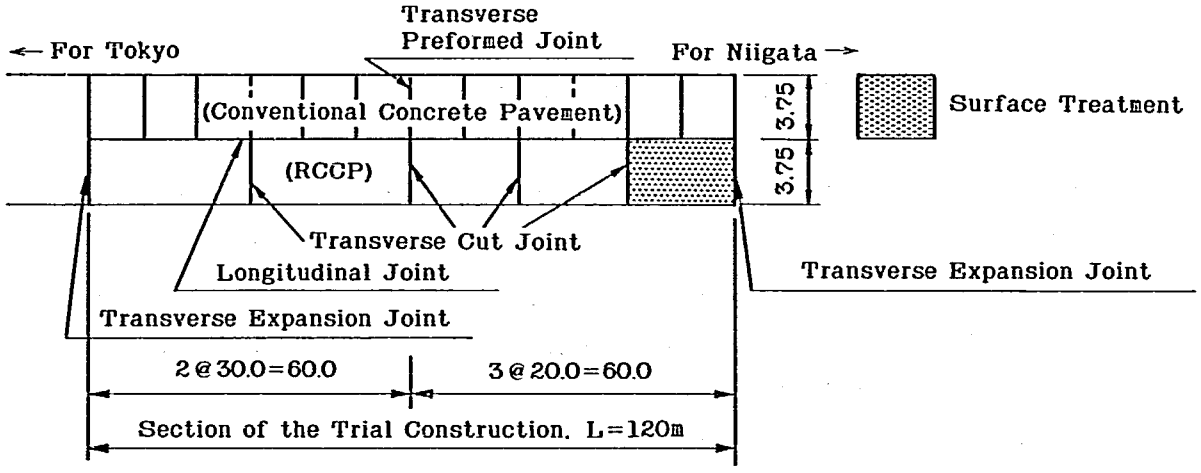


Figure 2 Plan of the Trial Construction

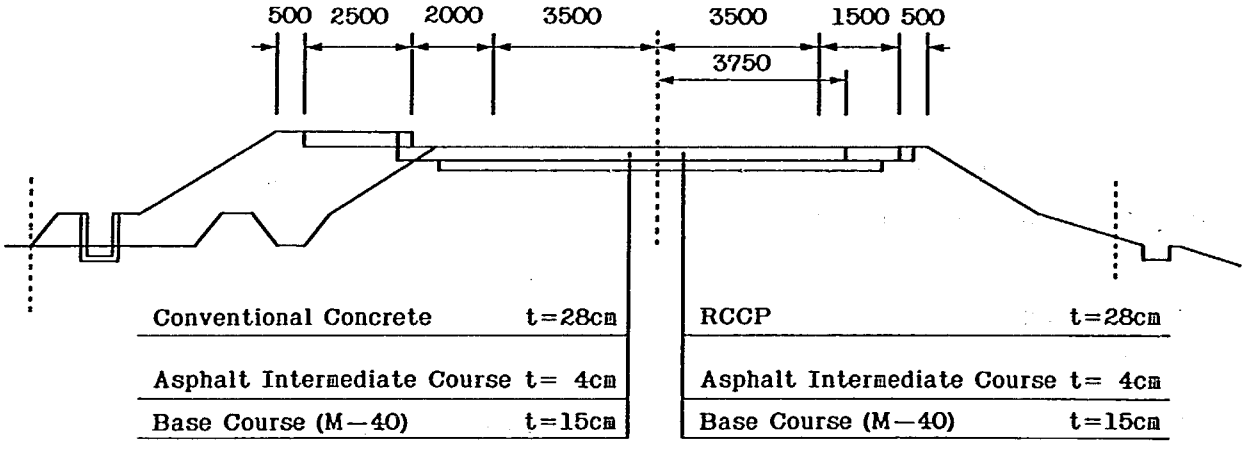


Figure 3 Typical Cross Section

obtained from the results of three tests: VC Consistency Method, CBR Impact Compaction Method and Marshall Test Method.

The final mix design based on the test results is shown in Table 2.

The target strength σ_{br} was selected as 56 kgf/cm², considering an over-design factor of 1.25.

$$\sigma_{br} = \sigma_{bk} \times \gamma$$
$$= 45 \times 1.25$$
$$= 56 \text{ kgf/cm}^2$$

where,

σ_{br} : Target strength

σ_{bk} : Design strength

γ : Over-design factor

Table 2 Mix Proportion

Max. Size of Coarse Aggregate		mm	20
Sand Ratio		%	39
Water Cement Ratio		%	37
Unit Coarse Aggregate Volume			0.85
Unit Weight kg/m³	Water	W	105
	Cement	C	284
	Fine Aggregate	S	813
	Coarse Aggregate	G	1340
	Admixture. A. E.	AE	0.71
Cement Weight Ratio		%	11.8
Water Content		%	5.87
Kp			1.20
Km			1.44

$$Kp = \frac{\text{Volume of Cement Paste}}{\text{Void of Fine Aggregate}}$$

$$Km = \frac{\text{Volume of Mortar}}{\text{Void of Coarse Aggregate}}$$

$$\text{Water Content} = \frac{\text{Total Content of Water in Concrete}}{\text{Total Dry Weight of Cement and Aggregate}}$$

3.3/ CONSTRUCTION

3.3.1/ BATCH PLANT AND HAULING

For RCC mixture production, a batch plant using a double-shaft pugmill mixer was selected. Mixing capacity was 1.5m³ (nominal 2 m³) per batch.

Though a shorter hauling time is preferable, the distance of 10.5 km between plant and site required hauling time of about 20 minutes.

The mixture was hauled in 11 ton dump trucks covered with a plastic sheet, to avoid evaporation of water.

3.3.2/ PLACING AND COMPACTION

Table 3 shows the equipment used and Figure 4 shows the construction process.

Table 3 Main Equipment

Name of Equipment	Type	Size	Manufacturing Company	Application
Asphalt Paver	S—2000		Vögel	Placing
Vibration Roller	SW—70		Sakai Heavy Industry	Initial and Secondary Compaction
Pneumatic Tired Roller	TR—15	15t	ditto	Finish Rolling
Oscillation Roller	N3—R	8t	ditto	ditto
Small-Sized Vibration Roller	BW—65S	650Kg	Bomag	Side Compaction

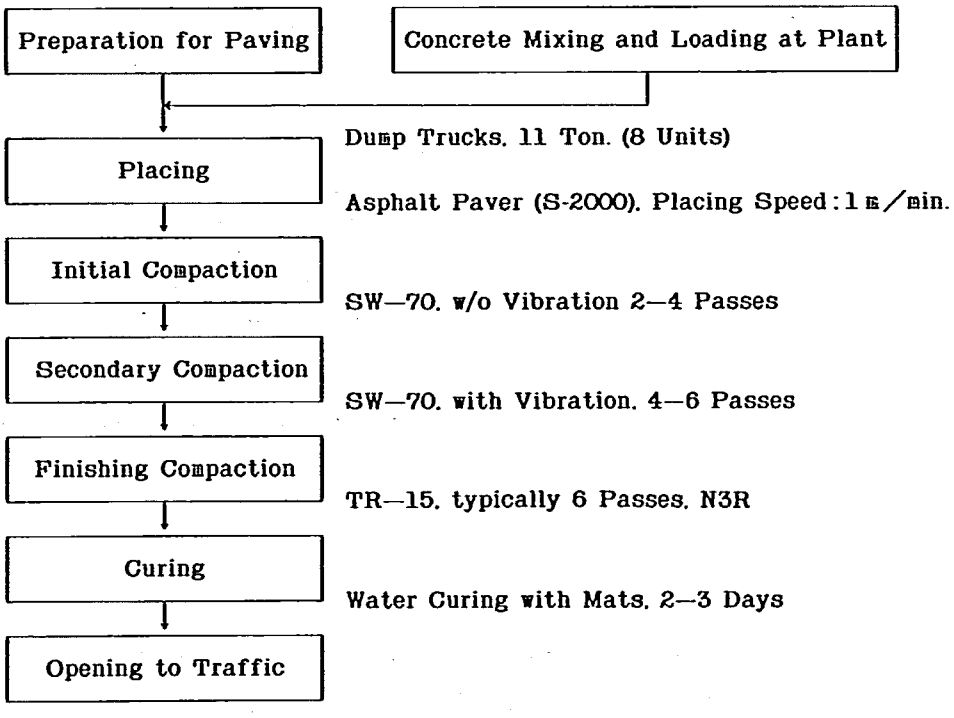


Figure 4 Construction Process

A high-compaction type asphalt paver with normal TV screed was selected for placing of concrete and 15-20 % of compaction allowance was estimated, based on preliminary tests at plant site.

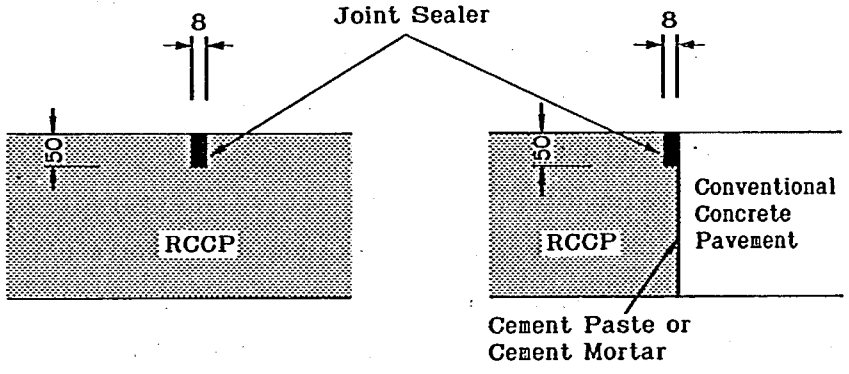
Two to four compactions without vibration (initial compaction) and four to six compactions with vibration (intermediate compaction) were done with a 7

ton vibratory roller. The final compaction (six times) was done with 15 ton pneumatic tire roller. To maintain surface texture, concrete mortar was spread on coarse parts of the surface during the compaction process. To obtain a smoother surface texture, an oscillating roller was used to eliminate hair cracks generated by vibrating compaction.

After completion of compaction, the surface was completely covered with a curing mat for three days and sprinkled with water. It took five days, including shoulder works, until the road was opened to traffic.

3.3.3/ JOINT

Transverse joints were set at 20 or 30 m intervals. Longitudinal and transverse joints were totally saw-cut as shown in Figure 5. Saw cutting was done two days after finishing, and one crack was found. It is desirable to make joints as soon as possible, though the decision on accurate cutting time is difficult.



(a) Transverse Joint (b) Longitudinal Joint

Figure 5 Structure of Joints

3.4/ QUALITY AND WORKMANSHIP

3.4.1/ QUALITY CONTROL AT PLANT

1) Control of consistency

From the results of impact compaction tests, a compaction degree of 96.5 % immediately after mixing and 96.3 % at the site were obtained, so no loss of consistency was found.

2) Strength of RCC

Flexural strength test results are as shown in Table 4. The average flexural

Table 4 Flexural Strength Test Result

Age \ Item / No.		Standard Curing		Field Curing	
		Rate of Compaction (%)	Strength Kgf/cm²	Rate of Compaction (%)	Strength Kgf/cm²
7 Days	1	98.0	57.6	95.9	67.5
	2	98.1	57.0	96.4	61.8
	3	97.4	61.8	96.5	67.2
	Mean	97.8	58.8	96.6	65.5
28 Days	1	97.0	79.8	95.7	78.0
	2	96.3	80.1	96.2	76.2
	3	97.4	78.9	97.0	73.2
	Mean	96.9	79.6	96.3	75.8

strength obtained at 28 days under standard curing conditions was as high as 79.6 kgf/cm², and 65.5 kgf/cm² was obtained at 7 days under site curing conditions. It can be said that the early strength of RCC is remarkable.

3) Abrasion resistance and durability

The maximum allowable value of 9.5 cm² was set as the target value of spike-ravelling tests to evaluate abrasion resistance of asphalt pavement in the Hokuriku Region. An excellent result of 6.0 cm² was obtained from this RCCP trial. For evaluation of durability, a target value of 60 % or more was set for the freezing and thawing index in the freezing and thawing tests. A good result of 80 % was obtained from this trial.

3.4.2/ QUALITY CONTROL AT THE SITE

1) Evaluation of density

For measurement of density, the Radio Isotope Method was used. The average degree of compaction was 97.1 %, and no result less than the target value of 96 % was obtained.

2) Compaction allowance

The average value is 15 % with a range of between 10 and 18 %.

3) Surface evenness by 3 m profilometer

Test results are shown in Table 5. The average value of 3.37 for two lines

is unsatisfactory, leaving the task of obtaining good evenness of RCCP surface for the future. The result may have been caused by a thick lift of 28 cm in one layer and the misselection of screed of paver.

Table 5 Result of Measured Surface Evenness

Measured Point	Outside	Inside	Mean
Standard Deviation	2.60	4.14	3.37

4) Skid resistance

Test results obtained using a portable skid resistance tester are shown in Table 6. As can be seen, all results are satisfactory and these are on the same level as those of conventional cement concrete surfaces.

Table 6 Result of Measured Skid Resistance (BPN)

Type of Construction	Skid Resistance Converted into Values at 20°C					
	1	2	3	4	5	6
RCCP	75.6	82.4	72.0	78.6	82.0	78.1

4/ TRIAL ON NATIONAL HIGHWAY ROUTE 18

In a new 3.4 km long cement concrete pavement construction project on a bypass, a 150 m two-lane RCCP trial section was selected.
This trial was the second one in the region, following the first undertaken in 1988. Based on experience with Route 17, construction of a 30 cm thick single layer slab was tried to confirm the applicability of RCCP for heavy traffic conditions and the remaining problem of surface evenness.

4.1/ THE OUTLINE OF TRIAL

The outline, typical cross section and plan are shown in Table 7 and Figures 6, 7 and 8 respectively.

4.2/ MIX DESIGN

The fundamental mix design was based on the Manual for RCCP Trials as for Route 17. Tests using CBR Impact Compaction Method and VC Consistency Method were conducted.

Table 7 Outline of the Project

Description of the Construction and Date Constructed	
Lenghth	150m
width	9.5m
Area	1425m²
Construction Thickness	30cm
Date of Trial Construction	Jul. 14 and 17, 1989
Date Opened to Traffic	Nov. 1, 1989

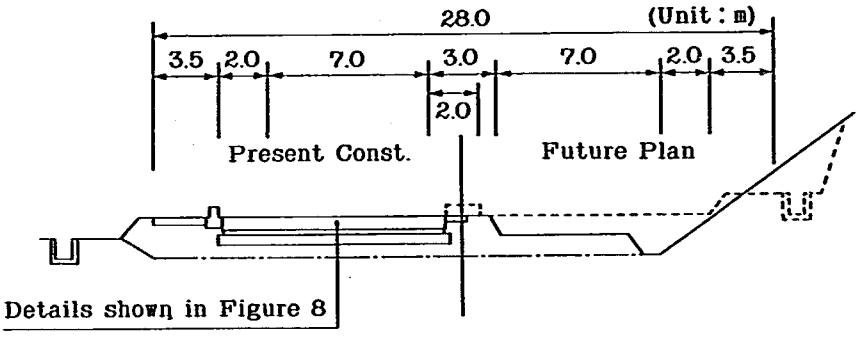


Figure 6 Typical Cross Section

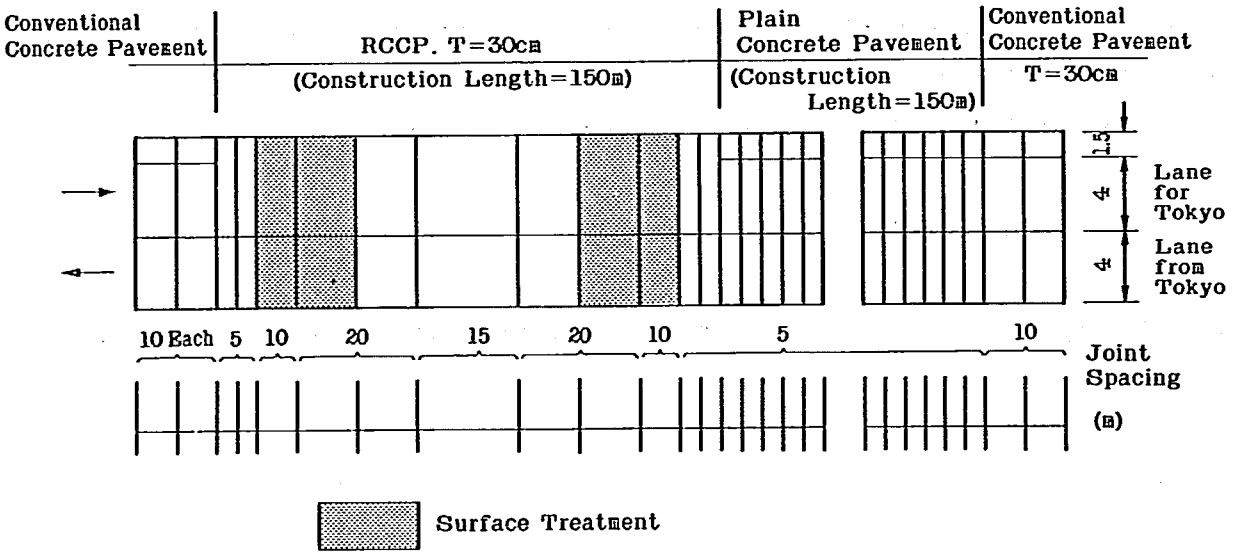


Figure 7 Plan of the Trial Construction

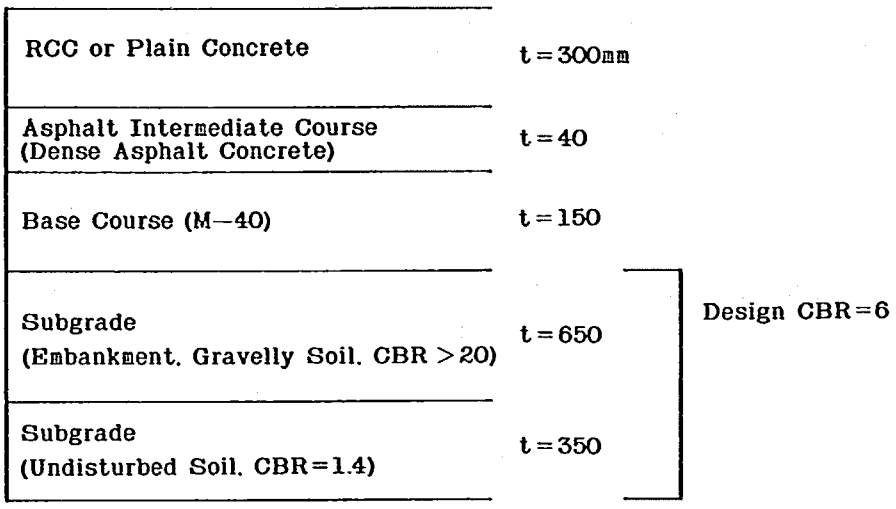


Figure 8 Pavement Structure

The final mix design based on test results was as shown in Table 8. Design flexural strength was 45 kgf/cm², and target strength was 56 kgf/cm², with an over-design factor of 1.25.

Table 8 Mix Proportion

Max. Size of Coarse Aggregate		mm	20
Sand Ratio		%	42
Water Cement Ratio		%	41
Unit Coarse Aggregate Volume			0.82
Unit Weight kg/m³	Water	W	105
	Cement	C	257
	Fine Aggregate	S	886
	Coarse Aggregate	G	1219
	Admixture. A. E.	AE	2.57
Cement Weight Ratio		%	11.1
Water Content		%	6.5
Kp			1.07
Km			1.34

4.3/ CONSTRUCTION

4.3.1/ MIXING PLANT

For mixture production, the nearest ready-mix plant was 20 minutes away. A double-shaft pugmill with a mixing capacity of 1.6-1.7 m³ (nominal 3 m³) was used.

The mixture was hauled by 11 ton dump trucks covered with a plastic sheet to avoid evaporation of water.

4.3.2/ PLACING AND COMPACTION

Table 9 shows a list of equipment used. The construction procedure was the same as that for Route 17.

Table 9 Main Equipment

Name of Equipment	Type	Size	Manufacturing Company	Application
Asphalt Paver	Tytan 410S	Width 3.0~7.5m	ABG, Germany	Placing (with Double Tamper)
Heavy Vibration Roller	SW—100	10t	Sakai Heavy Industry	Initial and Secondary Compaction, Rubber Coated on both Wheels
Pneumatic Tired Roller	TS290	25t	ditto	Finish Rolling
Vibroplate	—	50kg	—	Side Compaction
Road Sprinkler	—	4t	—	Spray Curing

For placing, a high compaction paver with double tamper screed was selected. A compaction allowance of 8 % was expected based on preliminary tests at plant site. When single-operation placing of 5.5 m wide section was tried, a lack of ununiformity in transverse density was found during placing, and this caused transverse unevenness during compaction. Surface texture was corrected in the same manner as that employed for Route 17. The finished surface was thoroughly protected with curing mat for three days and sprinkled with water for curing.

4.3.3/ JOINT

Joint spacing of 5 m minimum, 20 m maximum was selected (Figure 7).
As construction was carried out in summer, a maximum spacing of 20 m instead of 30 m was selected to avoid cracking.
Both longitudinal and transverse joints were saw-cut to the dimensions shown in Figure 9. To avoid cracking found within 48 hours in the trial on Route

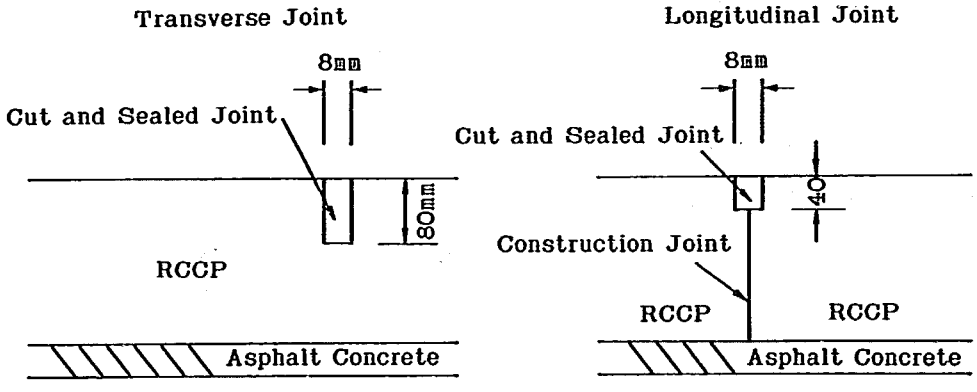


Figure 9 Structure of Joints

17, the first cutting was done on the day of finishing with one saw blade; saw cuting with a double blade was carried out again 3 to 4 days later.

4.4/ QUALITY AND WORKMANSHIP

4.4.1/ QUALITY CONTROL AT PLANT

1) Control of consistency

The good test results for water content and density were obtained with deviations of 0.2 % and 0.01 g/cm^3 respectively.

2) Strength of RCC

As shown in Table 10, high flexural strength exceeding 70 kgf/cm^2 at 28 days under standard curing conditions was obtained. Strength at 3 days also was exceeded 45 kgf/cm^2, and it was confirmed that RCC is capable of attaining high early strength.

Table 10 Flexural Strength Test Result (Unit : kgf/cm^2)

	Standard Curing						Field Curing					
	7 Days		14 Days		28 Days		3 Days		7 Days		28 Days	
Jul. 14	55.7	55.5	66.7	68.4	81.8	71.4	61.1	55.5	56.8	57.4	74.6	70.0
	53.4		69.7		64.2		57.9		55.4		67.9	
	55.8		68.8		68.3		47.6		60.0		67.6	
Jul. 17	70.6	61.1	65.7	67.0	75.3	72.8	46.8	47.5	55.7	55.7	72.7	69.5
	58.5		65.5		70.9		50.4		53.2		69.8	
	54.3		69.7		72.2		45.3		58.2		66.1	

3) Abrasion resistance and durability

Satisfactory test results for both spike-ravelling and freezing and thawing tests were 5.0 cm^2(allowable target value of 9.5 cm^2) and 80 % (target value of 60 % or more) respectively.

4.4.2/ TEST RESULTS AT THE SITE

1) Measured results of core samples

Measured results of core samples are shown in Table 11. A compaction degree of more than 95 % was obtained for both in-bound and out-bound lanes.

However, the slightly lower value obtained for the out-bound lane might have been caused by lower water content and/or the mechanical insufficiency of the paver. The degree of compaction at the bottom is lower than that of the upper part.

Table 11 Slab Thickness and Density

	No.	Thickness (cm)		Density (g/cm^3)		Rate of Compaction (%)		Degree of Compaction (%)		Rate of Compaction of Each Layer (%)	
			Mean		Mean		Mean		Mean	According to the Core No. 1	
Lane for Tokyo W=5.5m (Jul. 14)	1	29.5	31.4	2.341	2.349	94.9	95.2	97.9	98.2	Upper Layer	99.2
	2	32.0		2.338		94.8		97.8		Middle Layer	95.7
	3	32.7		2.368		96.0		99.0		Lower Layer	92.1
Lane from Tokyo W=4.0m (Jul. 17)	1	31.6	31.7	2.382	2.372	96.6	96.1	99.5	99.1	Upper Layer	99.8
	2	31.6		2.364		95.8		98.7		Middle Layer	97.2
	3	31.9		2.369		96.0		99.0		Lower Layer	93.7

Note : Degree of Compaction = $\frac{\text{Density of Cored Sample}}{\text{Compacted Density at the Date of Construction}}$

Compacted Density at the Date of Construction = 2.391(Jul. 14). 2.394(Jul. 17)

2) Compaction allowance (Decrease in thickness from compaction)

Compaction allowance of 8 % was estimated prior to the trial, and the results were within a range of between 6 and 8 %.

3) Density test results

Density was measured at site with a RI in the same manner as described in the trial on Route 17. An average value of 95.4 % was obtained.

4) Surface evenness

As shown in Table 12, good surface evenness of 2.03 was obtained. Careful water content control during mixture production and the use of a double tamper paver might have contributed to the achievement of this result.

Table 12 Result of Measured Evenness

	RCCP	
Date of Construction	Jul. 14	Jul. 17
Evenness	2.13	1.92
Mean	2.03	

5) Skid resistance

Table 13 shows the results of skid resistance as measured with a portable skid resistance tester. Though the values are slightly lower than those on conventional concrete slab surface, the results are still satisfactory.

Table 13 Result of Measured Skid Resistance(BPN)

Type of Pavement	Skid Resistance Converted into Values at 20°C			
	1	2	3	4
RCCP	74	76	74	75
Conventional Concrete Pavement	83	86	86	85

5/ SURVEY RESULTS AFTER OPENING ROAD TO TRAFFIC

5.1/ TRIAL SECTION ON ROUTE 17

After two winters in service, visual inspection indicates thar the surface of RCCP is in good condition, though surface texture is rougher than that of conventional concrete slab. Table 14 shows the follow-up survey results.

1)Rut depth

The change of rut depth is shown in Figure 10 where initial rut depth is shown as 0. As can be seen, asphalt pavement shows the deepest ruts and RCCP shows almost the same depth as conventional concrete or precast concrete slab.

2) Skid resistance

At the initial stage, RCCP shows skid resistance equivalent to that of conventional concrete. However, the value of 70 for RCCP at present is slightly lower than that of 76 for conventional concrete.

3) Crack

Two cracks exist. One appeared immediately after construction, and its 4.3 m length grew to 5.4 m at present; crack width widened from 0.6 mm to 6 mm. Another 0.7 m long and 0.1 mm wide crack appeared after two winters.

5.2/ TRIAL SECTION ON ROUTE 18

After one winter in service, visual inspection shows overall good results. Table 15 shows the survey results.

Table 14 Result of Follow-up Survey

Location	Type of Pavement	Month of Survey	Rut Depth (mm)	Evenness (mm)		Skid Resistance	Degree of Cracking (%)
			Mean	Outside	Inside		
Route 17 Horinouchimachi Shimojima	RCCP	Nov. 1988	0.0	4.38	4.55	82.6	1.02
		Mar. 1989	1.1	3.77	5.20	77.8	1.21
		Nov. 1989	1.1	4.12	5.40	69.0	1.26
		Mar. 1990	1.5	4.01	5.32	70.5	1.45
	Conventional Concrete	Nov. 1988	0.0	1.48	—	84.8	0.0
		Mar. 1989	0.8	1.53	—	77.3	0.0
		Nov. 1989	0.8	1.75	—	73.1	0.0
		Mar. 1990	1.5	1.67	—	76.6	0.0
	Precast PC Slab	Nov. 1988	0.0	—	—	—	0.0
		Mar. 1989	1.0	—	—	—	0.0
		Nov. 1989	1.0	—	—	—	0.0
		Mar. 1990	1.2	—	—	—	0.0
	Asphalt Concrete	Nov. 1988	0.0	1.48	—	—	0.0
		Mar. 1989	2.0	1.58	—	—	0.0
		Nov. 1989	2.2	1.24	—	—	0.0
		Mar. 1990	6.7	1.42	—	—	0.0

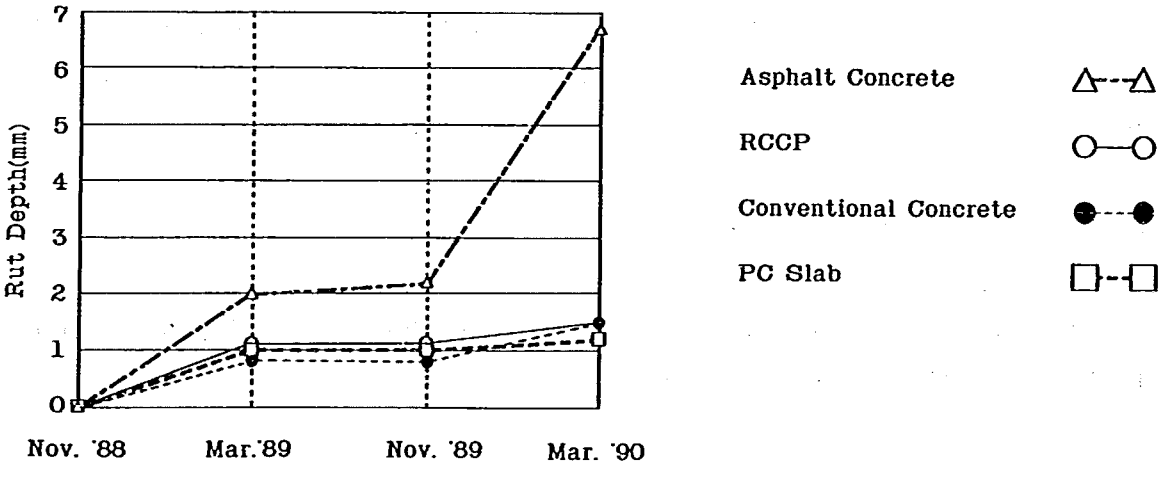


Figure 10 Periodically Measured Rut Depth

Table 15 Result of Follow-up Survey

Location	Type of Pavement	Time of Survey	Rut Depth (mm)	Evenness (mm)		Skid Resistance	Degree of Cracking (%)
			Mean	Outside	Inside		
Route 18 Arai	Conventional Concrete	Nov. '89	0.0	1.65	1.25	85.1	0
		Mar. '90	0.3	1.18	1.30	67.0	0
	Plain Concrete	Nov. '89	0.0	2.35	1.42	89.2	0
		Mar. '90	0.2	2.23	2.42	66.1	0
	RCCP	Nov. '89	0.0	1.75	1.85	82.5	0
		Mar. '90	0.7	1.79	1.92	73.4	0
	Asphalt Concrete	Nov. '89	0.0	1.78	1.90	83.4	0
		Mar. '90	2.0	2.33	2.25	74.1	0

1) Rut depth

As shown in Figure 11 (initial rut depth is shown as 0), the rut depth of RCCP is smaller than that of asphalt pavement, but slightly larger than that of conventional concrete.

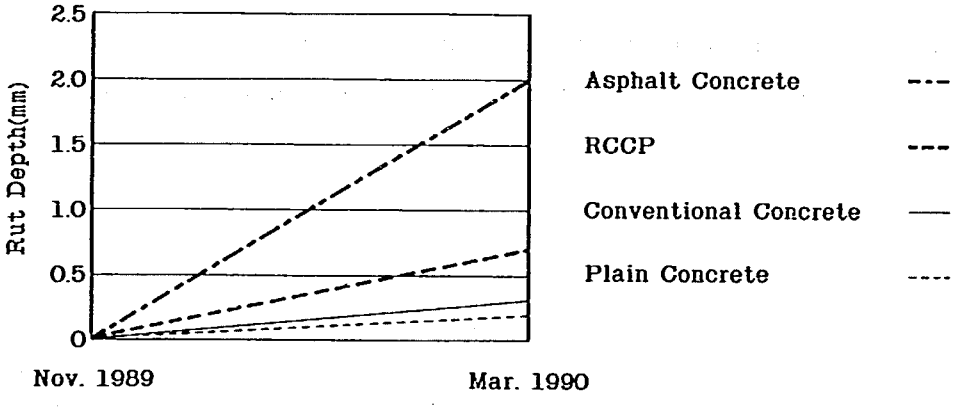


Figure 11 Periodic Change of Rut Depth

2) Skid resistance

At the initial stage, conventional concrete had higher skid resistance; however, RCCP has higher resistance after one winter in service.

3) Cracking

No cracking exists at present.

6/ CONCLUSION

Test results of trials show that RCCP has good possibilities for application as a concrete slab in cold areas. The following points are confirmed through these trials;

1) It is possible to obtain good surface evenness with careful control of water content during mixing and a selection of a high-compaction screed paver.
2) Density is lower at the bottom of the RCCP slab. The influence of this lower density on durability and improvement of construction methods to obtain uniform density require further study.
3) Surface texture is maintained satisfactorily with little abrasion.
4) Joint spacing can be 30 m maximum except when construction is carried out in summer. Saw cutting of joints shall be done as soon as possible, unless the edges of the cut groove have been broken.
5) As early strength can be attained after three days, curing for three days prior to opening of the road to traffic is adequate.
6) Use of steel forms at side end(s) will give better results, improve productivity, and reduce loss of mixture when a thicker slab is constructed.

A final evaluation of RCCP durability in cold areas must be based on the results of follow-up surveys of these trial sections and additional trial in the future.

Present Status and Subject of Roller Compacted Concrete Pavement in Japan

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PRESENT STATUS AND SUBJECT OF ROLLER COMPACTED CONCRETE PAVEMENT IN JAPAN

by

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ABSTRUCT

Research and development of Roller Compacted Concrete Pavement(RCCP) in JAPAN started in 1987, and test pavement of RCCP has been tried all of the country. RCCP has many advantages, such as construction is easy and speedy, the time for curing is short, etc., which improves defects of conventional cement concrete pavement. Therefore, construction area of RCCP is increasing every year, total of them is 250,000m2, with 100 places from 1987 to 1990. On the other way, smoothness (roughness) or surface texture of RCCP is not sufficient for high speed driving of vehicles. SO RCCP is in the test stage in JAPAN for major road pavement. Now, we are studing oh technical subject of RCCP, and overviews of Techinical Guidelines for RCCP, which we are drawing up.

PRESENT STATUS AND SUBJECT OF ROLLER COMPACTED CONCRETE PAVEMENT IN JAPAN

by

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1.. Introduction

In the last few years, world attention has been focused on Roller Compacted Concrete Pavement (RCCP). The technique compacts concrete of zero-slump to extremely low unit water content by use of vibratory rollers. In addition to durability, RCCP shows exclusive characteristics such as economical rationalization and earlier opening to traffic. RCCP has made rapid advances in Japan since it was introduced and in the three years up to the end of 1989 it was applied in about 100 construction sites, 230,000m2. The Japan Road Association is now in the process of preparing a technical guide on RCCP, and this will be published this year. The present situation in Japan, an outline of the technical guide and the problems of application of RCCP to road pavements are discussed in this paper.

2. Characteristics of RCCP method

The amount of cement required in concrete for RCCP (hereafter called RCC) is the same as that for concrete pavements of conventional type but the unit content of water is similar to that for cement stabilization. That is, the application method in RCCP is similar to that to promote cement stabilization as it is roller compacted, and the concrete quality is nearly that of conventional paving. In the RCD method, which is similar to RCCP, a similar mix is used with cement stabilization judged on the amount of cement. RCCP shows the following characteristics when compared with conventional concrete pavements.

(1) lication machinery required is not large-sized as is usual for concrete pavements, but of medium size as in asphalt paving e.g. asphalt finishers, vibratory rollers, tire rollers and so on. This saves on time?and eventual cost and can be applied to small scale projects.

(2) As RCCP is compacted by rollers, it has a large load capacity at the early age and can be opened to traffic much earlier than is conventionally.

(3) The drying shrinkage amount of RCCP is quite small as it has low unit water content. Transversal shrinkage joints can in fact be omitted or the intervals of joints can be lengthened, and this is a benefit for vehicles using the surface.

(4) Construction is possible without moulds and slab thickness can be selected.

(5) The strength and durability of RCC can be expected to be close to conventional pavement concrete, while the following problems with RCCP still remain.

i. Inferior surface smoothness compared with conventional can appear as roller compaction is required to obtain sufficient strength on RCC slab.

ii. Inferior mixing of RCC and its application can cause a segregation of materials and surface roughness. Potholes can result.

iii. RCC is an ultra-stiff-consistency concrete and rough finish on the surface is impossible. Further study is required to investigate skid friction for vehicles travelling at high speed.

3. Study and Development of RCCP in Japan

Demand for durable, low-cost pavement has promoted study and development of RCCP in recent years. In 1954, a 5-year Plan for Road Formation was started and road paving in this country increased as shown in Table-1. The cost for repair and renewal also rapidly increased. When the cost of repairing newly constructed pavements was reviewed, they were found to have exceeded expectation and the difference was becoming bigger every year. As roads are constantly required, pavement constructions must be continued. Durable pavement has therefore come into demand to decrease costs of repairs as the growth of budget for pavement repair seems quite limited.

As shown in Table-1, the ratio of asphalt pavement is larger than any other kind as this is relatively simple to apply and requires lower initial cost. By the middle of 1960, the ratio of concrete pavement to asphalt pavement (high grade) hung in the balance, but by 1987 it was 1:4. As shown in Table-1, a number of minucipal roads were included in the concrete pavement category. Most concrete pavements of minucipal road can be regarded as of simple structure, while high grade pavement with concrete is of a ratio equivalent to asphalt pavements. These have been limited to roads under prefectural control. The actual percentage of structurally reliable concrete pavements therefore is 1%. As the number of pavements have increased, demand from local government for pavement not requiring maintenance has also increased. The local governments have a larger total length of roads under their control, with fewer personnel and less traffic than other municipal governments.

One reason for the decrease in the number of concrete pavements is that traffic must be stopped for a long time to accomodate repair work. Therefore concrete pavements are overlayed with asphalt for repair. Vehicles have become of larger size and traffic amounts have increased recently, so damage of main roads has resulted. Pavements with high durability are required and concrete pavements are appraised by road controllers on the condition that earlier opening to traffic is possible. The outline of repair work in a tunnel applying precast RC floor slabs was presented as an example in the last conference in Canada. Since the 1970s, concrete paving in Japan has been carried out using large-sized machinery for the specific

purpose to maintain quality and curtail cost. This has been a hindrance to the search for wider ranges of concrete pavement application, and easier methods have been demanded.

Table -1 (Annual Report of Road Statistics 1987)

Type of road / Surface	National Highway	Prefectural Road			Minicipal road	Total
		Principal local road	Others	Total		
Cement concrete pavement	(4.2) 1,942	(1.4) 693	(1.4) 1,071	(1.4) 1,763	(5.1) 47,051	(4.6) 50,756
High quality asphalt pavement	(80.9) 37,630	(55.3) 27,652	(34.8) 27,024	(42.8) 54,675	(8.7) 79,984	(15.7) 172,289
Low cost road pavement(asphalt)	(12.4) 5,747	(37.3) 18,650	(51.8) 40,244	(46.1) 58,893	(46.3) 426,731	(44.9) 491,371
Gravel road	(2.5) 1,203	(6.0) 3,037	(12.0) 9,313	(9.7) 12,351	(39.9) 367,051	(34.8) 380,605
Total	(100) 46,523	(100) 50,031	(100) 77,651	(100) 127,682	(100) 920,817	(100) 1,095,021

(Notice) Upper : Percentage (%) Lower : Length (km)

4. Characteristics of RCCP in Japan

4.1 Development of RCCP

Studies and development of RCCP started three years ago in February 1987 in Osaka when the experimental application mentioned before was carried out. Since then, many organizations have experimented with applications of RCCP from various standpoints and technical improvement has been remarkable, although unknown uses remain to be discovered. RCCP, especially the mix design method of concrete for RCCP has been focused on.
Application of RCCP started as pavements for private premises and industrial areas. As experience increased, pavements were laid in lumber depositories, harbour yards and public roads. RCCP was regarded as suitable in yards as the area was so huge that low-cost was required and the pavement surface smoothness was not expected to be like roads. Study on application of RCCP on public roads was started by the Kanto Regional Construction Bureau, Ministry of Construction in cooperation with the Cement Association in 1987 as a 3-year plan. Since then, experiments and surveys to investigate the possibilities of applying RCCP have been undertaken on roads where the traffic is not so busy. Local bureaus of construction have applied RCCP experimentally on roads for construction and state roads constructed as part of pilot projects and the Japan Road Association ordered local governments to apply RCCP experimentally on 24 prefectural roads in 1988 through 1989. Since 1989, RCCP has been applied even on busy national highway. Here, RCCP will be discussed on the basis of application results in Japan.

4.2 Locations of RCCP applied

Most RCCP applied in Japan up until now has been experimental. In some cases, RCCP has been applied by private firms as costs are quite reasonable compared with other methods. In Figure-1, the results of RCCP application have been classified according to location. Applications in industrial areas had a considerable share at the beginning, but as technically improvements were made, RCCP was applied more to public roads. RCCP was first applied to industrial areas to confirm workability and performance before it was applied to public roads. In the experimental applications, mixes of RCC, combination of machinery and various methods were tried out and the

results put into practice on RCC application on public roads. General lengths of public roads within 200m were experimentally applied in most cases. For this reason, the volume of RCCP applied in yards occupies about 30% the application but 50% the size. The application of RCCP on public roads started where traffic was light. In the early years RCC was applied as a thick surface on main roads busy with large-sized vehicles. RCC slabs have also been applied as foundation beds of composite pavements.

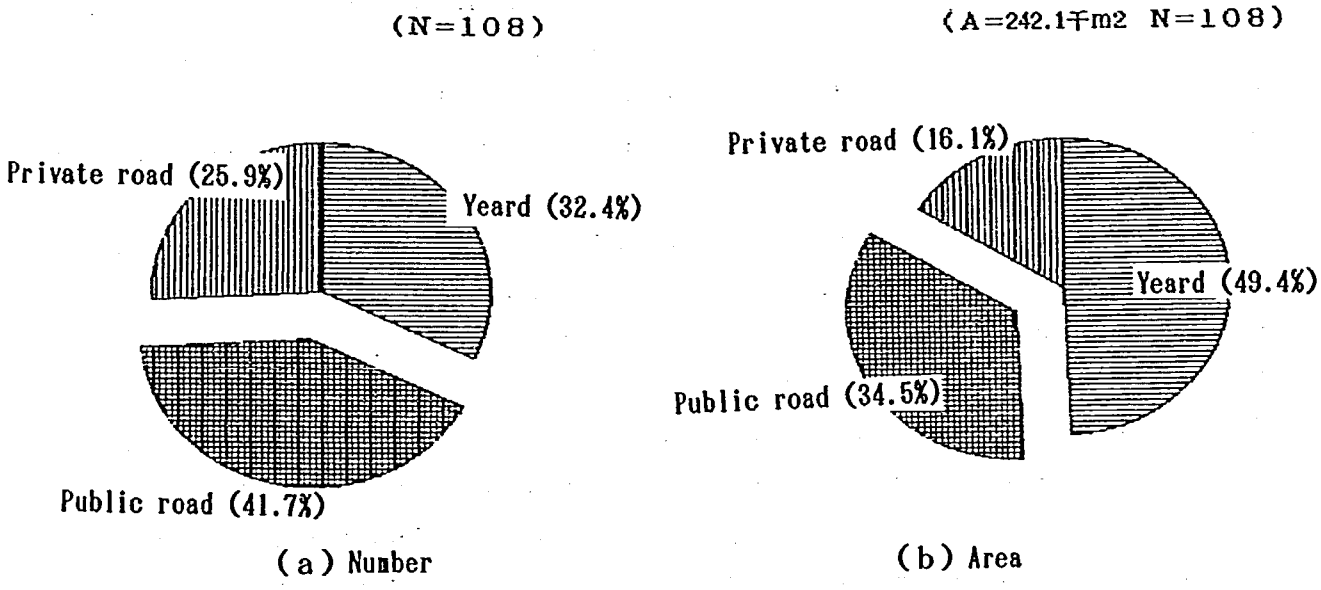


Fig-1 Application of RCCP

4.3 Pavement Structure

4.3.1 Slab thickness

The slab thicknesses of RCCP used so far have been within 10-38cm as shown in Figure-2 and most are as "Manual for Design and Concrete Pavement" stipulates. The slab thickness for L Traffic is 15cm, A Traffic 20cm, and B Traffic 25cm. Requisites of RCCP concrete have been flexural strength as in conventional concrete pavements, and physical properties of hardened concrete slabs are regarded to be the same as RCCP. The structures of both RCCP and conventional concrete should basically be the same. A load carrying capacity of base course layer,

$K30 \geqq 20$ kgf/cm3 is required as a standard regardless of traffic classification in the experimental application specified by the Japan Road Association, as this encourages compaction of concrete slabs and achieves strength.

The Technical Guide of the Japan Road Association, being edited at the moment, also includes this formula. Table-2 shows concrete slab thicknesses according to the classification of traffic amount in the technical guide. To limit the maximum slab thickness to 25cm, the bending strength has been designed to be larger on pavements of busy traffic roads.

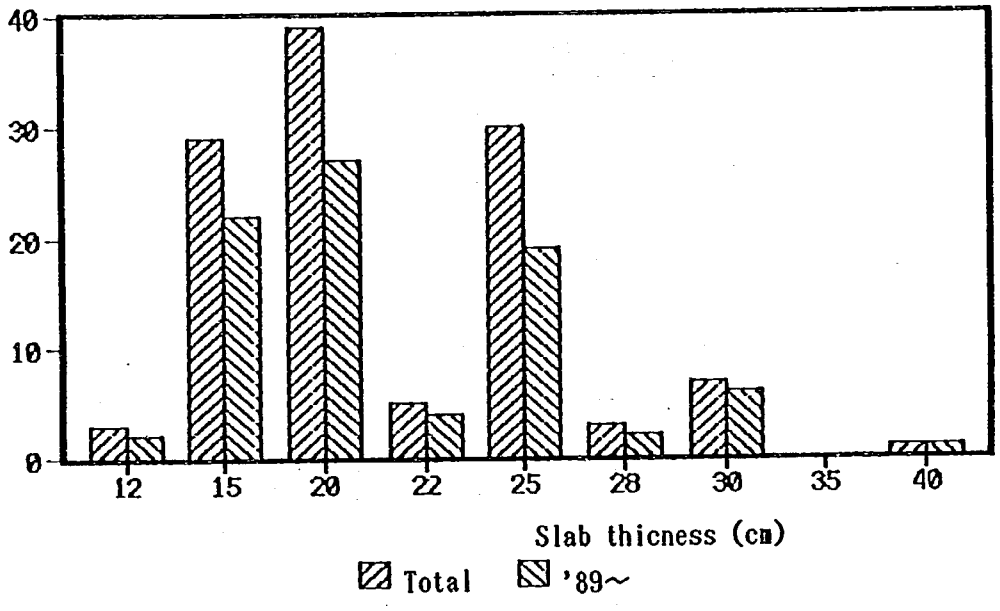


Fig -2 Distribution of RCCP slab thickness

Table-2 Thickness of RCCP slab (unit : cm)

Traffic volume classification	Traffic volume of heavy vehicles(unit/day/direction)	Design bending strength	
		45kgf/cm2	50kgf/cm2
L-traffic	less than 100	15	—
A-traffic	more or equal to 100 but less than 250	20	18 *
B-traffic	more or equal to 250 but less than 1000	25	22 *
C-traffic	more or equal to 1000 but less than 3000	—	25

*: You may choose this thickness if you have some reason for construction.

(unit : cm, kgf/cm², %)

Design CBR		4	6	8 or more
L Traffic	Granular materials	RCC σ_b=45 15 Mechanically stabilized layer CBR > 80 20 crusher-run CBR > 20 25	RCC σ_b=45 15 Mechanically stabilized layer CBR > 80 25	RCC σ_b=45 15 Mechanically stabilized layer CBR > 80 20
	Granular materials + Asphaltic layer	RCC σ_b= 45 15 AC 4 Mechanically stabilized layer CBR > 80 10 crusher-run CBR > 20 25	RCC σ_b= 45 15 AC 4 Mechanically stabilized layer CBR > 80 15	RCC σ_b= 45 15 AC 4 Mechanically stabilized layer CBR > 80 15
	Cement treated base	RCC σ_b=45 15 CTB σ_7=20 20	RCC σ_b=45 15 CTB σ_7=20 15	RCC σ_b=45 15 CTB σ_7=20 15
A Traffic	Granular materials	RCC σ_b=45 20 Mechanically stabilized layer CBR > 80 20 crusher-run CBR > 20 25	RCC σ_b=45 20 Mechanically stabilized layer CBR > 80 25	RCC σ_b=45 20 Mechanically stabilized layer CBR > 80 20
	Granular materials + Asphaltic layer	RCC σ_b= 45 20 AC 4 Mechanically stabilized layer CBR > 80 10 crusher-run CBR > 20 25	RCC σ_b= 45 20 AC 4 Mechanically stabilized layer CBR > 80 15	RCC σ_b= 45 20 AC 4 Mechanically stabilized layer CBR > 80 15
	Cement treated base	RCC σ_b=45 20 CTB σ_7=20 20	RCC σ_b=45 20 CTB σ_7=20 15	RCC σ_b=45 20 CTB σ_7=20 15
B Traffic	Granular materials + Asphaltic layer	RCC σ_b= 45 25 AC 4 Mechanically stabilized layer CBR > 80 10 crusher-run CBR > 20 25	RCC σ_b= 45 25 AC 4 Mechanically stabilized layer CBR > 80 15	RCC σ_b= 45 25 AC 4 Mechanically stabilized layer CBR > 80 15
	Cement treated base	RCC σ_b=45 25 CTB σ_7=20 20	RCC σ_b=45 25 CTB σ_7=20 15	RCC σ_b=45 25 CTB σ_7=20 15

Fig.-2.2 Design examples of RCCP (Design bending strength 45kgf/cm²)

(unit : cm, kgf/cm², %)

Design CBR		4	6	8 or more
A Traffic	Granular materials	RCC σ_b=50 18 Mechanically stabilized layer CBR > 80 20 crusher-run CBR > 20 25	RCC σ_b=50 18 Mechanically stabilized layer CBR > 80 25	RCC σ_b=50 18 Mechanically stabilized layer CBR > 80 20
	Granular materials + Asphaltic layer	RCC σ_b= 50 18 AC 4 Mechanically stabilized layer CBR > 80 10 crusher-run CBR > 20 25	RCC σ_b= 50 18 AC 4 Mechanically stabilized layer CBR > 80 15	RCC σ_b= 50 18 AC 4 Mechanically stabilized layer CBR > 80 15
	Cement treated base	RCC σ_b= 50 18 CTB σ_7=20 20	RCC σ_b= 50 18 CTB σ_7=20 15	RCC σ_b= 50 18 CTB σ_7=20 15
B Traffic *	Granular materials + Asphaltic layer	RCC σ_b= 50 22 AC 4 Mechanically stabilized layer CBR > 80 10 crusher-run CBR > 20 25	RCC σ_b= 50 22 AC 4 Mechanically stabilized layer CBR > 80 15	RCC σ_b= 50 22 AC 4 Mechanically stabilized layer CBR > 80 15
	Cement treated base	RCC σ_b= 50 22 CTB σ_7=20 20	RCC σ_b= 50 22 CTB σ_7=20 15	RCC σ_b= 50 22 CTB σ_7=20 15
C TRaffic *	Granular materials + Asphaltic layer	RCC σ_b= 50 25 AC 4 Mechanically stabilized layer CBR > 80 10 crusher-run CBR > 20 25	RCC σ_b= 50 25 AC 4 Mechanically stabilized layer CBR > 80 15	RCC σ_b= 50 25 AC 4 Mechanically stabilized layer CBR > 80 15
	Cement treated base	RCC σ_b= 50 25 CTB σ_7=20 20	RCC σ_b= 50 25 CTB σ_7=20 15	RCC σ_b= 50 25 CTB σ_7=20 15

*: You may choose this thickness if you have some reason for construction.

Fig-2.3 Design examples of RCCP (Design bending strength 50kgf/cm²)

4.3.2 Joints

For the purpose to improve durability of concrete pavement wire mesh is included to stop crack propagation and slip-bars and tie-bars are fixed at joints to obtain load transfer while the corner ends have reinforcing steel. Joints in conventional concrete pavements are essential to leesen stress acting on slabs by allowing drying shrinkage and expansion due to temperature change. Joints are liable to be weak in strength and have to be reinforced by steel. Careful attention should be paid therefore to the applications. It is, however, quite difficult to reinforce the concrete slabs by placing metal inside as the RCCP properties of stiff-consistency and roller compaction are affected. In RCCP, drying shrinkage has been observed to be less than in conventional concrete due to the smaller amounts of water and cement contained. In RCCP, shrinkage joints have not been used because the cutters cause artificial cracks. Reinforcement using slip-bars has not been applied even for the cutter joint method, so for example, for paving in industrial areas, shrinkage joints are not included and shrinkage joints with cutters have been set at intervals of 1.5-2 times those of conventional pavement. With summer applications, no irregular cracks due to shrinkage have yet been observed apart from joints including cracks which occurred before the cutters started work.

4.4 RCC Mix

4.4.1 Mix design method

The process of mix design of RCC is as shown in Figure-3 in accordance with results of survey and research from the past. To begin with, the optimum moisture content or unit content of water for proper consistency of the RCCP mixture with proper aggregation particle size is obtained. The specimens (10x10x40cm) are then subjected to flexural strength testing varying the unit content of cement to find out the optimum cement content which can meet the requirements of mix strength at 28-days of age. The consistency of RCC is subject to the smoothness of the roads and the condition of compaction as well as the condition of compaction by vibratory rollers. The optimum consistency therefore matters very much in the conditions of mix design. However, neither conventional slump tests nor shaking table consistency tests can be practiced on ultra stiff-consistency concrete. Three methods have been applied so far are:

1. Soil compaction testing by means of tamping,
2. Modified VC values calculated by VC test machine or filling ratio,
3. Compaction rammer to produce a marshal specimen.

All three methods make little difference in the result of the mix, so method "3." has been nominated as the standard for the technical guide of the Japan Road Association. Other methods can also be applied if necessary.

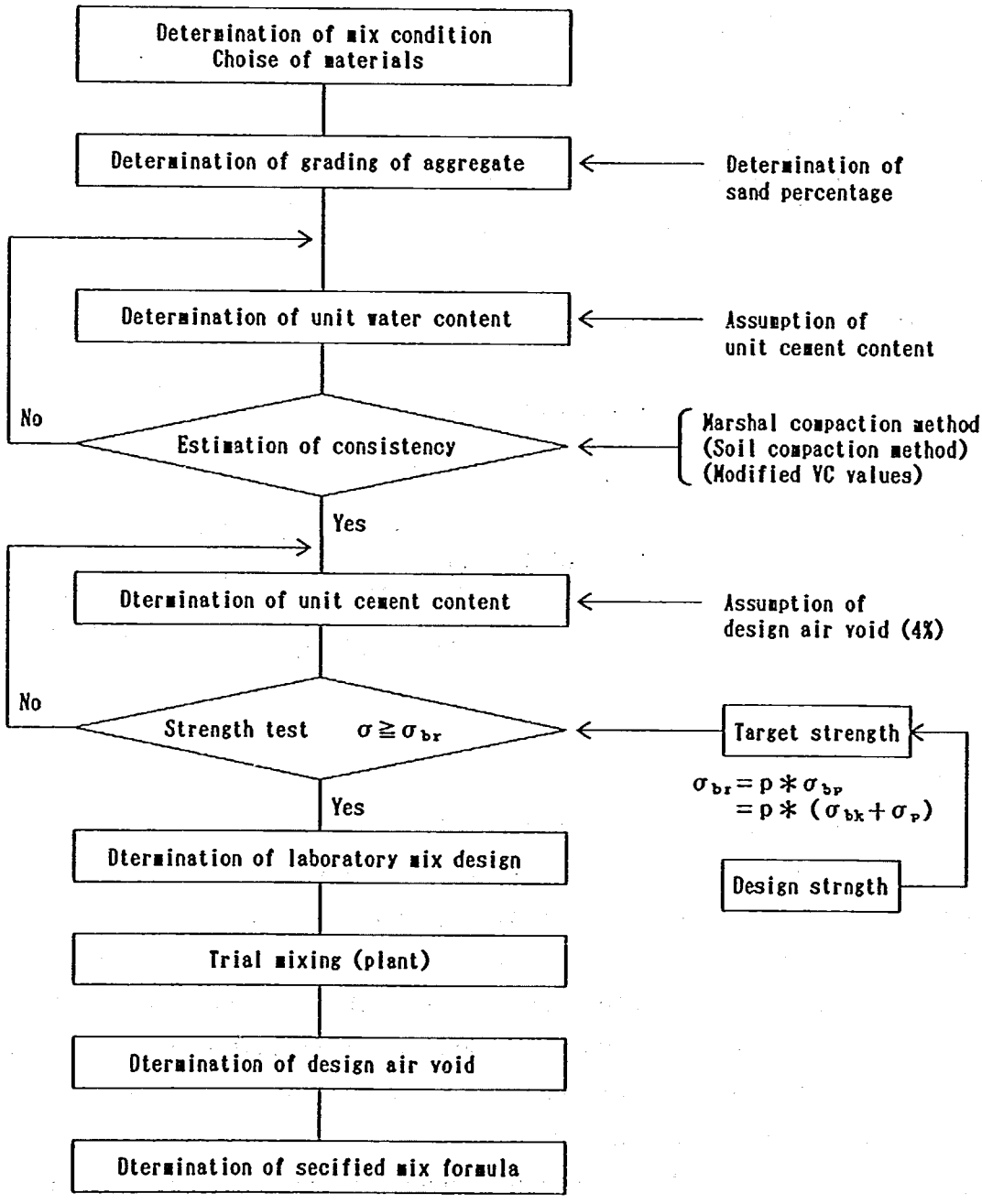


Fig-3 Mix design procedure

Table-3 Estamination method of consistency

Method	Marshal compaction method	Soil compaction method	Modified VC values
Testing apparatus	Marshal rammer,Mould Balance(about 5kg)	4.5kg rammer Mould(φ10*20cm) Balance(about 5kg)	VC test machine AC 200v,Stopwatch Balance(about 30kg)
Compaction mechanism	Indirect impact	Direct impact	Bivration(3000vpm,a=1mm),Weight(20kg)
Quantity of samples	about 2kg	about 4kg	about 15kg
Measurement time	about 3min.	about 6min.	about 5min.
Estimation method	Degree of compaction	Degree of compaction	Degree of compaction Modified VC values
Advantage	• Short measurement time • Easy opperation • Easy estimation of energy (13.1kg*cm/cm3)	• Standardzed by JIS • Many experiment (contractor) • Easy estimation of energy (13.1kg*cm/cm3)	• Sensitive to variation of unit water content • Little sampling error(large sample)
Disadvantage	• Much sampling error(small sample)	• Long measurment time • Many compaction times (65times*3layer)	• Large test machine (need AC 200v) • VC value varies dpends Testing Apparatus.

4.4.2. RCC mix

RCCP is a pavement paved with stiff-consistency concrete after applying finisher on asphalt pavement. The mix of RCC should have a workability in accordance with the form of application, and should show the expected strength after drying. In RCC, material segregation is rarely observed in the pavement and the mix should provide for quality of easy compaction, fine surface and smoothness. Compared with conventional concrete, a mix of RCC requires 30-40kg/m3 less unit content of water and larger amount of coarse aggregates per unit. The RCC mix for road pavement used in recent times, provides a larger S/A to obtain a dense surface texture. Figure-4 shows distribution of RCC mix in Japan.

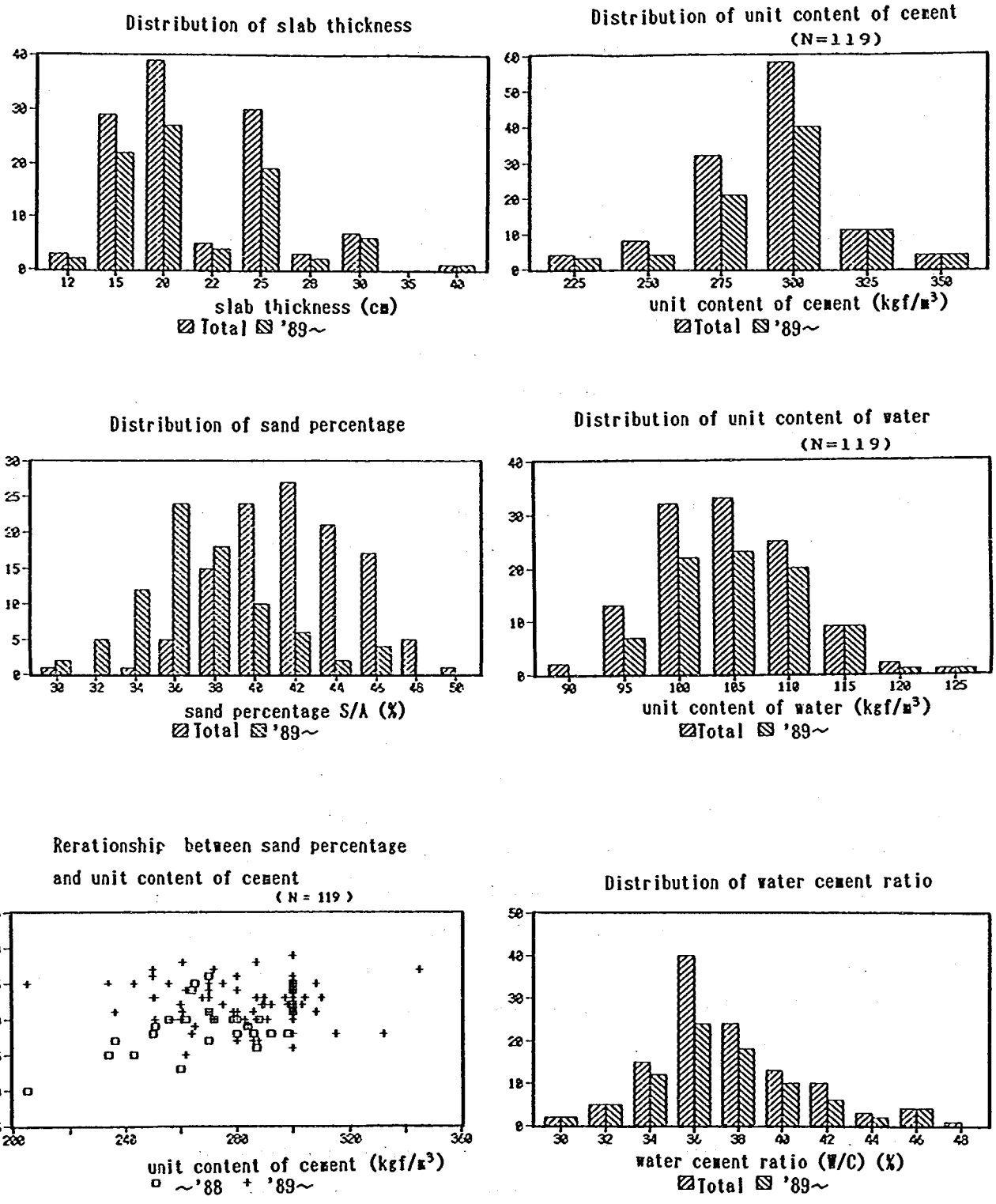


Fig -4 Distribution of RCC mix

4.5 Construction

Examples of RCCP construction process are shown in Figure-5. RCC has so far been mixed in conventional concrete mixing plants. The most important factor in the mixing process is a strict control of the unit content of water. RCC consistency influences the whole construction. When water is in excess, the surface of the mixture moves, producing waves and finally insufficient smoothness. On the other hand, when water is in shortage, insufficient density is caused and the strength does not come to full play. During the construction, segregation is liable to occur and a fine surface texture cannot be obtained. Control of the unit content of water (especially surface water of sand) is therefore the key to a good mixture. During haulage, the mixture in the truck should be covered with a plastic sheet to avoid evaporation of water. According to the technical guide ?from the Japan Road Association, the total length of time until completion of compaction including haulage should be less than one hour and with retarder, no later than 90 minutes.

4.6 Surface smoothness

Of the seven experimental applications checked by Japan Road Association in 1988, data on surface smoothness along a longitudinal section measured with a 3m profile meter are shown in Table-4. When compared with the standard for cement concrete quoted in the manual i.e. 2.0mm, 2.4mm for asphalt pavement, the values are not favorable. This shows the difficulty in getting smoothness in RCCP. For this reason, the total lengths of construction so far have been kept to 50-200m as the process is still experimental, and uniform RCC slabs cannot be assured as construction was intermittent due to insufficient supplies of concrete.

Table-4 Surface smoothness of experimental applications (1988)

Slab thickness (cm)	15	20	25
Surface smoothness(mm)	1.4~1.6	1.4~2.9	2.4~3.0

5. Problems and the Future of RCCP

Although there are some problems to overcome, the RCCP method still provides?outstanding characteristics and is expected to come into wider use. The?problems with RCCP are mentioned hereafter.

5.1 Construction in heavy traffic areas and main roads

Constructions with RCCP have been carried out on country roads where the traffic is not busy and favorable results have been shown so far. In wider use on public roads, RCCP will have to be applied in heavy traffic areas and high standard roads. Concrete pavements including RCCP provide excellent durability but are inferior in terms of thickness and construction cost because a certain thickness of concrete slab has to be assured even on light traffic roads. In asphalt pavings, as traffic loads are bigger, the growth of thickness expected for the structure is rapid, but in concrete pavements, the growth of thickness is moderate. The difference in initial cost for both pavements is closer. For this reason, RCCP have been applied on heavy traffic roads. In the use of RCCP on heavy traffic roads, it is more difficult to get favorable smoothness as thickness increases. On main roads, however, road surface quality as well as durability are requirements compared with on local roads where the length of trip is comparatively short. The limit of RCC thickness to obtain favorable smoothneess is considered to be 25cm for a one-layer construction using present available construction machinery and techniques. According to the new technical guide, design bending strength shall be 50kgf/cm2 to limit the maximum thickness of 25cm?for C Traffic. For D Traffic, the base course has until now been reinforced and RCC pavement slab has been overlayed by an asphalt surface. On high-speed roads, the above-mentioned composite pavement shall be applied to?maintain surface smoothness and slip resistance. Nevertheless, the structural design?of composite pavements has not been considered established yet, and will?remain a theme for further study.

5.2 Establishment of constant supply system of RCC

Construction speed of RCCP is fast. (approximately 0.5-1m for spreading finishers) Daily construction amounts will be determined according to the

supply from concrete mixing plants. Being of ultra-stiff consistency, the production amount of RCC is lower, 0.5-0.7 times that previously considered. In addition, as unit content of water is small, consistency under construction is easily varied. Intermission in the construction of?finishers due to delay of concrete causes the quality of the concrete to be inferior, and badly influences compaction work. Therefore, for wider use of RCCP, the establishment of production techniques and a constant supply of RCC with reliable quality will be essential.

6. Conclusion

RCCP has great potential in development but also has a lot of problems. In Japan, however, to make the most of the excellent characteristics of RCCP, a technical guide is now being produced based on present techniques to promote the technique. The work has not been completed yet, and further study and investigation are expected.

A Twenty Five Year Report on the Continuously Reinforced Concrete Pavement in Koriyama

H. Ohta, T. Obara, Ministry of Construction

A twenty five year report on the Continuously Reinforced Concrete Pavement in Koriyama

by

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(now: Japan Sewage Works Agency)

ABSTRACT

In the Tohoku region where is cold and snowy area, many vehcles wear studded tires or tire chains in winter. Many highways are paved with concrete, because of its abrasive resistance.

However, the concrete pavement has a defect, namely transverse joint. Therefore, a study on Continuously Reinforced Concrete Pavement (CRCP) without transverse joints, was carried out.

The Koriyama CRCP was built along the national highway Route 4, in 1963, by the Koriyama National Highway Construction Office, and has served general traffic for 25 years. However, longitudinal cracks, a sign of ultimate failure, broke out recently and overall repairs were required.

So, the final survey was carried out in 1988, and the 25 year records were reported. Those are as follows;

(1) Decreasing of the mean spacing of transverse cracks stopped 8 - 10 years after construction. And final values of those were 24 cm - 28 cm.

(2) According to the observation of core samples, almost all the crack openings were less than 0.2 mm, and no corrosion was observed.

(3) A mechanical analysis of the service life was done, and the mechanism of the outbreak of longitudinal cracks was described.

(4) It was proved that the CRCP had been maintained in good condition for 25 years and had long durability.

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1/ INTRODUCTION

In the Tohoku region where is cold and snowy area, many vehicles wear studded tires or tire chains in winter. Many highways are paved with concrete, because of its abrasive resistance.

Concrete pavement has expansion joints to allow for the deformation caused by drying shrinkage and temperature changes. However, expansion joints are structural defects in themselves and hampering comfortable driving.

Although concrete pavement requires less maintenance than asphalt pavement, minimum maintenance is still required to prevent the invasion of water and pebbles. Expansion joints hinder mechanical execution as well. Concrete pavement without expansion joints, if possible, would be the most ideal pavement structure.

In 1960, the Ministry of Construction had started to study continuously reinforced concrete pavement (CRCP), which eliminates expansion joints. The study had been conducted by the Public Works Research Institute, constructing several types of CRCP in the yard. However, the lengths of the concrete slab were limited and they had not served public traffic. So, further experiments on the national highway were required.

In 1963, the Koriyama CRCP was constructed by the Koriyama National Highway Construction Office and has served general traffic for 25 years in cold area.

Although the structural condition of the CRCP is stable at present, remarkable damage of the surface caused by studded tires requires overlay repairs. A final study on the Koriyama CRCP was conducted, and the results of the study are reported in this paper.

2/ REGIONAL CHARACTERISTICS

2.1/ LOCATION AND CLIMATE

The Tohoku region is located in the northernmost part of Honshu, the largest island of Japan, including six prefectures of Aomori, Iwate, Miyagi, Akita, Yamagata and Fukusima. The population of the region is about 9.7 million which is 8% of the nation's total population. Its land area is about 67,000 km², accounting for 18% of the total national land area. Most part of the region is cold and snowy in winter. Areas facing the Sea of Japan has particularly much snow due to the wet northwestern seasonal wind. Areas on the Pacific coast are relatively dry in winter. It is very hot in summer, particularly in inland areas.(See Table 2-1)

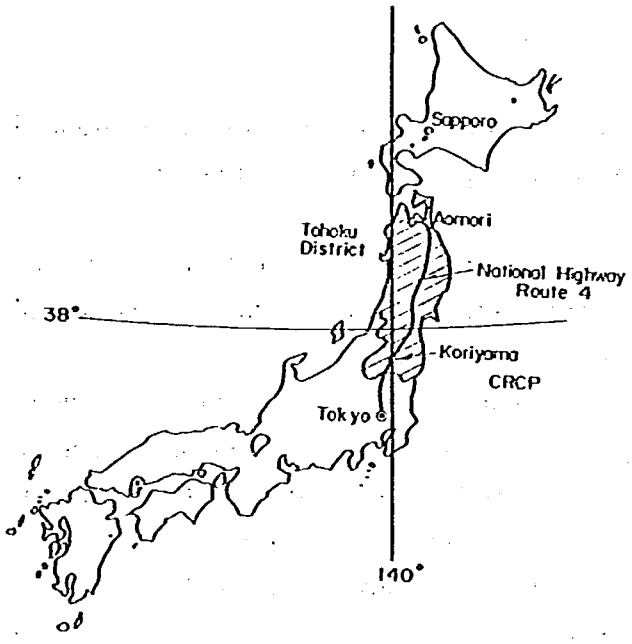


Fig. 2-1 Location of the Tohoku District

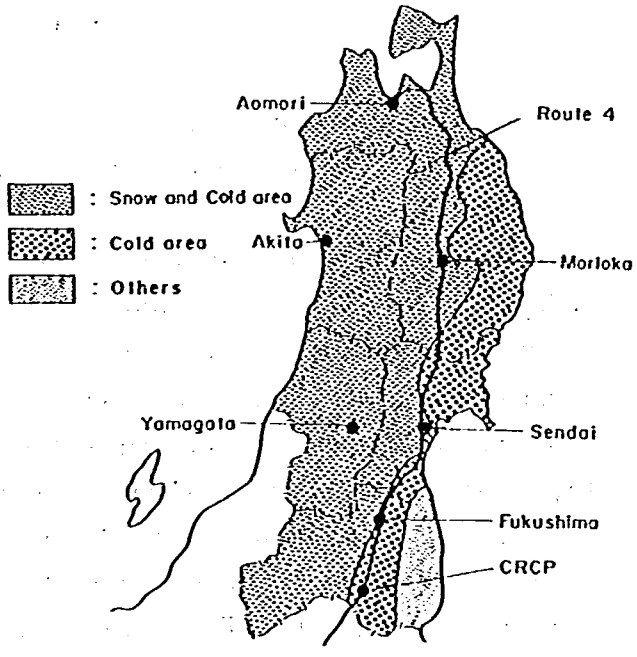


Fig. 2-2 Snow and Cold Area in the Tohoku District

Table 2-1 The Mean Monthly Minimum and Maximum Temperature in the Tohoku District

	Aomori	Morioka	Sendai	Akita	Yamagata	Fukushima
The mean monthly minimum daily temperature in Jan.	-5.1	-6.5	-2.7	-3.4	-4.7	-2.7
The mean monthly maximum daily temperature in Aug.	27.2	27.8	28.2	28.4	29.8	30.2

Most areas in the region have been designated as special cold and snowy areas in accordance with the "Special Law For Securing Road Traffic In Designated Cold And Snowy Areas"(See Fig. 2-2)

The CRCP was constructed along the National Highway Route 4 in Kagamiishi-machi, Iwase-gun, Fukushima Prefecture, which is about 210 km north of Tokyo. This area has an inland climate with fluctuant temperatures. The daily lowest temperatures in January average -2.7 ℃, and the lowest temperature of -18.5 ℃ was recorded on February 4, 1891. The annual precipitation, snowfall and maximum snow depth in 1988 were 1,270.5 mm, 48 cm, 13 cm respectively. This area has been designated as a cold area in accordance with the Law mentioned above.

2.2/ TRAFFIC CONDITIONS

There are many cities and towns scattered throughout the spacious Tohoku region. As such, highways are located to link them together. The total length of roads in the Tohoku region is about 140,000 km, of which national and prefectural highways come out at about 20,000 km. The Tohoku Regional Construction Bureau controls 2,470 km of them. Of the length, 2,070 km (83.8%) is paved with asphalt and 400 km (16.2%) is concrete.

In the Tohoku region, traffic is heavier on roads than railroads. The number of cars owned in the area is more than the national average. The daily carloadings in and around the region average 1.3 million ton. In the region, many vehicles wear studded tire or tire chains in winter. The percentage of studded tire use was 82% in 1984.

The National Highway Route 4 is a major trunk which links Tokyo and Aomori and the location where the CRCP was constructed is close to Koriyama, one of the major cities in the region. As such, the CRCP was used for considerably heavy traffic including a number of large trucks. Table 2-2 shows the traffic density observed near the CRCP after 1962. It totals about 140 million vehicles, of which large ones account for about 49 million, 34%.

Traffic load counted at a point 18 km from the CRCP in the direction toward Tokyo is shown in Table 2-3.

Table 2-2 Traffic Density near the CRCP (Cars/2Lanes)

year	Traffic Density					
	12 hours			24 hours		
	Car	Truck	Total	Car	Truck	Total
'62	1,141	746	1,887	2,245	1,208	3,453
'65	2,890	1,200	4,090	4,546	2,448	6,994
'68	5,673	2,011	7,684	7,580	4,100	11,680
'71	9,135	2,317	11,452	12,660	4,290	16,950
'74	8,145	3,497	11,642	11,345	6,584	17,929
'77	7,822	2,902	10,724	9,465	6,268	15,733
'80	8,199	2,767	10,966	10,224	6,225	16,449
'83	8,695	2,793	11,488	11,644	5,586	17,230
'86	9,632	3,114	12,746	12,892	6,228	19,120
'88	10,702	3,210	13,912	14,450	6,420	20,870

Table 2-3 The Number of Traffic Wheel Loads per 24hours

Year	Traffic Wheel Load (ton)											Total
	0- 3	3- 4	4- 5	5- 6	6- 7	7- 8	8- 9	9-10	10-12	12-14	over14	
'77	4,820	1,207	559	401	289	207	113	66	0	9	1	7,672
'79	8,143	2,811	1,562	734	583	358	196	99	37	16	0	14,593
'80	13,030	1,739	800	494	172	60	10	0	1	0	0	16,306
'81	14,177	1,340	580	306	121	38	15	11	7	0	0	16,595
'82	13,731	1,371	596	380	179	59	30	7	6	0	0	16,359
'83	14,106	1,505	810	488	263	90	40	17	5	0	0	17,324
'84	16,925	1,199	719	297	128	34	14	5	3	0	0	19,324
'85	16,932	1,848	877	452	298	61	21	8	0	0	0	20,397
'86	17,913	1,979	998	490	161	46	15	4	2	0	0	20,608
Mean	13,308	1,666	833	449	232	99	50	24	6.8	2.8	0	16,670

Station : National Highway Route 4, Kinshoji, Shirakawa city, Fukusima Pref.
Lane : for Aomori

3/ DESIGN, CONSTRUCTION AND SURVEY OF THE CRCP

3.1/ DESIGN AND CONSTRUCTION

3.1.1/ THE STRUCTURE OF THE CRCP

(1) Road alignment

Both plane and profile alignments of the area where the CRCP is located are straight. The profile alignment has a descent of 1 in 1000 in the direction toward Tokyo.

(2) The structure of the CRCP

A cross-section and the longitudinal section of the CRCP are shown in Fig. 3-1 and Fig. 3-2, and the outline of CRCP is shown in Table 3-1.

The length of the Koriyama CRCP was 500 m, and the first 150 m from Aomori was a cutting of 6 m, and beyond that point was banked up 1 m on the surface of the rice paddy. The subgrade was clayey loam.

The thickness of the subbase and the base were 30 cm and 25 cm, respectively. Unscreened gravel was used.

The mean bearing capacity factors (K_{35}) of the base course were 51 kg / cm² in the lane for Aomori, and 43 kg / cm² in the lane for Tokyo.

The concrete slab is composed of two lanes combined at the longitudinal joint with tie bars. The lane width is 4.5 m each. (See Fig. 3-3)

Table 3-1 The Outline of the Koriyama CRCP

Concrete slab	Slab thickness	23 cm	Average flexural strength 44.5kg/cm² (Standard curing 28days)
	width	4.5m	
	length	500m	
	Longitudinal reinforcement	0.6%	φ13mm deformed bar c.t.c 9cm
Base	Base	25 cm	gravel K_{30}=47kg/cm²
	Subbase	30cm	
Subgrade	Clayey Loam		

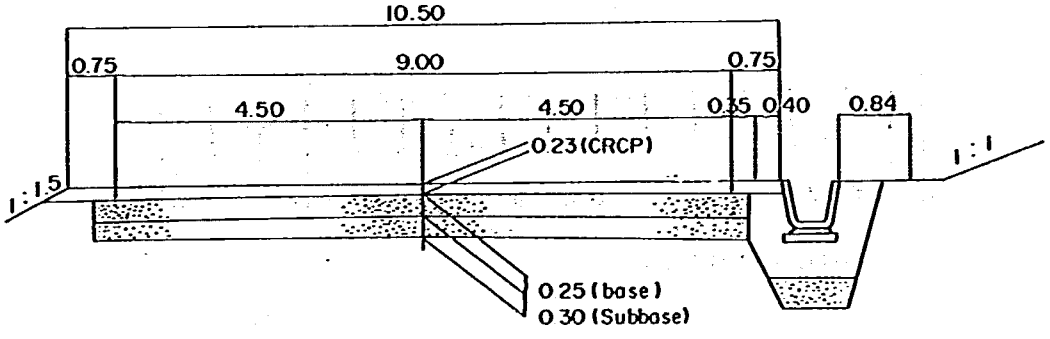


Fig. 3-1 A Cross Section of Koriyama CRCP

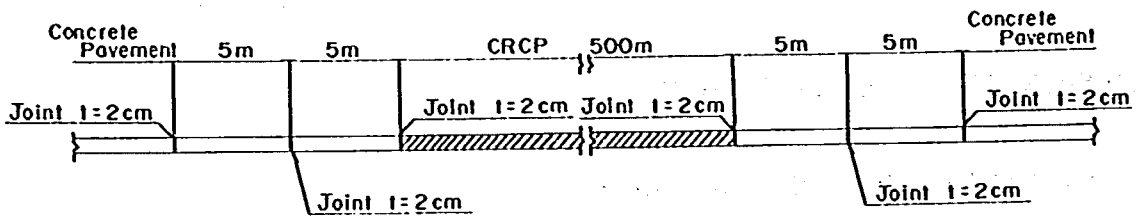


Fig. 3-2 The Longitudinal Section of Koriyama CRCP

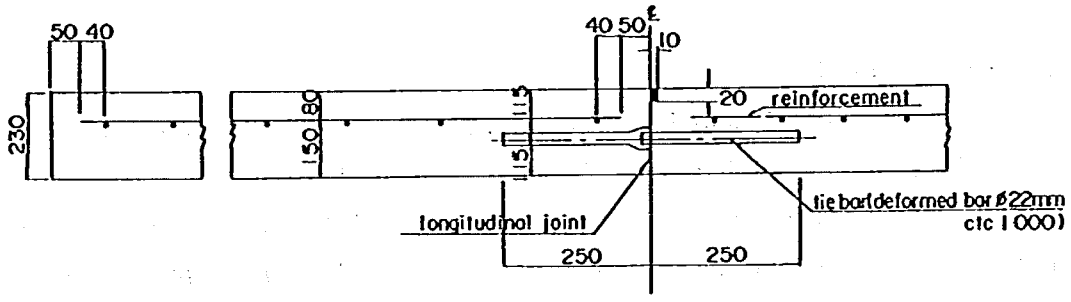


Fig. 3-3 The Structure of Longitudinal Joint

The ends of the CRCP had expansion joints. The cross section of slip bars was larger than that of the longitudinal steel bars of the CRCP so that it could prevent blow-ups due to the reduced rigidity of the CRCP. The adjoining conventional concrete slabs were equipped with two expansion joints each five meters apart to absorb expansion in hot weather. (Figs. 3-2, 3-4)

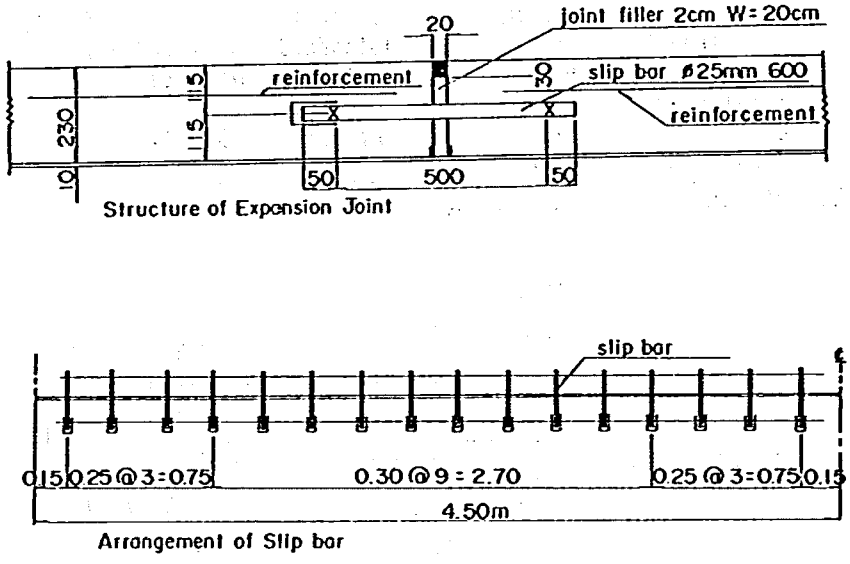


Fig. 3-4 The Structure of Expansion Joint

3.2.1/ DESIGN OF THE CRCP

(1) Thickness of concrete slab

Since the CRCP contains a large quantity of reinforcing steel, it is more strengthened against traffic load than conventional concrete pavement. As such, the CRCP is generally thinner than the conventional one by about one inch in the U.S. The thickness of the CRCP used in this survey was calculated at 20 cm, but it was actually constructed with a thickness of 23 cm by reason of its connection with adjoining concrete and the future increase of traffic.

(2) Arrangement of reinforcing bars

The purpose of the CRCP reinforcements is to disperse shrinkage cracks over a concrete slab so that crack widths can be minimized. The adequate quantity of the reinforcements can be determined by either the Vetter's formula or rule of thumb within the rage of 0.5% to 1.0% of the rate of reinforcement. In this case, it was set at 0.6%, according to the examples set by the Public Works Research Institute and the Vetter's formula (1). [3)]

$$p = \frac{\sigma_c}{\sigma_s + Z E_s - (E_s/E_c)\,\sigma_c} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots (1)$$

where

p : the adequate quantity of reinforcements

σ_c : the tensile strength of concrete (=25kg/cm²)

σ_s : the yield strength of steel bar (=4,000kg/cm²)

E_c : the Young's modulus of concrete (=300,000kg/cm²)

E_s : the Young's modulus of steel bar (=2,000,000kg/cm²)

Z : drying shrinkage of the concrete (150×10^{-6})

The values in parenthesis were assumed for design.

Transformed steel bars 13 mm in diameter were used as longitudinal reinforcements, and 9 mm diameter steel bars as transverse reinforcements. But, to increase rigidity, the 13 mm diameter transformed steel bars were used for every fourth transverse reinforcement, and intersections of those reinforcements were welded.

The size of the steel net was 3.5 m x 2.3 m and weighed 104 kg.(Fig. 3-5). The two steel nets were laid parallel, to make a width 4.4 m, in accordance with the one side pavement width of 4.5 m. The overlapping area of the steel nets was set at 20 cm in width and 35 cm in length, and 64.5 cm for the joint portion. The covering of the reinforcement was set at 8 cm.

The specified mix of the concrete is shown in Table 3-2 and the results of a strength test of 28 days of standard curing are shown in Table 3-3.

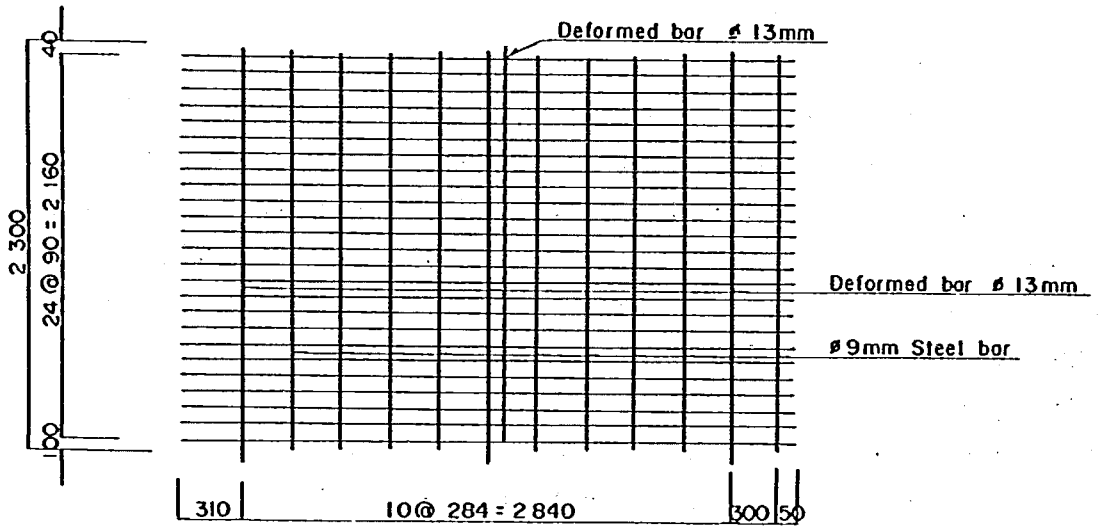


Fig. 3-5 The Arrangement of Reinforcement

Table 3-2 The Specified Mix of the Concrete

Aggregate Maximum Size	Slump	Air Content	Water	Cement	Water Content Ratio	Sand Percentage	Fine Aggregate	Coarse Aggregate		Admixture
								mm 50~25	mm 25~ 5	
mm 50	cm 2.5	% 4±1	kg 121.6	kg 300	% 40.5	% 32.1	kg 629.1	kg 502.2	kg 838.2	% 0.5

Table 3-3 The Results of strength tests of the Concrete

	number of test piece	Mean	Standard deviation
Flexural strength	87	44.5 kg/cm²	2.8 kg/cm²
Compressive strength	18	311.3 kg/cm²	26.0 kg/cm²
Tensile strength	21	27.5 kg/cm²	3.0 kg/cm²
Modulus of Static Elasticity	5	288,000 kg/cm²	9,800 kg/cm²
Modulus of Dynamic Elasticity	4	390,750 kg/cm²	27,500 kg/cm²
Poisson's ration	5	0.2	0.04

3.1.3/ CONSTRUCTION

The construction of the concrete slab was started on June 1, 1963 and completed on June 28, with 9 work-days. The mean construction rate was 120 m/lane/day, the same as that of conventional concrete pavement.

The project was started from the Tokyo side on the lane towards Aomori, then reversed from Aomori to Tokyo.

3.2/ SURVEY

The survey of the CRCP was mainly an observation of the outbreak of cracks in the concrete slab.

The terms of the survey are shown in Table 3-4.

The observation points were set every 5 m from the starting point at Tokyo side, so the last point at Aomori side being No.100.

Table 3-4 Date and Terms of the Survay

No.	Date	Days from the Completion	Crack Observation				Defrection	Profile
			Distribution	Width	Length	Core		
Construction	June 1~28, '63	—						
1	July 12~19, '63	14	O	O	O			
2	Aug. 19~22, '63	52	O	O	O			
3	Nov. 13~19, '63	138	O	O	O	O		
4	Nov. 15 , '64	505	O	O	O			
5	July 11~16, '66	1,110	O	O	O			
6	Apr. 13~22, '70	2,480	O	O	O		O	
7	Nov. 14~18, '77	5,250	O	O	O	O	O	O
8	Nov. 8~ 9, '88	9,260	O	O	O	O	O	O

4/ THE OUTBREAK OF CRACKS AND SURFACE CONDITIONS

The outbreak of cracks in the CRCP had been observed from 1 month after completion until 25 years. The surface conditions between section No. 20 and No. 30, as a typical example, are shown in Fig. 4-1.

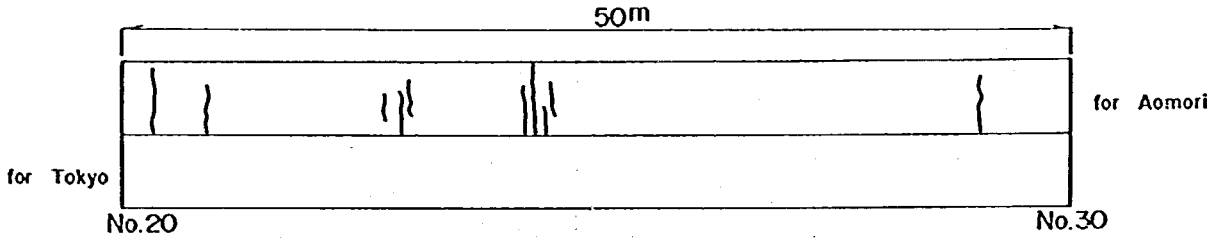
4.1/ TRANSVERSE CRACKS

From Fig. 4-1 (1), (2), and (3), which show early stages of the CRCP, fewer cracks were observed in the lane for Tokyo, which was completed later, compared with the lane for Aomori. The comparison in Fig. 4-1 between (1) and (2) indicates that new cracks have a tendency to break out near the existing cracks rather than between them. Fig. 4-1 (4) shows the result of the survey in 1977, 14 years after the completion. The outbreak of cracks were almost the same on the both lanes. [4)・5)]

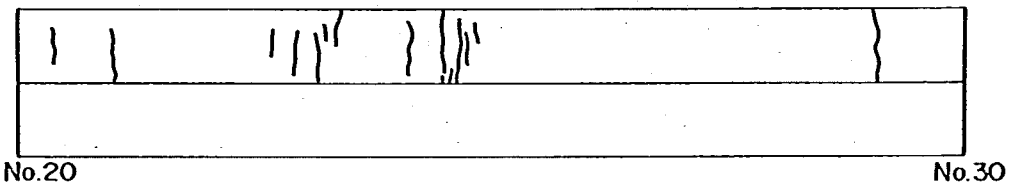
Fig. 4-2 shows an increasing of the total length of the transverse cracks. The total length of cracks increases straight against the laps of time (logarithmic scale) and the length of cracks on both lanes met at 1977, 14 years after construction.
Fig. 4-3 shows a decrease of the spacing of transverse cracks with a lapse of time. The average spacing of transverse cracks settled 8-10 years after construction, then declined thereafter. In 1988, the average spacing of transverse cracks reached 24 cm on the lane for Tokyo, and 29 cm on the lane for Aomori. [5)・8)]

4.2/ LONGITUDINAL CRACKS

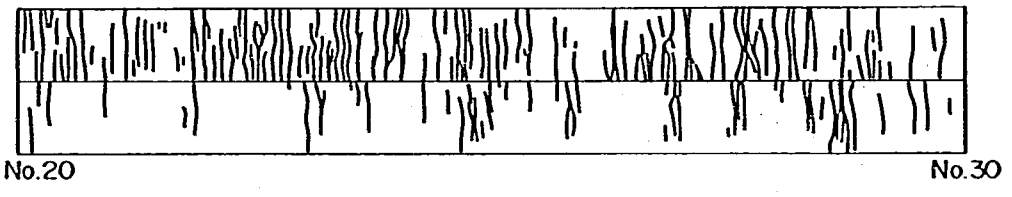
Longitudinal cracks due to traffic load stress are detrimental to the durability of the CRCP. Such cracks were observed in the 1977 survey for the first time. The outbreak of the longitudinal cracks were concentrated in the southern half of the lane for Aomori. This section was banked on poor ground, formerly a rice paddy.
In 1988, longitudinal cracks were observed all over the CRCP; the total length of the cracks was about 170 m. The total length of longitudinal cracks was two times greater than that of the same area measured in 1977. Particularly in the lane for Aomori, the outbreak of longitudinal cracks accounted for 80% of the length of the section.
Since longitudinal cracks indicate a structural failure in CRCP, the Koriyama CRCP was considered a failure. [7)・8)]



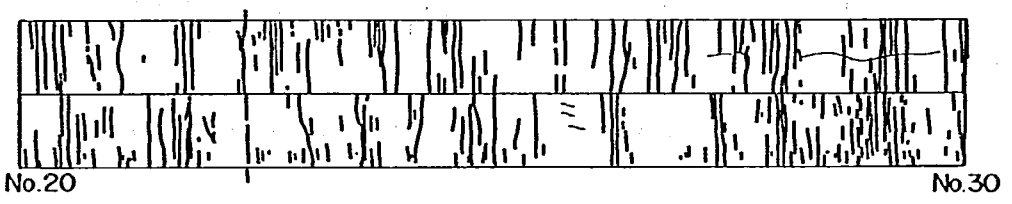
(1) July, 12~19, 1963



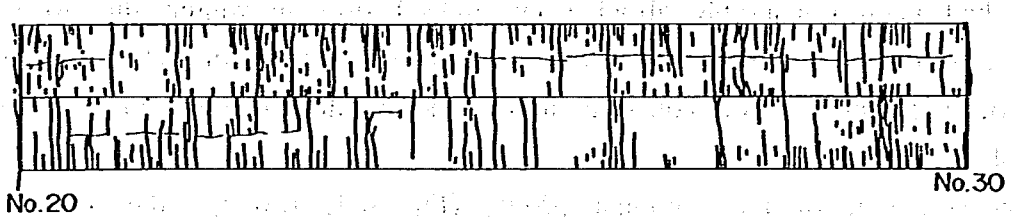
(2) Aug., 19~22, 1963



(3) July, 11~17, 1966



(4) Nov., 14~18, 1977



(5) Nov., 8~9, 1988

Fig. 4-1 The Surface Conditions between Section No. 20 and No. 30

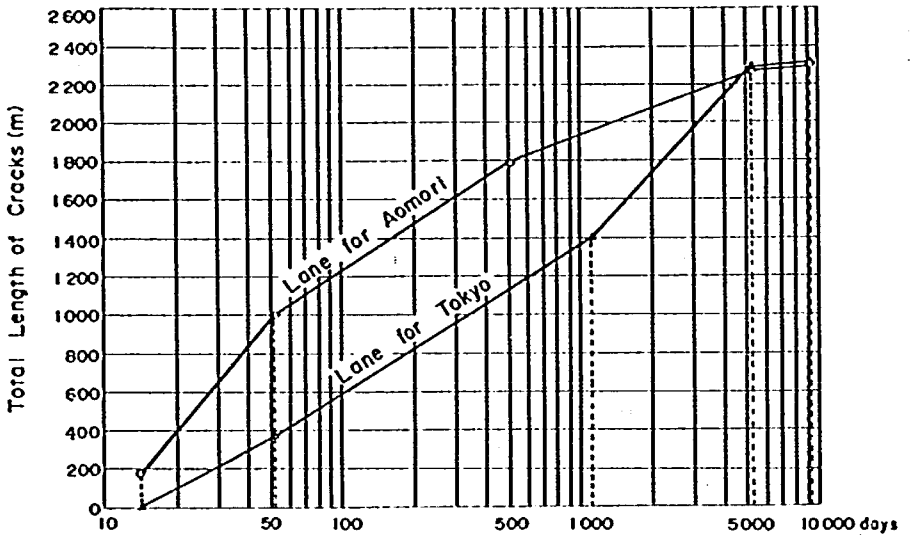


Fig. 4-2 The Increasing of the Total Length of Transverse Cracks

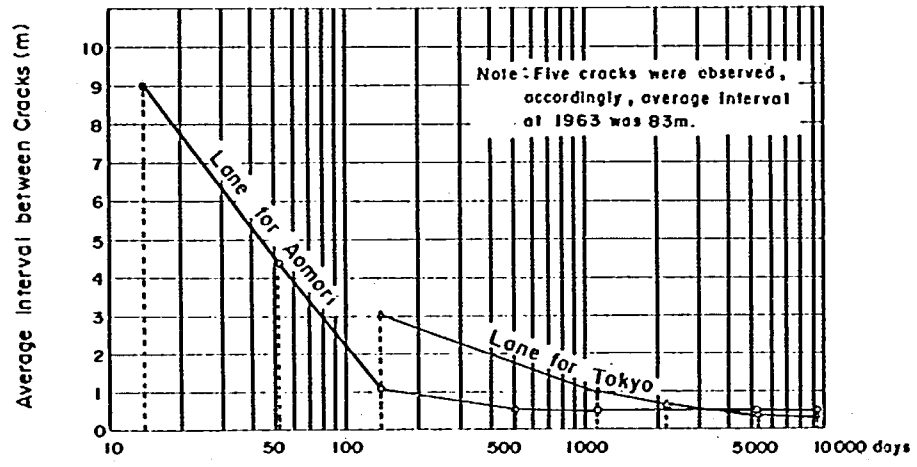


Fig. 4-3 The Decrease of the Average Interval between transverse Cracks

4.3/ SURFACE CONDITIONS

Scaling of the cracks have occurred since 1966, 3 years after construction. The CRCP had been remarkably abraded by studded tires in winter, due to its location in the cold northern region. In 1988, the last survey was conducted, and the several cases of the abrasion of concrete above the reinforcements were observed. (See Photo 4-1)

The measurement of the sectional profile (Fig. 4-4) revealed the abrasion on the part of 1 and 3 meters from the center line where most vehicles run. The abrasion on the left tire trails was particularly severe, reaching about 3 cm in depth. [5), 8)]

The average annual abrasion before 1977 was less than 1 ㎜, while it doubled after 1977. This is because the number of vehicles wearing studded tires increased.

Conventional concrete pavement adjoining the CRCP was overlaid with asphalt in 1983 to repair its structural failure due to joint openings, abrasion, and cracks. But the CRCP remained in good condition with almost no repair for 25 years, showing its long durability.

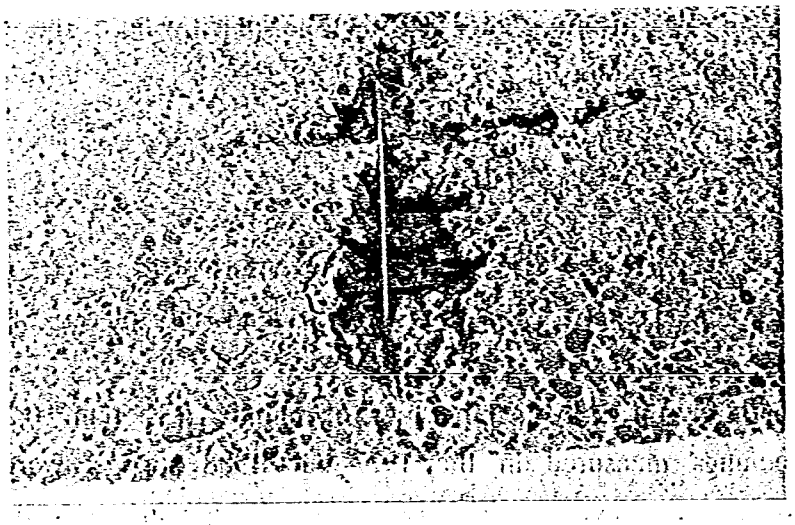


Photo 4-1 Damaged Surface of the CRCP

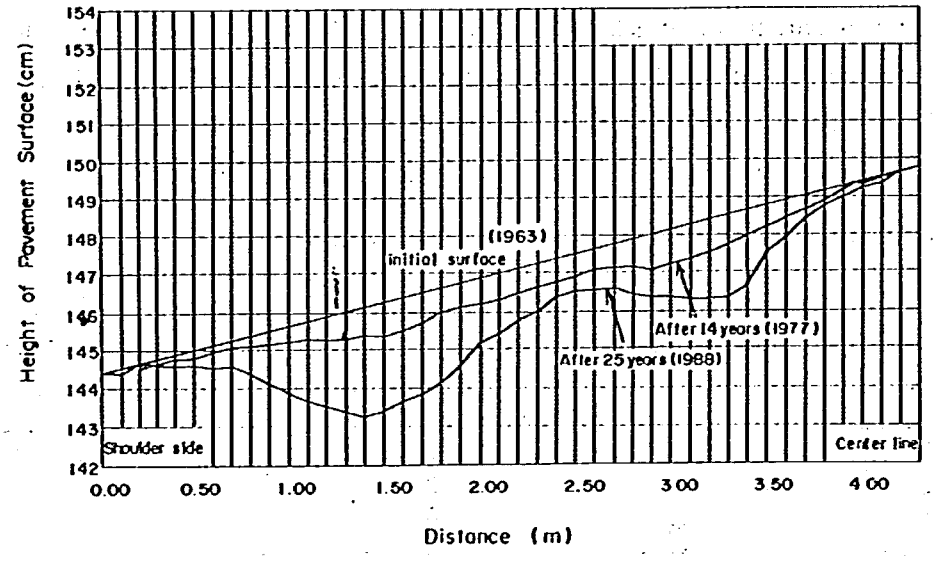


Fig. 4-4 A Sectional Profile of the Pavement

5/ THE INNER STRUCTURE AND THE DURABILITY OF THE CRACKS

The cracks of CRCP disperse the shrinkage of the concrete slab, but tend to become defects. So, it is desirable that crack openings remain minimal by the longitudinal reinforcements. In 1977 and 1988, core samples were collected from all sections of the CRCP. Their inner cracks and the corrosion of the steel bars were surveyed.

The total numbers of core samples were 42 and 45, in 1977 and 1988 respectively. The core samples in 1988 were gathered near the place where the core samples in 1977 were gathered, for comparison.

5.1/ TRANSVERSE CRACKS

Figure 5-1 shows the survey results regarding transverse crack openings. The vertical line shows the distance from the surface, and the places of the steel bars are shown in Fig. 5-1.

The crack openings in the core samples were minute, less than 0.2 mm, and the average crack openings measured in the 1977 and 1988 surveys were 0.027 mm and 0.038 mm respectively. the average increase in crack opening was 0.012 mm ; so, it seemed settle. The distributions of the depth of crack openings are narrow at the positions of the longitudinal steel bars.

After the measurement of crack openings, the state of corrosion of the steel bars was also observed. The steel bars had little rust on the surface, and no decrease of the area of the cross section observed, because of the confinement of the cracks.

Thus the crack openings must have been watertight and negligible.

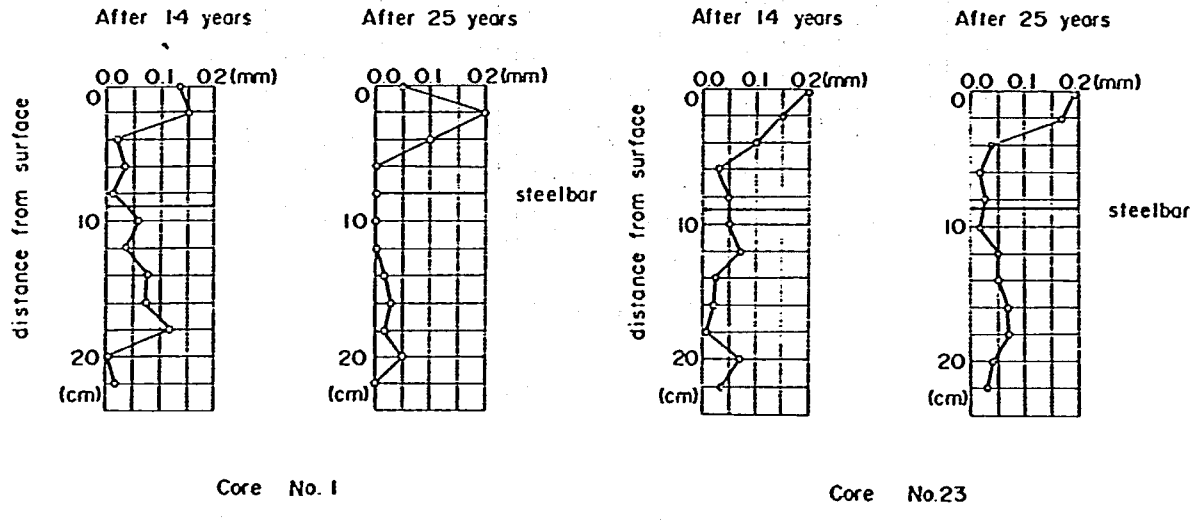


Fig. 5-1 The Crack Openings of Sampled Cores

5.2/ LONGITUDINAL CRACKS

Fig. 5-2 shows the survey results regarding the openings of longitudinal cracks in the core samples. The openings of longitudinal cracks are generally great, reaching about 1 ㎜ at the bottom, because the constraint effect of reinforcements is weak. Longitudinal cracks are no doubt detrimental to the durability of the CRCP.

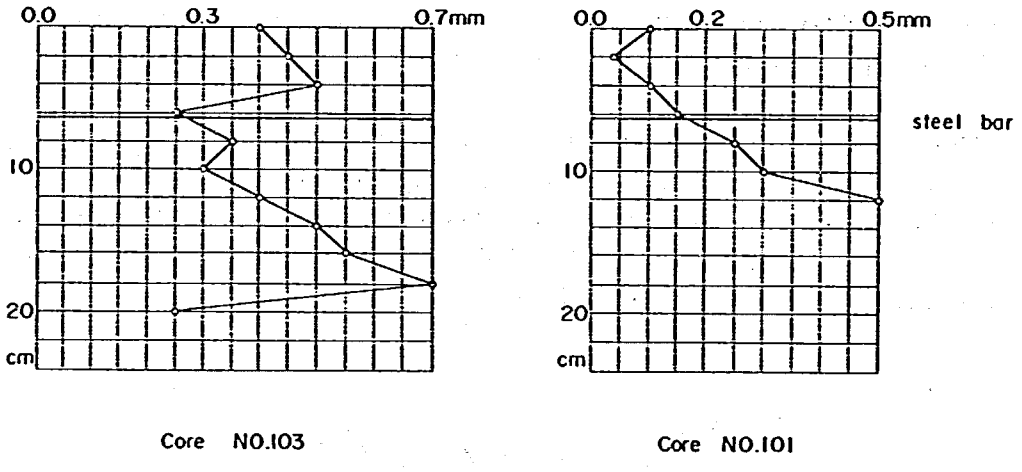


Fig. 5-2 The Crack Openings of Sampled Cores

6/ SURVEY OF THE PAVEMENT BEARING CAPACITY

In 1977 and 1988, the maximum, residual, and elastic deflections of the CRCP were measured at points 1 m and 3 m from the center line of each lane by use of the Benkelman Beam. Rear wheels of a truck (load:5t, air pressure:6.5 kg / cm^2) were used 20 m apart as loading. In 1988, the deflections (D_0, D_{20}, D_{45}, D_{90}, D_{150}) of the CRCP were measured at 0, 20, 45, 90, and 150 cm from the loading center, using the Falling Weight Deflectometer (FWD). Impact loads of about 5 t were used 20 m apart on the outside trails on which most large vehicles run.

Figs. 6-1 and 6-2 show deflections measured on the lane toward Tokyo using the Benkelman Beam and the FWD in 1988. Both figures show that the deflection is greater on the Tokyo side than Aomori side. This is because the section of 350 m on the Tokyo side was banking and the section of 150 m on the Aomori side was cutting.

Fig. 6-2 shows that the change of D_0 corresponds to that of D_{150}. In the case of the CRCP, the change of the bearing capacity of subgrade is thought to be equal to that of the strength of the pavement. Therefore, we assume that the reason why longitudinal cracks concentrated on the Tokyo side at the early stage was that the bearing capacity of subgrade was insufficient.

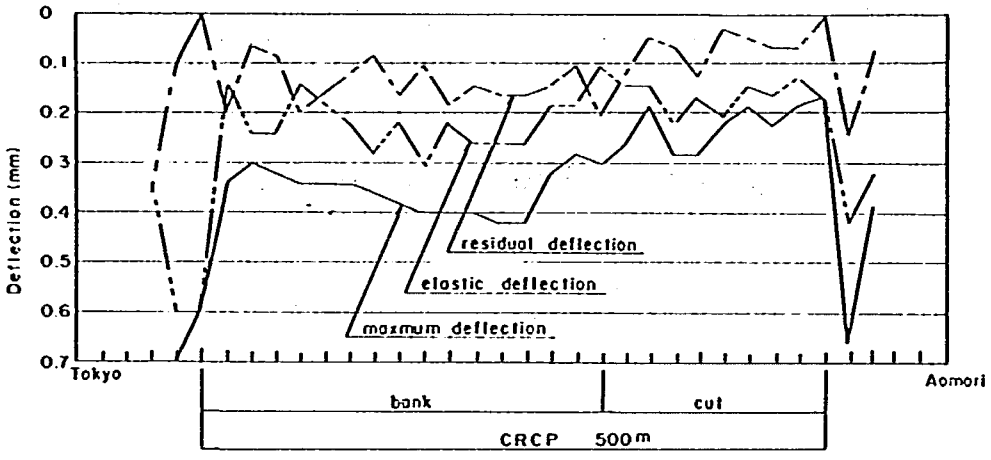


Fig. 6-1 The Deflection of CRCP measured by Benkelman Beam

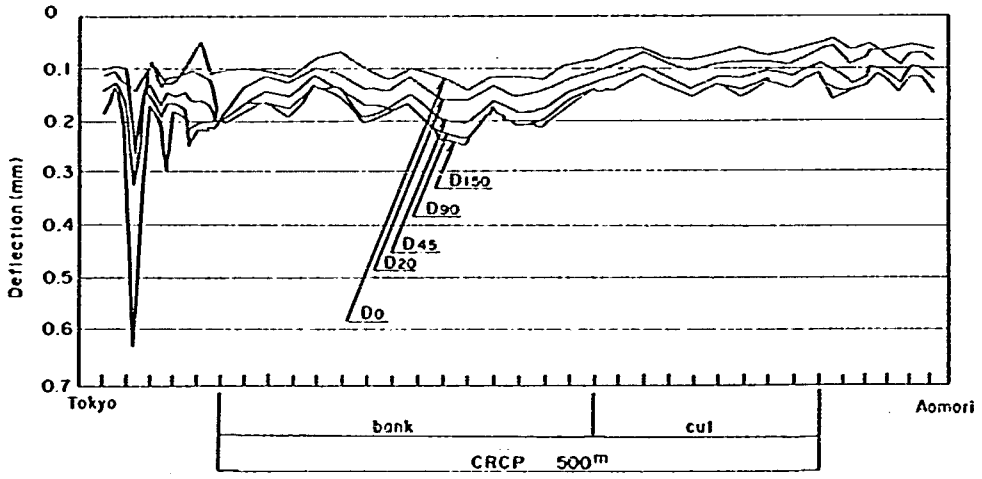


Fig. 6-2 The Deflection of CRCP measured by FWD

7/ STUDY OF SERVICE LIFE

There is no authorized mechanical model to evaluate the service life of the CRCP. Nishizawa et al. [9] tried to evaluate the service life of the CRCP by the FEM model. In that model, the CRCP was taken as a plate divided by transverse cracks and connected by longitudinal reinforcements. However, the model is not used practically, because the mechanical constant value necessary to the analysis is not clear yet.

In this paper, the central loading stress formula was applied when the transverse cracks were confined tightly, assuming that the CRCP was a slab without joints. And the edge loading stress formula was applied when the transverse cracks open widely, assuming that the cracks were equal to joints. The service life of the CRCP was evaluated by comparing the results of each analysis.

The conditions, the evaluating system and the results are as follows;

7.1/ TRAFFIC LOAD

The traffic load data used for the analysis is taken from Table 2-3. For the traffic load, the annual mean value is applied.

7.2/ PAVEMENT STRUCTURE AND VALUES OF MATERIAL

Young's modulus of concrete: $E=300,000$ kg/cm^3

Poisson's ratio of concrete: $\mu=0.2$

Flexural strength of concrete: $\sigma_{bk}=45$ kg/cm^2

Coefficient of thermal expansion: $\alpha=10 \times 10^{-6}$/℃

Thickness of concrete slab: Initial value H=23 cm, decrease with laps of time, and H=20 cm after 25 years.

Bearing capacity factor of the base course: Initial value $K_{75}=20$ kg/cm^3, decrease with lapse of time, and $K_{75}=10$ kg/cm^3 after 25 years.

The decrease of the thickness of the concrete slab was caused by abrasion. The decrease of capacity factor of the the base was caused by the construction method, with river gravel and earth as binder which was used at that time. [2] The percentage of decrease was set at 50%.

7.3/ SYSTEM FOR ANALYSIS

The fatigue analysis was performed by modifying the mechanical planning method of the Cement Concrete Pavement Manual.[10] Those modifications are as follows;

1) A flexural fatigue analysis against longitudinal cracks was done. When the transverse crack open widely, the edge loading stress formula[10] (the Pavement Manual (1)) was applied. And when the transverse crack was confined tightly, the central loading stress formula[11] (Westergaad formula(2)) was applied.

$$\sigma_e = 1.59(1+0.54\mu)\frac{P}{h^2}(\log \ell - 0.75 \log a - 0.18) \quad \cdots\cdots\cdots \quad (1)$$

$$\sigma_i = 1.1(1+\mu)\frac{P}{h^2}\left(\log\frac{\ell}{b} + 0.268\right) \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (2)$$

where

σ_e : the edge loading stress (kg/cm²)

σ_i : the central loading stress (kg/cm²)

P : wheel load (kg)

a : radius of contact area $= 12 + \frac{P}{1000}$ (cm)

h : slab thickness (cm)

E : Young's modulus (kg/cm²)

μ : Poisson's ratio

ℓ : radius of relative stiffness $= \frac{E \cdot h^3}{12(1-\mu^2)\, K_{75}}$ (cm)

K_{75} : Bearing capacity factor of the base course (kg/cm²)

b : radius of equivalent contact area $= 1.6a^2 + h^2 - 0.675h$ (cm)

2) For the flexural fatigue curve, the formula by Koyanagawa et al.[12] (Failure probability 50%) was applied.

$$P = 50\,\% : S = 1.082 - 0.054 \log N \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (3)$$

where

P : failure probability

S : ratio of maximum stress of repeated stress(σ_{max}) and static strength(σ_b)

N : allowable number of cycle

3) For the distribution of traffic loads, the mean value of Table 2-3 was applied.

4) For the process of fatigue analysis, the Pavement Manual was applied.

First, the value of composite stress was calculated from the traffic load stress and the temperature stress, and its frequency was calculated. Then the fatigue value by the composite stress was calculated. By applying Miner's law, it was set that the

longitudinal cracks would break out when the fatigue value came to 1.0. Data necessary for the calculation of the temperature stress were based on the Pavement Manual.

7.4/ RESULT OF ANALYSIS

Fig. 7-1 shows the result of fatigue analysis. Line-A shows the relation between the fatigue value, calculated by the edge loading stress formula, and the service year. Line-B shows the relation between the fatigue value, calculated by the central loading stress formula, and the service year. It is considered that the practical behavior of the CRCP exists in the sphere between the both lines. And it agrees with the fact that many longitudinal cracks have broken out after 20 to 25 service years. Conventional concrete pavement agrees with Line-A. It is clear that the fatigue of the CRCP, and the outbreak of cracks, is slower than conventional concrete pavement.

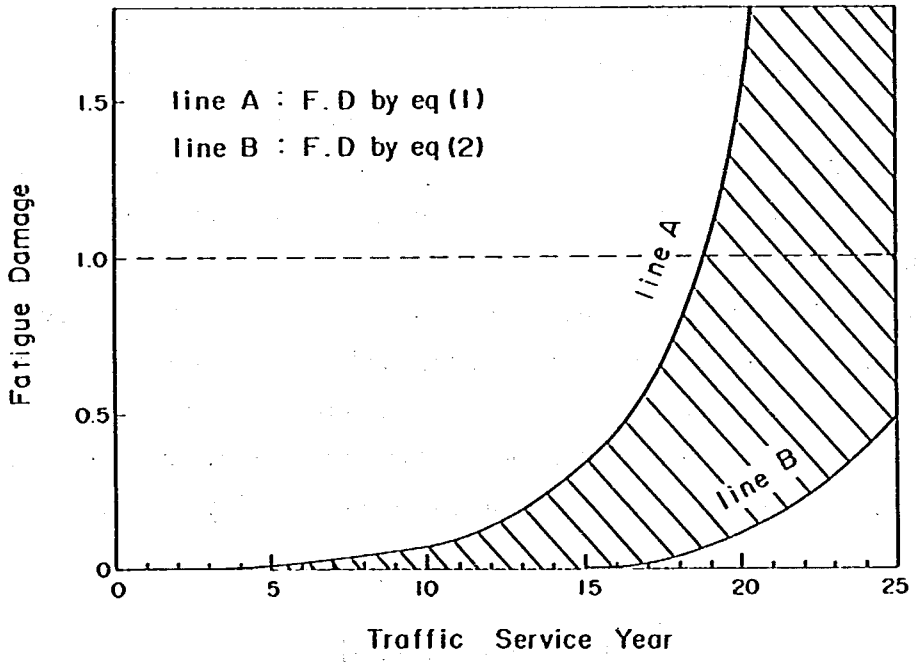


Fig. 7-1 The Relation between Traffic Service Year and the Fatigue Damage of CRCP

8/ CONCLUSIONS

The Koriyama CRCP had served traffic for 25 years, and much valuable data has been recorded, from the completion to the ultimate failure. This paper summarizes the evaluation of the service life of the CRCP, focusing on the results of the crack observation.

The results are as follows;

(1) In the early period, many cracks broke out in the lane for Tokyo. But spacing of transverse cracks became stable in 1966, 3 years after construction, and no differences between the lanes were observed in 1977, 14 years later.

In 1988, 25 years later, the last survey was conducted. The mean spacing of transverse cracks was 24 cm in the lane for Tokyo, and 29 cm in the lane for Aomori.

(2) The crack openings of the core samples were almost less than 0.2 mm and the steel bars had little rust on the surface. The transverse cracks have no connection with deterioration.

(3) In 1970 (7 years later), the longitudinal cracks were observed locally, on the Tokyo side which was constructed on the poor ground. We assume from the survey results that the reason for this is that the bearing capacity of the subgrade was smaller than that of other sections. In 1988 (25 years later), longitudinal cracks were observed in all sections. From this fact, it was clear that this CRCP had ultimately failed.

(4) To evaluate the CRCP, behavior analysis system was applied. As a result of this analysis, it was clarified that the longitudinal cracks outbreak according to the central loading stress formula and the edge loading stress formula.

(5) Unlike conventional concrete pavement which requires repair work on joints and other parts, the CRCP remained in good condition for 25 years and proved its long durability.

[Acknowledgements]

The authors wish to acknowledge the cooperation and assistance received from organizations and individuals who had worked to survey the CRCP, which was done by the Koriyama National Highway Construction Office with the cooperation of the Public Works Research Institute. And without valuable advise of Professor Fukuda of Tohoku University, this paper would have either been impossible or far less complete.

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Concrete Pavement Rehabilitation and Overlay-Ontario's Experience

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CONCRETE PAVEMENT REHABILITATION AND OVERLAY

- ONTARIO'S EXPERIENCE

by

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CONCRETE PAVEMENT REHABILITATION AND OVERLAY - ONTARIO'S EXPERIENCE

ABSTRACT

Currently, highway authorities are faced with the challenge of rehabilitating Portland Cement Concrete (PCC) pavements on high volume freeways. In 1989, the Ministry of Transportation of Ontario initiated a demonstration contract to rehabilitate one such freeway using various innovative PCC pavement repair techniques in one direction and an unbonded PCC overlay in the opposing direction.

The site used to demonstrate these procedures is classed as a four lane divided arterial with 30,000 AADT, 10% identified as commercial. The existing pavement consisted of 225 mm mesh reinforced dowelled PCC with a 21.3 metre joint spacing constructed in 1963.

The rehabilitation techniques used included full depth repair, partial depth repair, diamond grinding and joint sealant replacement on the northbound lanes (N.B.L.) which had experienced moderate deterioration. The southbound lanes (S.B.L.) received a 180 mm thick plain jointed unbonded PCC overlay to address the severe 'D' cracking and spalling of all joints and cracks.

This paper will elaborate on design and construction details and discuss the performance of the pavement, before and after rehabilitation, in terms of load transfer efficiencies and pavement edge deflections based on FWD testing; roughness using the profilograph, PURD, and ARAN; skid resistance using the ASTM brake force trailer; Pavement Condition Ratings and a crack survey. Observations of noise levels, traffic volumes, and surface texture are also presented.

INTRODUCTION

Repair of portland cement concrete (PCC) pavements in Ontario has always posed a challenge. Traditionally rigid pavements have been rehabilitated by removing severely distressed concrete full depth and spalls partial depth and then filled with asphaltic concrete. The patched PCC pavement was then generally overlain with varying thicknesses of hot mix depending on the serviceability level of the pavement and the road classification. This repair technique was used extensively and performed adequately in the short term, however over time the hot mix patches began to distort which significantly reduced the pavement ride.

In order to improve on the Ministry's traditional PCC pavement repair methods, a number of alternative rehabilitation techniques have been tried in recent years. Some of the techniques tried and proven successful included full depth removal of distressed concrete and replacement with either normal PCC or "fast track" PCC [1]. Subsealing of concrete pavement has also been utilized to stabilize slabs which were severely stepped/faulted [2]. A demonstration contract utilizing PCC pavement rehabilitation techniques, unique in Canada, was a project on Highway 126 in London, Ontario, undertaken during the summer of 1989.

This project utilized the most diverse PCC pavement rehabilitation techniques ever attempted by the Province. The repair techniques ranged from partial and full depth repairs to diamond grinding on the N.B.L. and an unbonded concrete overlay on the S.B.L.

BACKGROUND

Highway 126 is located on the eastern limit of the City of London, Ontario. Classified as a four lane divided arterial this facility experiences traffic volumes ranging from 19,000 to 30,000 AADT and of that, approximately 3,000 are commercial vehicles. There is a high percentage of commuter traffic.

The existing pavement was constructed in 1963 with 225 mm of mesh reinforced PCC on 300 mm of granular base and subbase materials. The pavement was constructed with a 21.3 m joint spacing and load transfer dowel bars. The joints were sealed with preformed neoprene seals.

The pavement serviceability varied dramatically between the N.B. and S.B.L. The pavement condition on the N.B.L. was relatively good considering the 26 year service life. The main distresses were: three transverse cracks per slab with one being very severe, some spalling at joints and cracks, and a moderately polished surface. The joints exhibited minimal stepping and good load transfer. The joint seals were 80 - 90% intact, although significant bond failure was apparent throughout.

The pavement condition on the S.B.L. was considerably worse with numerous full and partial width transverse cracks in each slab, severe D-cracking and spalling of all joints throughout and moderate pavement edge distress. Deterioration had advanced to the stage where routine maintenance activities could no longer preserve a safe riding surface.

The variation in pavement performance was attributed to the two types of aggregate used on the opposing lanes. The N.B.L. were constructed utilizing a pit run gravel source for the

coarse aggregate and the S.B.L. coarse aggregate came from a quarried limestone source. The quarried aggregate exhibited absorptive characteristics and was highly susceptible to D-cracking under freeze-thaw conditions.

DESIGN

The design for this project included PCC rehabilitation methods on the N.B.L. and an unbonded concrete overlay on the S.B.L.. This option of concrete repairs with PCC, diamond grinding and an unbonded concrete overlay was selected as it best addressed the actual performance deficiencies exhibited by the pavement and provided the longest anticipated service life.

Northbound Lane Rehabilitation

The work on the N.B.L. involved full depth (1940 m^2) and partial depth PCC repairs, diamond grinding of the surface (29,900 m^2) and sealing of all joints and cracks (5450 m). One fairly significant revision was made to earlier specifications for full depth repairs. These required that the contractor use mechanically mounted hydraulic drills, a gang drill, to cut holes for installation of dowel bars. This new requirement was made to ensure that the holes were drilled in line with the pavement axis and also that they were not elongated or out of round. Adequate load transfer at the patch interface is one of the most critical aspects of the design of full depth repairs.

A plan view of the full depth repair is provided below in Figure 1. The minimum repair size is 2 m long by either one or two lanes wide. General dowel bar layout and references to jointing is also provided on this drawing. Longitudinal tie

bars were only used in repair sections greater than 2 m in length, and consisted of 15 mm deformed bars placed on 600 mm centres.

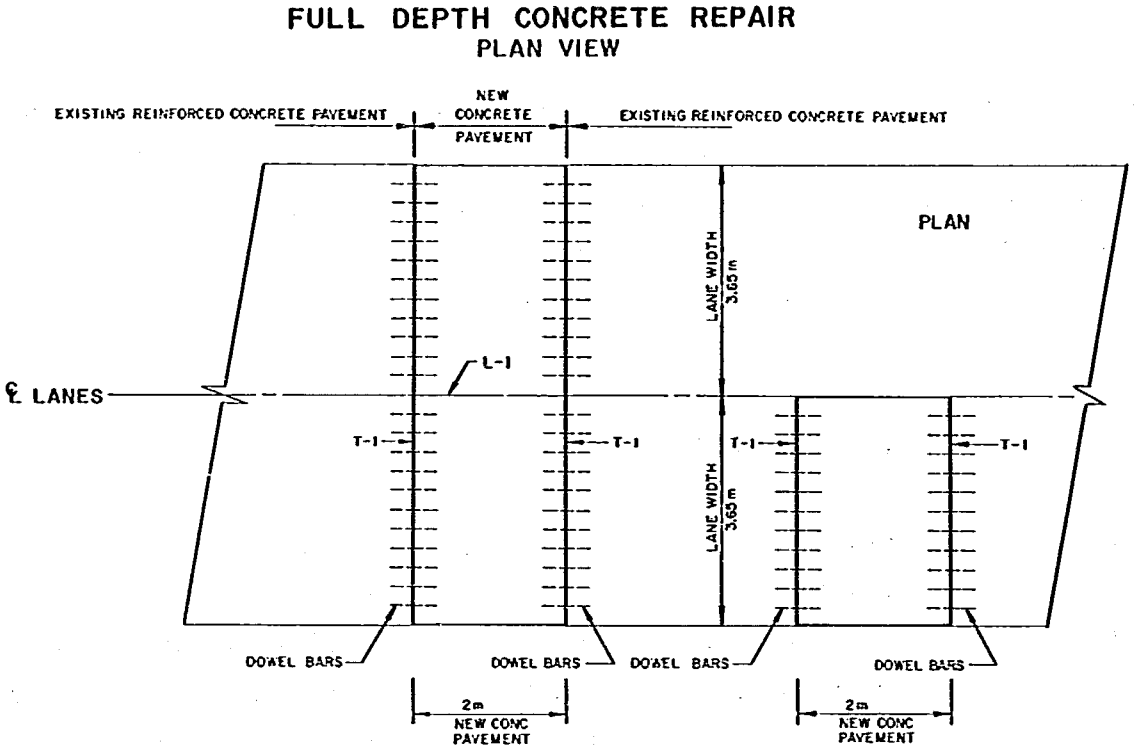


Figure 1

To provide load transfer between the existing and repaired slabs, 32 mm diameter epoxy coated dowels were resin grouted into the existing pavement at 300 mm intervals, mid-height in the slab. The exposed length of the dowel and the concrete face were coated with a bond breaker to insure a working contraction joint was provided. Figure 2 provides the details of the full depth repair and placement of the dowel bars.

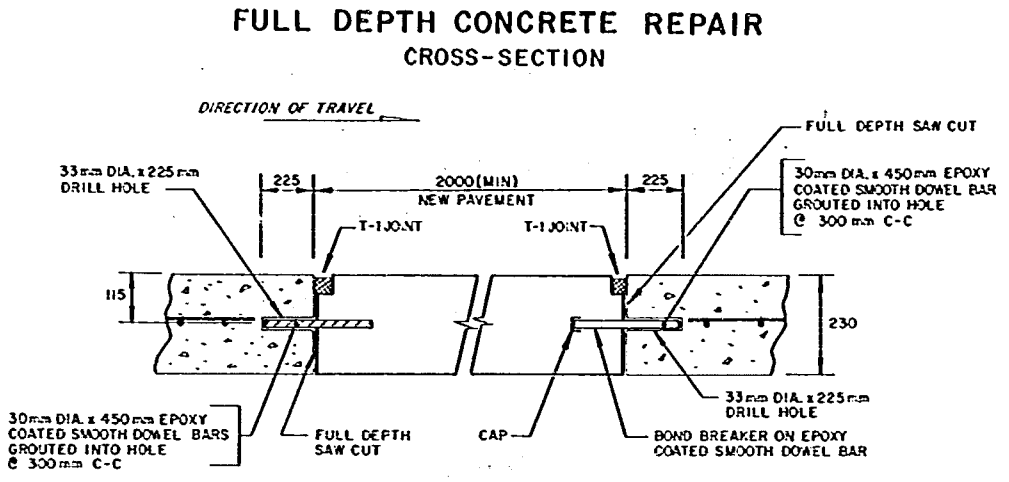


Figure 2

Partial depth repairs with PCC had not previously been undertaken in Ontario, so new drawings and specifications for this work were developed. Typical drawings used for this work are shown in Figure 3. The specification required all unsound spalled concrete to be removed, an average removal depth of 75 mm was used for estimating. Where spalls extended deeper than half the slab thickness, a full depth repair was specified. Following removal and immediately prior to placement of concrete, the exposed surfaces had to be abrasive blast cleaned, and then coated with a PCC paste. Where the repair was adjacent to a crack or joint, a gap had to be formed in this area to allow for expansion at the joint or crack and prevent spalling of the patch. Proper patch isolation is critical to minimize compression failure of the patch.

CONCRETE SPALL REPAIRS

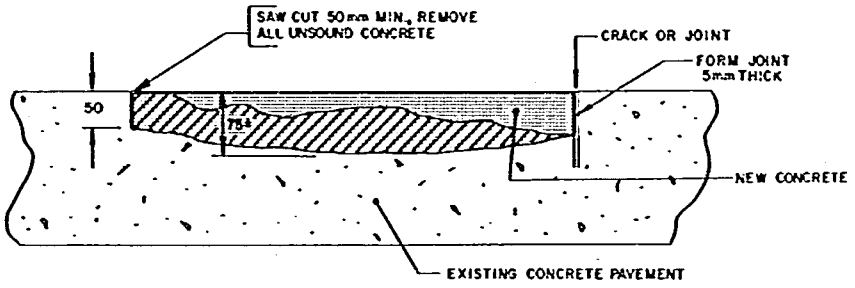


Figure 3

Following curing of full and partial depth repairs the complete northbound pavement surface was retextured by diamond grinding. Diamond grinding was specified to improve skid resistance, restore a smooth surface profile and reinstate an acceptable ride to the pavement surface. The grinding specification required specialized equipment having a minimum 900 mm wide grinding head, with 60 diamond blades per 300 mm of width. The final surface texture to be achieved was also specified.

Following texturing of the concrete surface all existing joints and cracks were cleaned of old joint sealant and incompressibles. Prior to sealant application all joints were sandblasted, cleaned and dried with a hot air lance. A bond breaker was then placed at the bottom of the joint and then sealed with a hot poured rubberized sealant. The joints between the full depth repair areas and the existing concrete were treated similarly except a saw cut for the sealant was required.

Southbound Lane Rehabilitation

The design of the unbonded concrete overlay on the S.B.L. included the removal of any loose spalled material from the existing badly cracked pavement. The spalled areas (427 m^2) were then patched with hot mix and a 20 mm asphaltic sand mix bond breaker (884 tonnes) was specified over the existing pavement surface. The sand mix was placed to prevent bonding between the new and old PCC pavements and minimize reflection cracking in the overlay. Following placement of the bond breaker a 180 mm thick unreinforced plain jointed concrete pavement (38,200 m^2) was placed. A typical cross-section showing these details is provided in Figure 4.

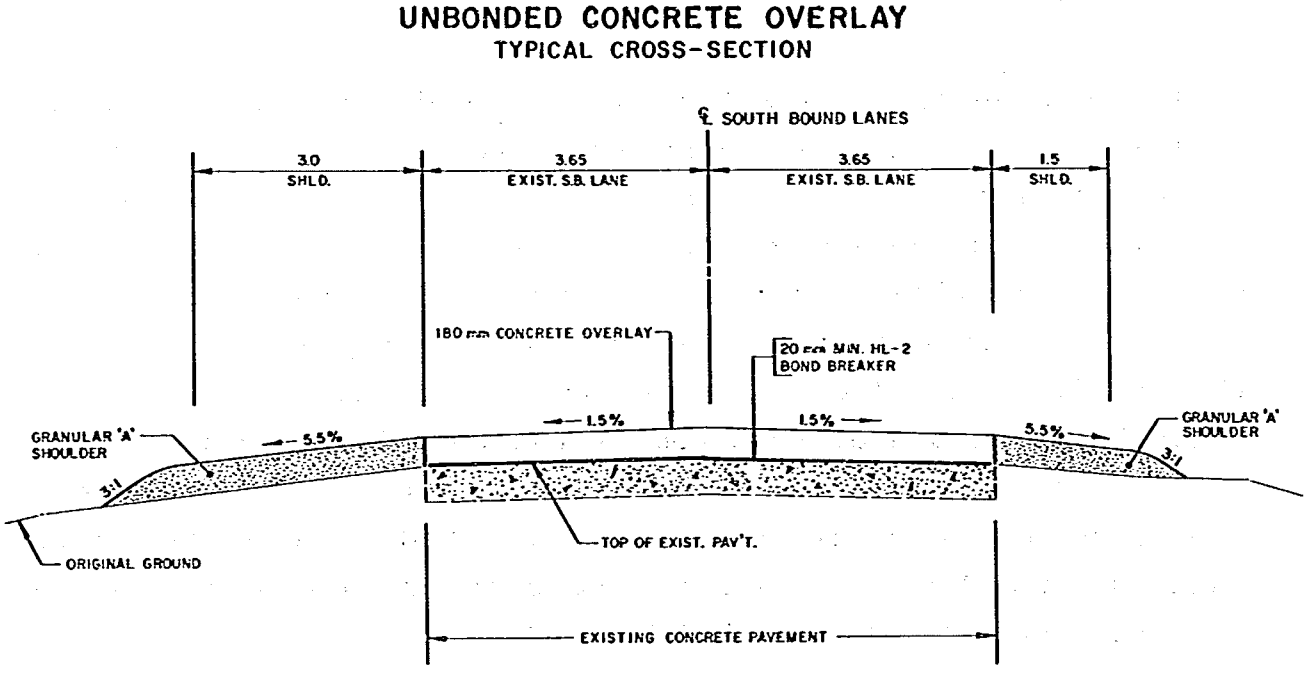


Figure 4

Slipforming of the unbonded overlay was carried out utilizing Ontario's revised specifications and drawings for concrete pavement. The revisions were based on experience gained on the experimental concrete pavement sections on Highway 3N [3, 4].

CONSTRUCTION

Generally, construction of the concrete repairs, diamond grinding and the unbonded overlay went smoothly following the initial start-up difficulties.

During construction the highway was closed to traffic which simplified construction staging. Sawcutting for full depth repairs was carried out in advance of slab removal and replacement.

Initially, during removal of the full depth concrete some spalling of the existing concrete at the sawcut edge occurred where only two sawcuts were made. The contractor then elected to make a third sawcut on one side of the repair to remove a small 150 mm wide slab prior to removal of the larger 2 m slab. This operation eliminated all spalling at the edge of the repair area. The contractor used a backhoe to remove the concrete slabs and disturbance of the underlying granular was negligible. Concrete removal was only undertaken where the placement of concrete could be completed on the same day. This minimized disturbance of the exposed granulars due to construction activity and infiltrated water.

Following removal of concrete, drilling for dowel bars was completed using a Tamrock gang drill. This system worked well and increased production significantly over the single drills used on previous contracts.

After air cleaning of the holes, the dowel bars were epoxied into place using a two part epoxy cartridge system. This method ensured that epoxy is uniformly distributed throughout the hole. The dowel bars were then inserted, rotated, and the grout was forced forward enveloping the bar. The exposed ends of the dowel bars were then coated with grease, a bond breaking agent, to allow for longitudinal movement of the pavement slabs.

Normal P.C.C., 25 MPa mix, was placed and finished flush with the existing slabs using a razor back screed. Matching of surface tolerances was not critical as the entire surface was to be diamond ground.

Spall repairs were undertaken following full depth repairs. This involved sawcutting around the spalled area to a depth of 50 mm and removing all unsound concrete. Spall repairs which were located adjacent to existing cracks or joints were removed such that small sections of sound concrete were left between repairs. These small sound areas fractured and spalled off in a day or two as the pavement expanded. In order to avoid this problem the entire joint width was repaired.

Following placement and curing of these repairs the northbound lanes were diamond ground. The grinding equipment was comprised of a grinding head mounted between the forward and rear wheels of a weighted frame. This arrangement planed the surface for a minimum depth of 1.5 mm, both retexturizing and reprofiling the concrete surface. An integral vacuum system was used to pick up the cuttings. This slurry was stored in a mobile tank and disposed of at the local sewage treatment plant.

Once diamond grinding was completed the cracks were routed, cleaned and sealed with hot poured rubberized asphalt. A short test section of silicone sealant was placed on the N.B.L. cracks.

Construction of the unbonded concrete overlay on the S.B.L. went smoothly following a few initial breakdowns on the slip-form paver. The two lanes were slipformed simultaneously with longitudinal tie bars being inserted every 600 mm on the centreline. The plastic concrete surface was dragged longitudinally with a burlap mat and tined transversely to achieve the specified surface texture. Speed change lanes were then slipformed and tied onto the outside of the core lanes where required.

No uncontrolled cracking of the pavement occurred as sawcutting was required within 12 hours after placement.

The pavement ride was somewhat better than our previous slipforming projects, which was in part due to a requirement for stringline control on both sides of the slip-form paver. Pavement ride is discussed in detail in the following section.

PAVEMENT PERFORMANCE

Roughness Measurements

The ride of a pavement is the most visible sign of pavement condition to the travelling public, therefore the collection and analysis of the ride condition or roughness data was undertaken.

Roughness was measured, just after construction and prior to opening to traffic, using the California Profilograph (S.B.L. only) and the Portable Universal Roughness Device (PURD).

The profilograph measurements were taken on the new concrete overlay with readings taken in both wheelpaths of the driving and passing lanes. The average profilograph reading, profile index, for the length of the project in the driving lane was 11.09 in/mile and 10.66 in/mile in the passing lane. These readings were taken prior to grinding 21 locations where the roadway did not meet the surface tolerance specifications of 3 mm in 3 m. This compares reasonably well with the numbers obtained by other authorities constructing considerably larger quantities of concrete pavement. Many of these agencies have performance specifications requiring a ride ranging from a high of 12 in/mile to as low as 7 in/mile with incentive bonuses for smoother pavements.

Roughness measurements using the PURD were also taken. The PURD is a trailer mounted accelerometer-based mechanical measuring device operated at constant highway speed, which measures and calculates the root mean square vertical acceleration, in milli g's, of the trailer axle as an indicator of pavement roughness. PURD measurements are converted into a ride rating using a transfer function for use in Pavement Management applications.

Both pavements were tested for roughness using the PURD prior to rehabilitation, after rehabilitation just prior to opening to traffic, and in the spring of 1990. All four traffic lanes were tested.

The ride improved significantly following rehabilitation with

a reduction in roughness from an average of 800 milli g for all lanes (R.C.R. of 5.8) to 230 milli g (R.C.R. of 9.2) for the N.B.L. and 360 milli g (R.C.R. of 7.8) for the SBL.

The eight month post construction PURD testing does not indicate any significant change in ride except on the S.B.L. where the ride has improved slightly. This is probably due to wearing of coarse projections on the tined surface. The detailed PURD measurements are provided in Figure 5 and summarized on Figure 6.

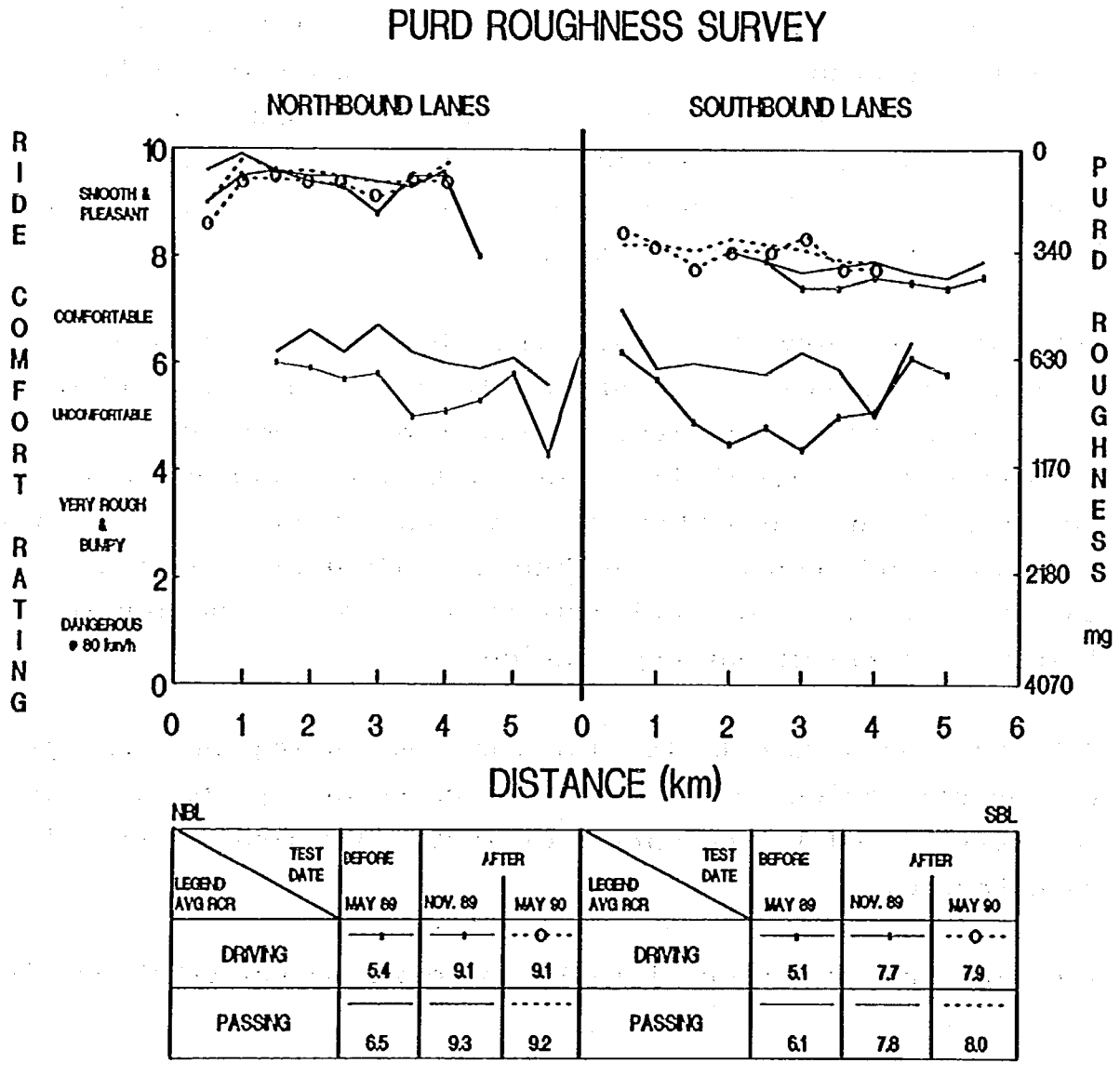


Figure 5

The roughness numbers for the diamond ground surface indicate a very smooth surface was obtained. The slip formed concrete which averages 360 milli g is somewhat rougher than the diamond ground surface, however it is only slightly rougher than the ride achieved on a new asphalt surface.

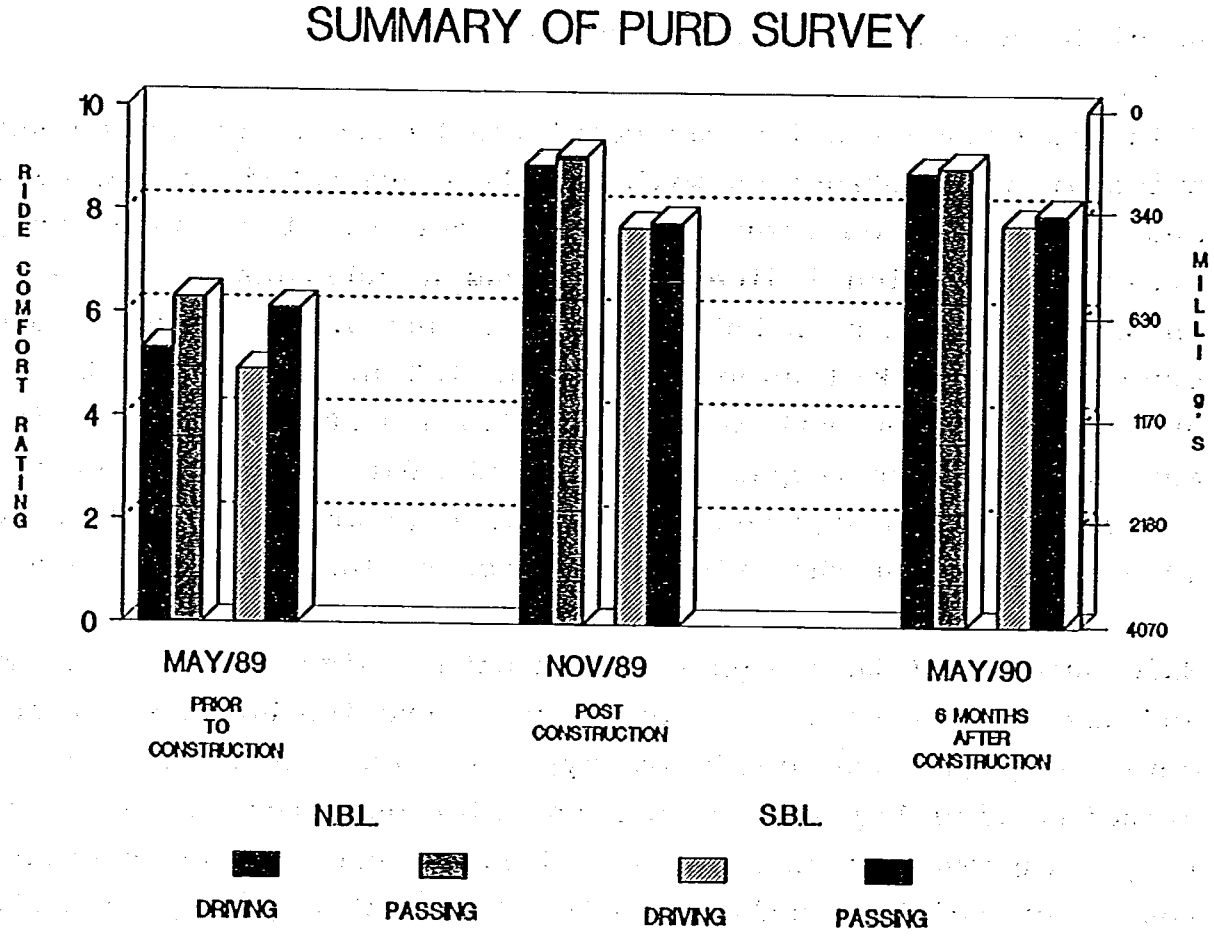


Figure 6

Skid Testing

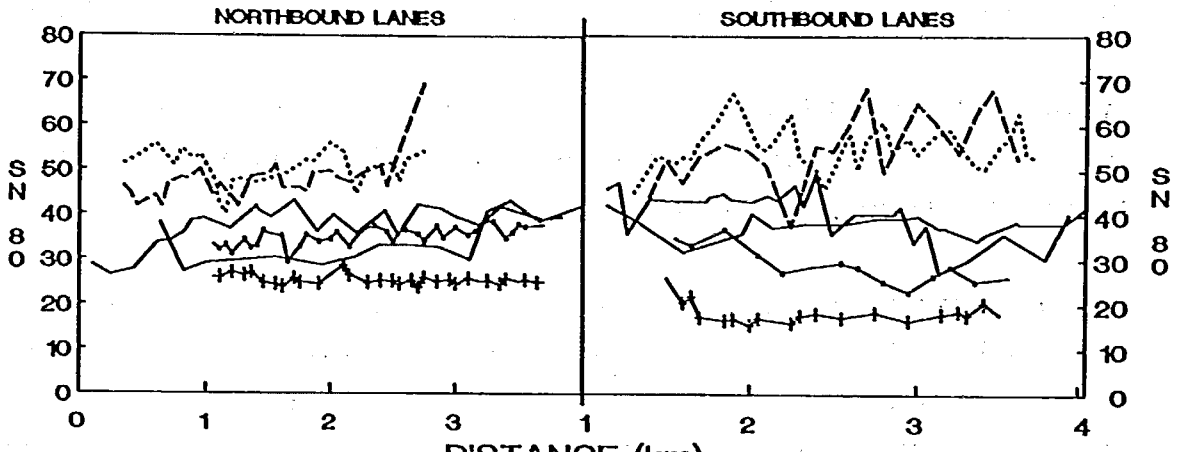
In North America the most commonly accepted method for measuring skid resistance is the skid trailer conforming to ASTM Standard E274. The test unit consists of a towing vehicle and a brake force trailer on which a wheel locks. The horizontal and vertical forces which are required to pull the locked wheel over wet pavement are measured. Water is applied to the pavement directly in front of the locked wheel. The measured friction force is described as a skid number (SN) at a specific speed, for example SN80 at 80 km/h.

Skid resistance testing was completed before construction and twice after construction with the data presented in Figure 7 and summarized on Figure 8. The test results immediately after construction indicate there was a substantial increase in skid resistance on both the N.B.L. and S.B.L. as one would expect. The skid numbers in the S.B.L. increased to an average of 55 for both lanes from 18.3 and 28.9 in the driving and passing lanes respectively. On the diamond ground surface the skid numbers improved from an average of 24.8 and 35.1 to 48.0 and 50.4 for the driving and passing lanes respectively.

Skid numbers of this magnitude indicate a high friction factor and are similar to numbers obtained on premium hot mix surface courses [5]. The corduroy type of texture developed by grinding visually indicates high frictional resistance. The ridges improve the surface macro-texture and provide an escape route for moisture under a tire. The broken peaks, concrete fractured between the saw blades, improves micro-texture.

Testing in the spring of 1990 has shown a fairly significant reduction in skid number from the initial post construction number and confirmed expectations. The 1990 skid tests provided average numbers of 37 and 40 in the passing lanes and

SKID RESISTANCE SURVEY



NBL

LEGEND / TEST DATE — AVG SKID	BEFORE MAY 89	AFTER NOV. 89	AFTER MAY 90
DRIVING	25	48	35
PASSING	35	50	37

SBL

LEGEND / TEST DATE — AVG SKID	BEFORE MAY 89	AFTER NOV. 89	AFTER MAY 90
DRIVING	18	55	38
PASSING	29	55	40

Figure 7

SUMMARY OF SKID SURVEY

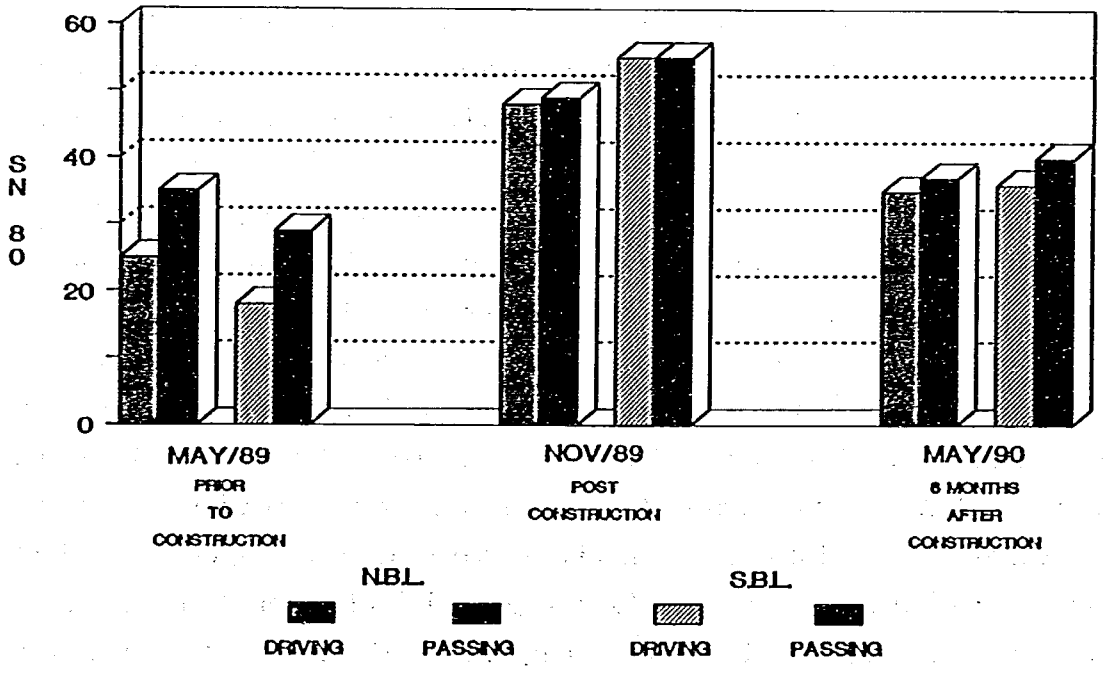


Figure 8

35 and 36 in the driving lanes for N.B. and S.B. lanes respectively. This is somewhat lower than the numbers ranging from 48 to 55 obtained in the immediate post construction tests.

Visual observations indicate that the micro texture in the wheelpaths on both the northbound and southbound lanes has undergone some polishing.

Based on previous experience, this decrease was anticipated as the initial sharp projections of both the ground and drag/tinned surfaces were worn or sheared by traffic and snow removal operations. This elimination of sharp projections reduces skid resistance, particularly in the short term. The rate of friction loss in the long term is heavily dependent upon traffic volume, abrasion of maintenance activities, and the aggregate mineralogy.

Sound Level Measurements

Sound level measurements were undertaken on Highway 126 prior to and following rehabilitation. Highway noise is becoming an ever increasing environmental concern to the public, therefore it is important to collect and maintain this information.

The procedure used to evaluate pavement noise on this project was to measure sound emissions from individual vehicle passbys at a 15 m distance from the travelled lane. The results are plotted on Figure 9 and indicate that the diamond ground surface was approximately 1 dBA quieter than the old worn concrete and the MTO reference sample. The ground surface is only 1 dBA louder than the open graded friction course which is used for its low noise and good skid resistance characteristics. The MTO sample is obtained from Reference [6] and was derived by testing several hundred vehicles on a variety of pavement surfaces across the province.

The burlap dragged and transversely tined concrete surface was approximately 1 dBA noisier than the worn concrete and MTO sample. The new concrete on the S.B.L. should become quieter as some of the very coarse tining wears off.

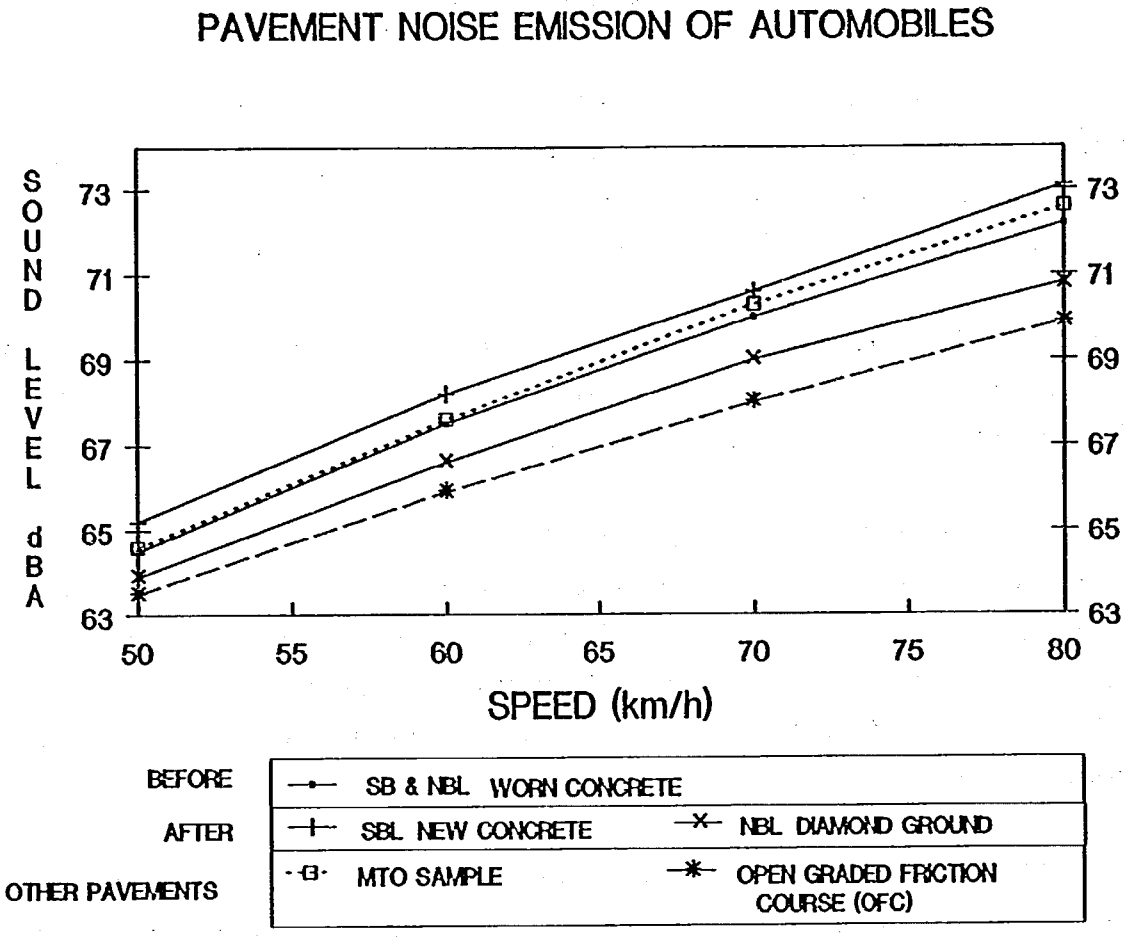


Figure 9

Load Response Characteristics

In order to compare the load/deflection characteristics and the joint load transfer efficiencies of the rehabilitated pavement a non-destructive load testing program was conducted.

Deflection testing was completed using the Dynatest 8000 falling weight deflectometer (FWD). The FWD operates by dropping a mass onto the pavement from various heights to simulate various axle loadings. A load cell is used to measure the transmitted load to the pavement and seven seismic sensors are used to measure displacement of the pavement.

The deflection measurements required to calculate load transfer for the approach and leave sides of the joint were obtained simultaneously with the corner slab measurements by "straddling" the joint using a trailing sensor.

The majority of the testing was performed early in the day to minimize the influence of slab curl resulting from differential slab temperatures.

Prior to rehabilitation, slabs were randomly selected for testing of corner to centre deflection ratios and load transfer efficiency across joints. This information was used for design to determine whether voids existed below the slabs and to establish the need for stabilization of the joints by subsealing.

Overall deflection testing indicated the existing pavement, both N.B. and S.B.L., were in a stable condition prior to rehabilitation. Load transfer across joints or cracks tested varied from a mean value of 78 to 89 percent on the northbound lanes and from 67 to 83 percent on the southbound lanes. Load transfer efficiency of 70 to 100 percent is observed in relatively new pavements and is considered acceptable [7]. These test values also indicated that voids beneath the slabs were very small or non-existent. The corner to centre deflection ratio testing on the old pavement verified acceptable slab support based on mean values ranging from 3.3 to 5.9 in the N.B.L. and from 4.4 to 4.8 in the S.B.L..

Following rehabilitation, and prior to traffic opening, deflection testing was undertaken at the same locations or at the nearest point to the preconstruction testing. This work was undertaken to evaluate the performance of the repairs and to provide a general comparison on the overall performance of the rehabilitation work.

Testing on the unbonded concrete overlay, indicated a substantial improvement in both load transfer and corner/centre deflection ratios as would be expected. The post construction load transfer testing indicated mean values ranging from 88 to 91 percent, an increase of 5 to 24 percent over mean preconstruction levels. The load transfer is also more uniform among the joints tested as evidenced by the reduction in the range and standard deviation of test values. The corner to centre deflection ratios were also substantially improved after rehabilitation with similar trends as observed for load transfer. This information is depicted graphically below in Figures 10a and 10b.

LOAD DEFLECTION RESULTS
SOUTHBOUND LANES

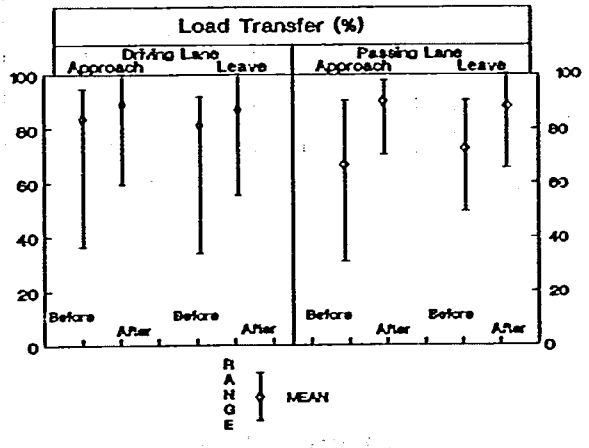


Figure 10a

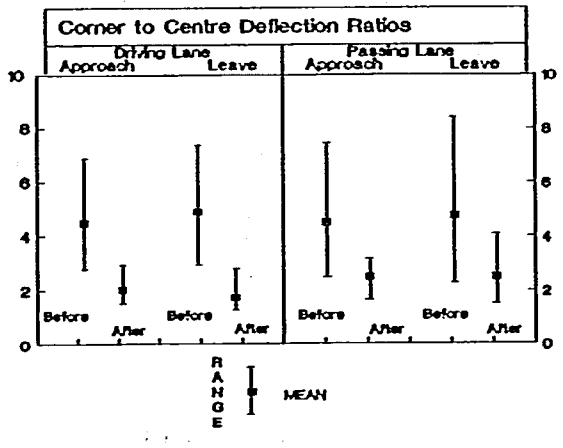


Figure 10b

On the N.B.L. post construction testing for load transfer and corner/centre slab deflection ratios were carried out on unrepaired joints and slabs as well as repaired sections. The results for the unrepaired sections were very similar to preconstruction results with some minor changes due to normal test variations.

The results of FWD testing across the full depth repair joints indicated load transfer had been reduced in areas where improvements were anticipated. A general pattern of lower load transfer values, in the order of 20 percent, was observed when loads were applied on the repair itself versus loading on the old concrete. This data is shown graphically on Figure 11a.

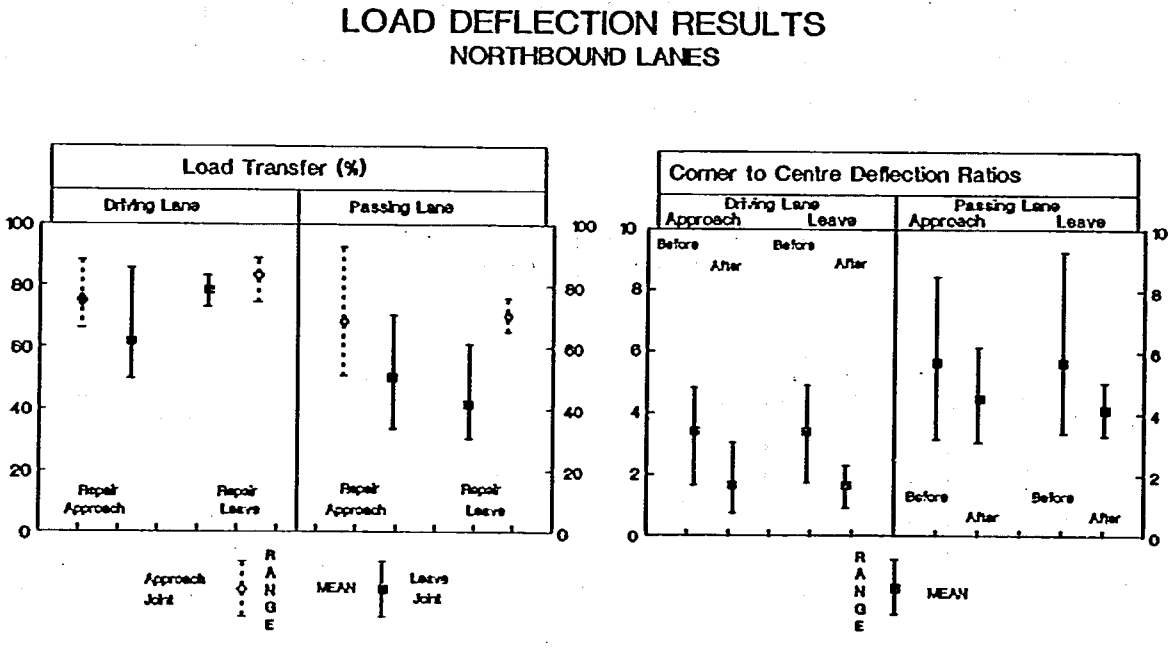


Figure 11a

Figure 11b

This pattern may be explained by surficial loosening/softening of the granular base in the repair area during full depth repair activities. These activities may create a thin loosened zone susceptible to deformation. The mechanism for reduction of load transfer is inconclusive and will require further investigation.

The use of grease as a bond breaking agent on the dowel bars in the repair has also been suggested as a possible contributor to reduced load transfer. The grease may create a small void around the dowel bar. Further investigation will also be required in this area.

Previous studies [8] have found that reduced load transfer is usually observed when the load is applied on the unrepaired concrete. This pattern was found to be a result of voids above the grouted bar in the existing concrete pavement. Results on this project are opposite to these previous studies.

Generally corner to centre slab deflection ratios were improved following rehabilitation, as anticipated. The most significant improvement was in the reduction of the range of values obtained for corner to centre deflection. This would indicate that the weakest locations were improved and the entire length of the pavement has experienced an increase in stiffness and slab support, as depicted in Figure 11b.

CONCLUSIONS

This report summarizes the successful design and construction of one of the most innovative PCC pavement rehabilitation projects undertaken in Ontario. It also provides an overview of the initial performance of these rehabilitated PCC pavements. Monitoring and data collection is continuing and future performance results will be documented.

The following general conclusions based on short term post construction results are presented.

1. A significant improvement in ride and skid resistance is attainable using diamond grinding techniques. Post construction ride and skid resistance measurements were as good as, if not better than, those values obtained with premium hot mix surface course.

2. The integrity and continuity of the overall pavement structure was improved by undertaking full depth repairs and the unbonded concrete overlay.

3. Noise level testing indicates that immediate post construction pavement noise levels are lower for the diamond ground surface when compared to pre-construction levels and marginally higher for the unbonded overlay.

4. The actual construction costs for this project were virtually identical to those obtained in estimates.

5. As an overall conclusion, the Highway 126 project has shown that full PCC pavement rehabilitation can be successfully and economically carried out in Ontario. These techniques provide an alternative methodology for addressing the rehabilitation of Ontario's aging highway infrastructure.

Several aspects of PCC pavement rehabilitation requiring further developmental work include:

i) improvements in retrofitting load transfer devices to minimize subbase disturbance for full depth repairs;

ii) improved methods of detecting and removing deteriorated concrete and replacement with economical high early strength patch materials;

iii) investigate and monitor the long term skid resistance of diamond ground pavements;

iv) develop an end result surface tolerance specification for burlap dragged/tined and diamond ground pavement surfaces in Ontario.

Acknowledgment

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Evaluation System of Airport Concrete Pavements

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EVALUATION SYSTEM OF AIRPORT CONCRETE PAVEMENTS

by

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ABSTRACT

In Japan, airport pavements have to be evaluated in two ways; namely, on surface conditions and on structural conditions. This is specified in Airport Pavement Maintenance and Rehabilitation Manual. The pavement surface conditions are specified to be evaluated using the sole index PRI (Pavement Rehabilitation Index) at civil airports in Japan as it is common that the surface conditions of the pavements are described by the sole index. The actual state of airport concrete pavements in Japan is described as periodical survey on the surface conditions has been conducted. In addition to this, the actual state of some airports in U.S.A., which is evaluated using PCI (Pavement Condition Index), is also presented.

A nondestructive structural evaluation method of concrete pavement using deflections measured by a Falling Weight Deflectometer (FWD), is described. In this procedure, the mechanical properties of a concrete slab and foundation can be obtained by backcalculating from the FWD deflections measured at the slab center. Load transfer efficiency at a joint can be quantified from the FWD deflections measured at the joint. Furthermore, a void beneath the concrete slab can be detected and its size can be estimated. Finally, some features of the structural conditions of airport concrete pavements in Japan are discussed.

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1. INTRODUCTION

The purpose of airport pavements is to let aircraft pass safely. Therefore, their conditions are required to be kept satisfactorily good when both repetitive heavy traffic loadings and repetitive severe climatic actions are applied. They will deteriorate gradually after they are open to the traffic, so subjects on maintenance and rehabilitation are considered significant. Since the budget for construction, maintenance and rehabilitation of public facilities does not increase recently, planning for future maintenance and rehabilitation works at airport pavements is also important.

This can be carried out through Pavement Management System (PMS). To develop PMS, the pavement evaluation system, which is recognized to be a main component in PMS, has to be established first of all.

In Japan, airport pavements have to be evaluated in two ways; namely, on surface conditions and on structural conditions. This is specified in Airport Pavement Maintenance and Rehabilitation Manual published in 1984.[1] It is common that the surface conditions of the pavements are described by the sole index since proposal of PSI (Present Serviceability Index).[2] As well as PSI, the pavement surface conditions are specified

to be evaluated using the sole index PRI (Pavement Rehabilitation Index) at civil airports in Japan. [3] The actual state of airport concrete pavements in Japan is described herein as periodical survey on the surface conditions has been conducted. In addition to this, the actual state of some airports in U.S.A., which is evaluated using PCI (Pavement Condition Index), is also presented.

A nondestructive evaluation system using Dynaflect has been developed for airport asphalt pavements in Japan. [4] However, a destructive testing procedure has been adopted as a structural evaluation method for airport concrete pavements in Japan. This procedure is not only costly but time-consuming. Recently, the Falling Weight Deflectometer (FWD), one of the nondestructive testing instruments for pavements, has been introduced to solve these problems. After its deployment, various kinds of works have been carried out to verify the applicability of FWD for structural evaluation of airport concrete pavement and to establish a nondestructive evaluation method using it. [5] After discussing the nondestructive structural evaluation system, the actual structural conditions of airport pavements in Japan are discussed.

2. EVALUATION METHOD OF SURFACE CONDITIONS OF AIRPORT CONCRETE PAVEMENTS

2.1 SURFACE CONDITIONS OF AIRPORT CONCRETE PAVEMENTS IN JAPAN

2.1.1 EVALUATION METHOD [3]

It is very important to quantitatively evaluate the pavement serviceability because it helps engineers to recognize the degree of pavement distress and the necessity of maintenance and rehabilitation works. An equation for evaluating the pavement serviceability was developed through comparing pavement engineers' opinions with objective values for the pavement surface conditions.

In Japan, PRI (Pavement Rehabilitation Index), which quantifies the surface conditions of airport pavements, has been developed. PRI is calculated by the use of equation (1) with measured values for the cracking index, failure ratio of joints and faulting. Facilities are divided into a lot of small units, which is 400m^2 in case of concrete pavements and then PRI should be calculated.

$$PRI = 10 - 0.290CR - 0.296JC - 0.535SV \qquad (1)$$

where, CR: cracking index(cm/m^2) defined by length of cracks/area of an unit,
JC: failure ratio of joints (%) defined by summation of the length of failed joints / total length of joints,
and
SV: faulting (mm) defined by the maximum amount in an unit.

Table 1 Criteria of PRI

PAVEMENT AREA	GRADE		
	A	B	C
RUNWAY	7.0≦	3.7≦, <7.0	<3.7
TAXIWAY	6.4≦	2.3≦, <6.4	<2.3
APRON	5.7≦	0.0≦, <5.7	<0.0

The necessity of the pavement rehabilitation is then judged on the basis of the threshold values shown in Table 1. This table shows the threshold values of PRI with respect to several levels of the necessity of rehabilitation. These levels are as follows;

A: rehabilitation works are not necessary

B:rehabilitation works are needed in the near future
C:rehabilitation works are needed immediately

2.1.2 ACTUAL STATE OF SURFACE CONDITIONS [6)]

Periodical survey on the pavement conditions has been carried out since 1979. Table 2 summarizes the results. Concrete pavements have been constructed mainly in the specified areas, namely aprons and taxiways in Japan. The surface conditions are kept good irrespective of types of the facilities. Table 3 shows the necessity of the maintenance and rehabilitation works according to Table 1. Some areas in taxiways are rated as C although more than 70 % are ranked as A in both taxiways and aprons.

Table 2 Surface Conditions of Airport Concrete Pavement

PAVEMENT TYPE	INVESTGATION AREA ($1000m^2$)	"D" CRACKING (%)	LINEAR CRACK		PATCHING (%)	CRACKING INDEX (cm/m^2)	FAILURE RATIO OF JOINTS (%)	MAXIMUM FAULTING (mm)	PRI
			LONGITUDINAL (%)	TRANVERSE (%)					
TOTAL	259	0.39	0.91	0.07	0.16	1.38	0.47	5.43	6.56
APRON	120	0.33	0.16	0.09	0.18	0.58	0.51	5.87	6.54
RUNWAY	9	0.00	0.00	0.01	0.00	0.01	0.77	5.89	6.62
TAXIWAY	130	0.48	1.67	0.06	0.16	2.21	0.40	5.00	6.57

Table 3 Evaluation of Airport Concrete Pavements by PRI

FACILITY NAME		TOTAL	A	B	C
RUNWAY	NUMBER	1	1	0	0
	(%)	100	100	0	0
TAXIWAY	NUMBER	35	24	7	4
	(%)	100	68.6	20.0	11.4
APRON	NUMBER	39	30	9	—
	(%)	100	76.9	23.1	—

2.2 SURFACE CONDITIONS OF AIRPORT CONCRETE PAVEMENTS IN U.S.A.

2.2.1 EVALUATION METHOD

Conditions of airport pavements in U.S.A. have been surveyed periodically. [7] In the survey, severity and density of many types of distress, which are shown in Table 4, are determined, and then PCI (Pavement Condition Index) is calculated based on these. The severity is classified into three ranks; namely, High, Medium and Light. The density is expressed by the percentage of the distressed concrete slabs to all concrete slabs in an unit, which consists of twenty slabs.

Table 4 Distress Types in Calculating PCI

SYMBOL	EXPLANATION
BU	BLOW-UP
CB	CORNER BREAK
LT	LONGITUDINAL, TRANSVERSE AND DIAGONAL CRACKING
DC	"D" CRACKING
JD	JOINT SEAL DAMAGE
P	SMALL PATCHING (<5ft^2) (<0.46m^2)
PU	LARGE PATCHING AND UTILITY CUT (>5ft^2) (>0.46m^2)
PO	POPOUTS
PM	PUMPING
SM	SCALING, MAP CRACKING AND CRAZING
SE	SETTLEMENT AND FAULT
SS	SHATTERED SLAB
SR	SHRINKAGE CRACK
JS	JOINT SPALLING
CS	CORNER SPALLING

PCI ranges between 0 and 100 (0:at the minimum and 100:at the maximum). According to PCI, the pavement condition is classified into seven ranks and the type of maintenance and rehabilitation works which should be adopted can be judged.

2.2.2 ACTUAL STATE OF SURFACE CONDITIONS [6]

By analyzing the results of the current surveys in 36 airfields, information of great interest is obtained. In contrast

with the asphalt pavements, the conditions of airport concrete pavements vary widely from Failed (PCI<10) to Excellent (PCI>85) as shown Fig.1. Many types of distress have been observed in the concrete pavements. Major types of distress are, in the order of their densities, Joint seal Damage, Longitudinal/Transverse/Diagonal cracking, Scaling/Map cracking/crazing and Joint Spalling. These distress types are over 80% of all distress in density (Fig.2).

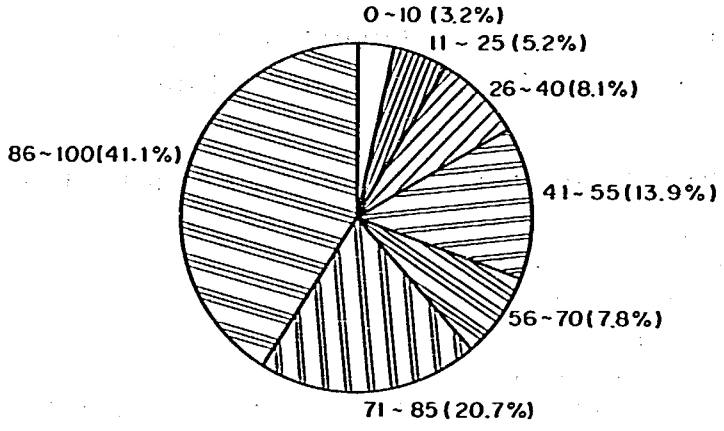


Fig.1 Evaluation of Airport Concrete Pavements by PCI

Fig.3 shows that the distressed area is widest in the case that PCI is between 25 and 40 and it decreases gradually with both increase and decrease of PCI. The density of the distress with high severity is found to increase with a decrease of PCI. The distress severity is also found to increase with an decrease of PCI.

Performance data of pavements in 88 civil airports in four states have been collected. Using these data, some statistical analyses on the relationships between PCI and service year are conducted. Fig. 4 shows the relationship of PCI to the services year for airport concrete pavements. When a linear regression line is applied, a coefficient of correlation is relatively high. These data plotted in Fig.4 do not show PCI at the specified points; namely, they show the surface conditions at various points. Therefore, PCI seems to spread relatively wider.

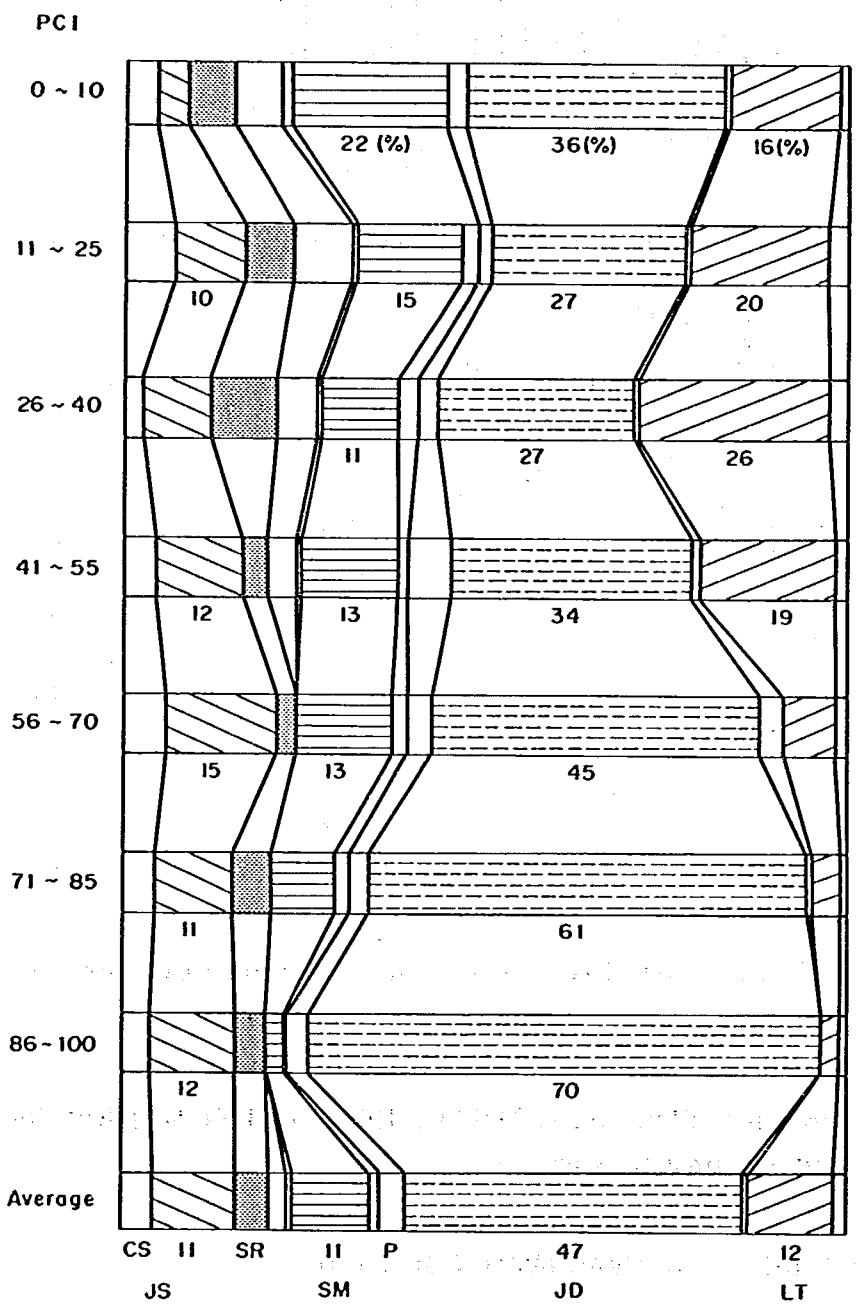


Fig.2 Distress Types on Airport Concrete Pavements

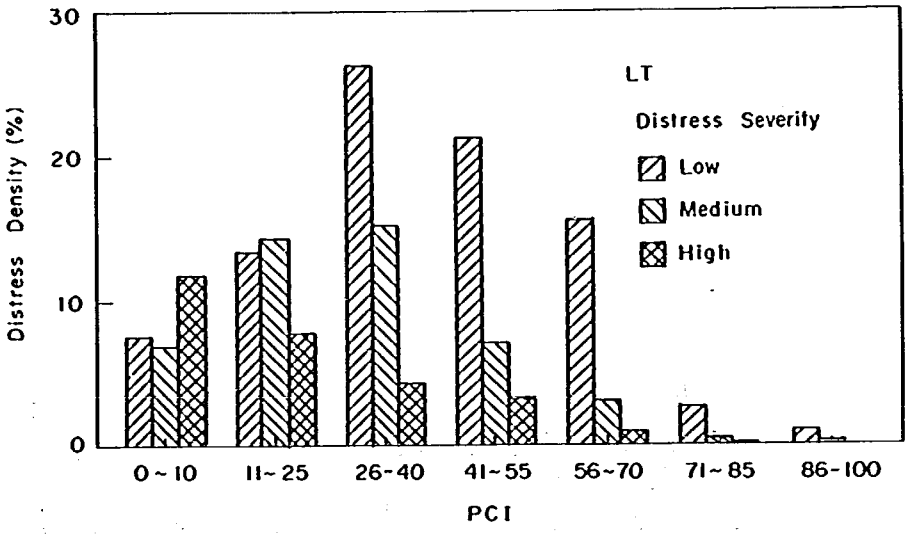


Fig.3 Relationship between PCI and Distress

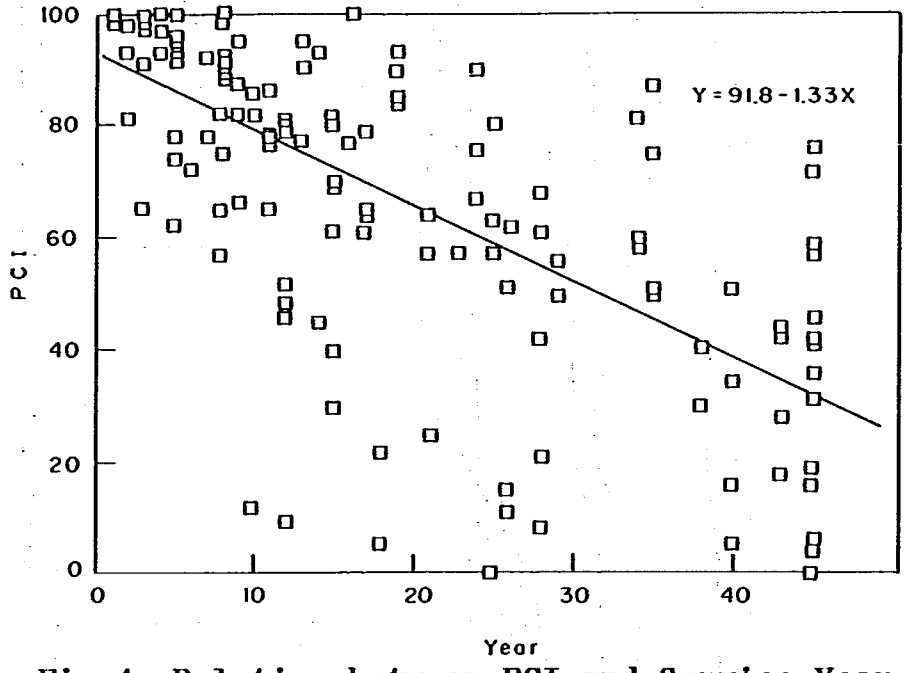


Fig.4 Relation between PCI and Service Year

3. NONDESTRUCTIVE STRUCTURAL EVALUATION METHOD OF AIRPORT CONCRETE PAVEMENTS

3.1 NONDESTRUCTIVE EVALUATION METHOD [5)]

The following nondestructive evaluation method of airport concrete pavement using Falling Weight Deflectometer (FWD) is proposed. Fig.5 is the flow chart.

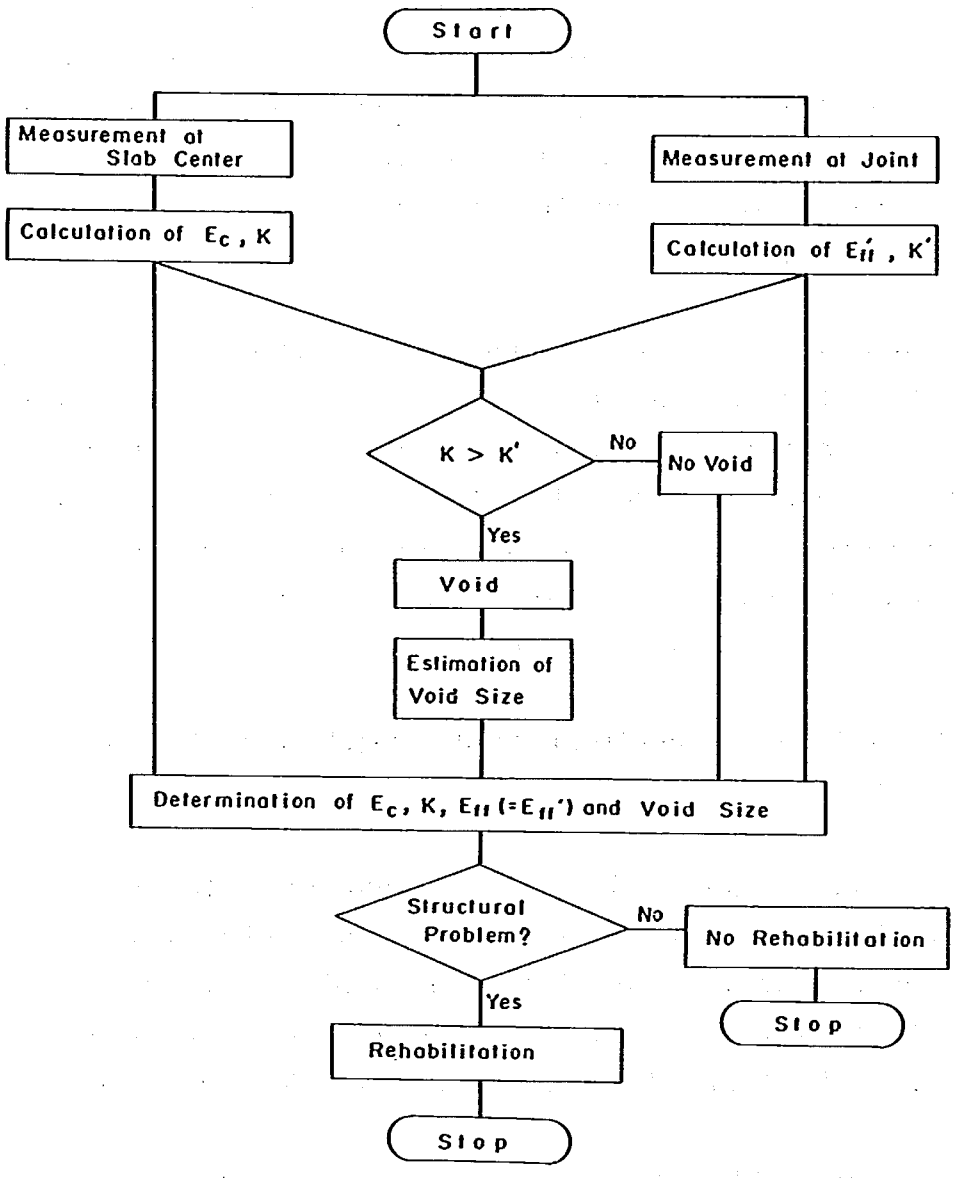


Fig.5 Flow Chart of Non-Destructive Evaluation Method

This method consists of three items; namely,

a) An elastic modulus of concrete(Ec) and a modulus of foundation reaction (K) are obtained from the FWD deflections measured at the slab center

b) A load transfer efficiency at joint is quantified using the deflections measured at joint

c) A void beneath concrete slab is detected using the deflections at joint

It is necessary to choose the points where can be assumed representative in the area in consideration of the pavement structure, service year, and so on. Both Airport Pavement Data Bank System which has been functioning since 1982 and Airport Facility Ledgers will be useful at that time. [8] Furthermore, lotting system used in surface condition evaluation will be also helpful. [1]

It is especially important to select the structurally sound points distributed over the area in measuring the deflections at the slab center. The points which are expected that there is no void beneath the concrete slab have been included when the FWD deflections at joints are obtained. In this structural evaluation system, the load of FWD within 5000 + 50 kgf is specified as the standard load.

3.2 ESTIMATION OF MECHANICAL PROPERTIES OF CONCRETE SLAB AND FOUNDATION

First of all, Ec and K have to be backcalculated from deflections obtained by FWD. Westergaard's equation for the pavement deflection in the case of interior-loading was used to do this because Westergaard's equation for the stress of the concrete slab is used in Airport Pavement Design Manual in Japan. [9]

As the factors affecting the FWD deflections at the slab center, cracks in the concrete slab and permanent deformation of foundation should be taken into consideration. The decrease of Ec, K will be caused by the former, the latter, respectively. However, the lower K will be obtained in both cases for Ec is assumed constant in this method. Therefore, a comparison to the modulus measured at the adjoining points is essential to find the cause when the lower K is obtained. The cracks should be suspected if K' estimated from the FWD deflections at joint is relatively large. On the contrary, the void may be caused if K' is small as well as K.

3.3 QUANTIFICATION OF EFFICIENCY OF LOAD TRANSFER AT JOINT

Load transfer efficiency at joint has to be determined in evaluating the airport concrete pavement structurally since the load transfer function is assumed to be sufficient in the design manual. Furthermore, the load transfer efficiency at joint is necessary to determine the overlay thickness according to the maintenance and rehabilitation manual.

The validity of the efficiency of load transfer at joint defined by equation (2) was verified. [10]

$$Eff = \frac{d_u}{(d_l + d_u)/2} \times 100 \qquad (2)$$

where, d_u is the deflection in the vicinity of a joint on a loaded slab and unloaded slab, respectively, which are obtained by plate loading test.

Eff cannot be evaluated using FWD directly because the points where deflections obtained by FWD are not the same as the points where deflections are used in equation (2); that is, the distance from joint is 15cm in case of FWD as shown in Fig.6. Thus the efficiency of load transfer at joint has to be quantified by another way, and now Eff', which is defined by the following equation, has been introduced.

$$Eff' = \frac{d_{12}}{(d_{11}+d_{12})/2} \times 100 \qquad (3)$$

where, d_{11}, d_{12} is the FWD deflection on the loaded and unloaded slab, respectively (shown as Loading 1 in Fig.6).

The FWD measurements have to be conducted at both sides of joint to increase the accuracy of estimation. Eff' can be recognized equal to Eff practically because a coefficient of

correlation between them is found to be 0.99 through numerical analysis.

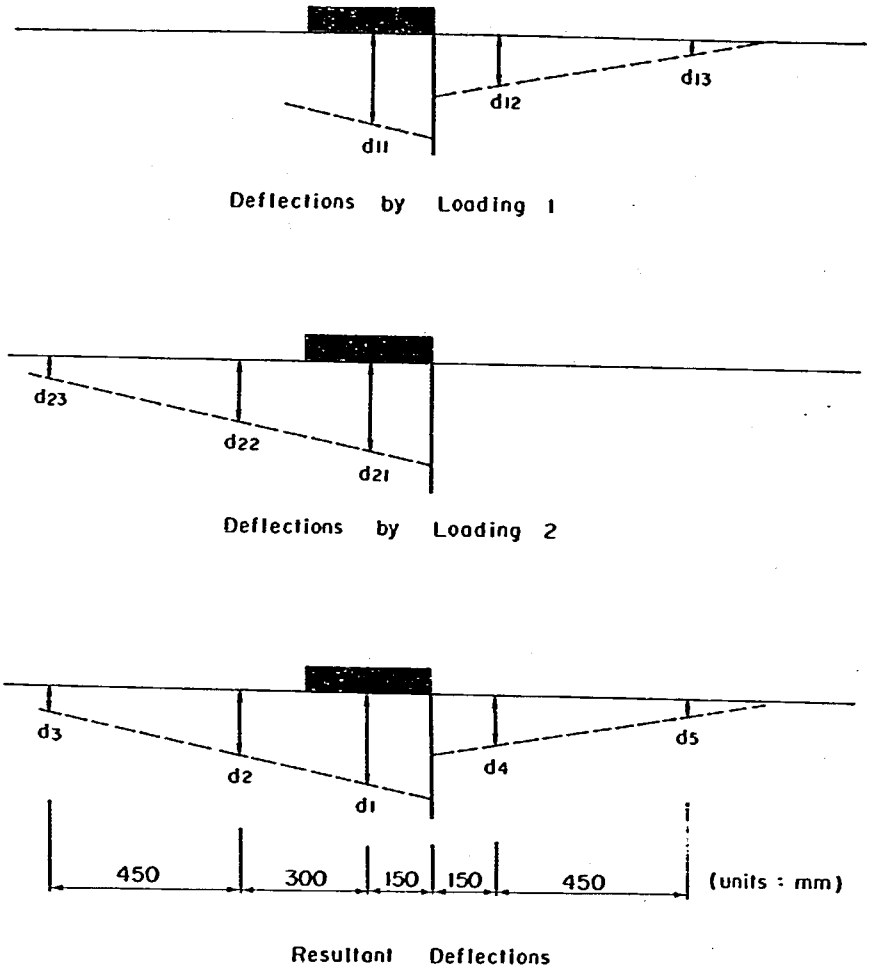


Fig.6 FWD Measurement Method at Joints

3.4 ESTIMATION OF VOID BENEATH CONCRETE SLAB

In this procedure, Ec and K obtained from the FWD deflections at the slab center are adopted, and Eff' is calculated by equation (3) using the deflections obtained according to Loading 1 in Fig.6. Then, an apparent modulus of foundation reaction (K'), which is considered as the modulus of foundation reaction backcalculated from the deflections measured at the joint on the assumption that there is no void, has to be obtained. After obtaining K' by backcalculating from those five deflections with reference to Ec and Eff', the presence of void can be judged in comparison of K and K'. The void should be suspected when K' is smaller than K.

4. APPLICATION OF FWD TO STRUCTURAL EVALUATION FOR AIRPORT CONCRETE PAVEMENTS

4.1 EVALUATION OF Ec AND K

There are a lot of projects conducted to ensure the applicability of FWD to structural evaluation of asphalt pavements. In contrast with this, the number of papers dealing with structural evaluation of concrete pavement using FWD is relatively small until now.

Some problems on deflection measurement by FWD should be solved before the applicability of FWD to the structural evaluation of airport concrete pavements is investigated. They are the reliability of measured deflections, the magnitude of the impact load, an effect of temperature in concrete pavement on the FWD deflections, and so on.

Fig.7, 8 shows the FWD deflections measured at Fukuoka Airport, Nagasaki Airport, respectively. These were obtained at nighttime in winter. Drop height of mass was kept to be 18.5 cm. It is seen that pavement thickness and loading position have influence on the FWD deflections. The magnitude of the impact load fluctuated even if the drop height is constant.

Bohn et al. [11] show the influence of the pavement structure on the load magnitude. They concluded that the load magnitude decreased by 4 - 5 percent when d_1, which is a deflection at the center of loading plate, increases up to 2 - 3 mm although the load was not affected by the pavement structure when d_1 is up to 1.3mm. In case of airport concrete pavements, neither the pavement structure nor the loading position has influence on the FWD deflections because such relatively large deflections have not been generally obtained. This is also verified analytically using mathematical models.

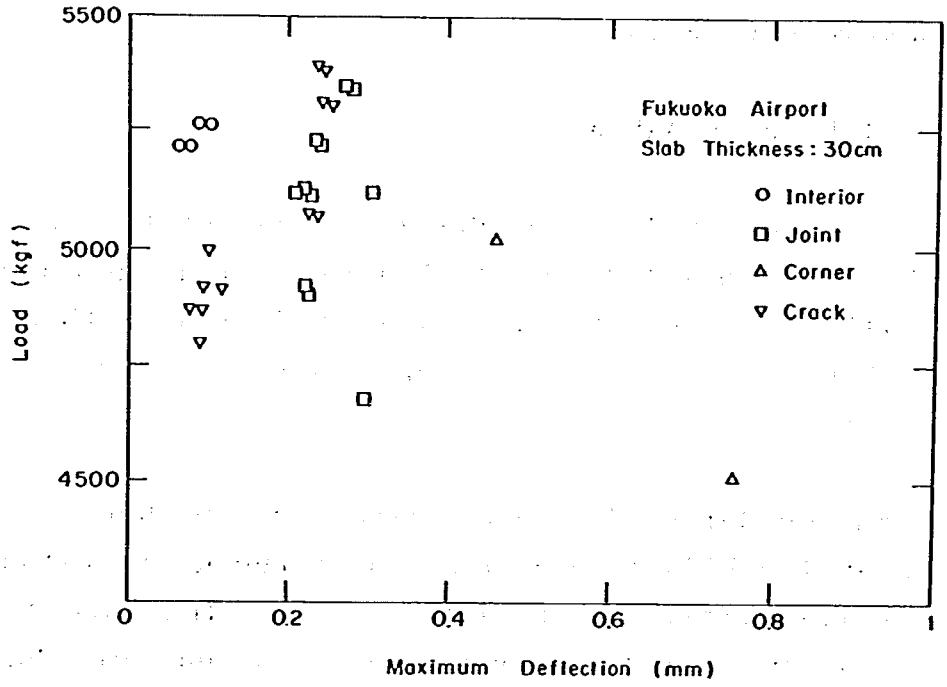


Fig.7 Deflections at Fukuoka Airport

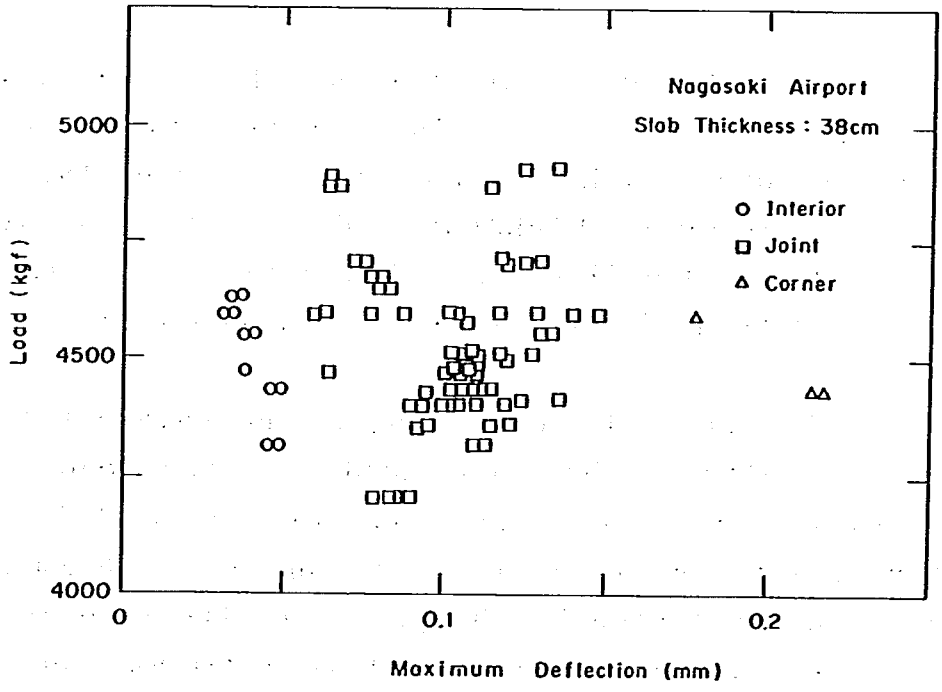


Fig.8 Deflections at Nagasaki Airport

However, the magnitude of the load fluctuates more or less irrespective of constant drop height, so it is specified that the load with the magnitude of 5000 + 50 kgf is ought to be applied to the pavements by adjusting the drop height.

Fig.9 shows the maximum deflections measured on test pavement. The FWD measurement has been conducted every two hours in a whole day. It is apparent that the deflections vary with time. It is expected that a modulus of foundation reaction changes with time as an elastic modulus of concrete is not affected by temperature in the concrete slab. It will be caused by warping of the concrete slab resulted from temperature variation in the concrete slab.

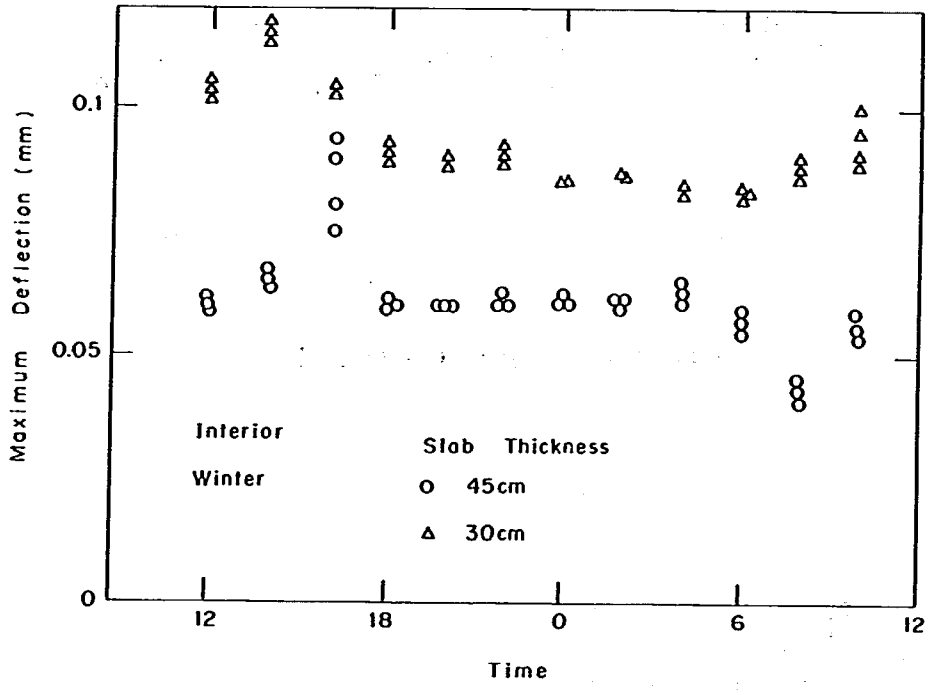


Fig.9 Changes of Deflections with Time

The concrete slab warps due to temperature variation in the vertical direction of the concrete slab. Fig.10, 11 shows Ec, K, respectively, which are backcalculated from 24-hr continuous measurement at test pavement. It is found that backcalculated Ec, which is not fixed in this calculation, is constant regardless of time in a day different from K. The backcalculated Ec seems to be smaller than that measured by laboratory compression test, but it may be explained by various factors such as the difference in the curing process of concrete.

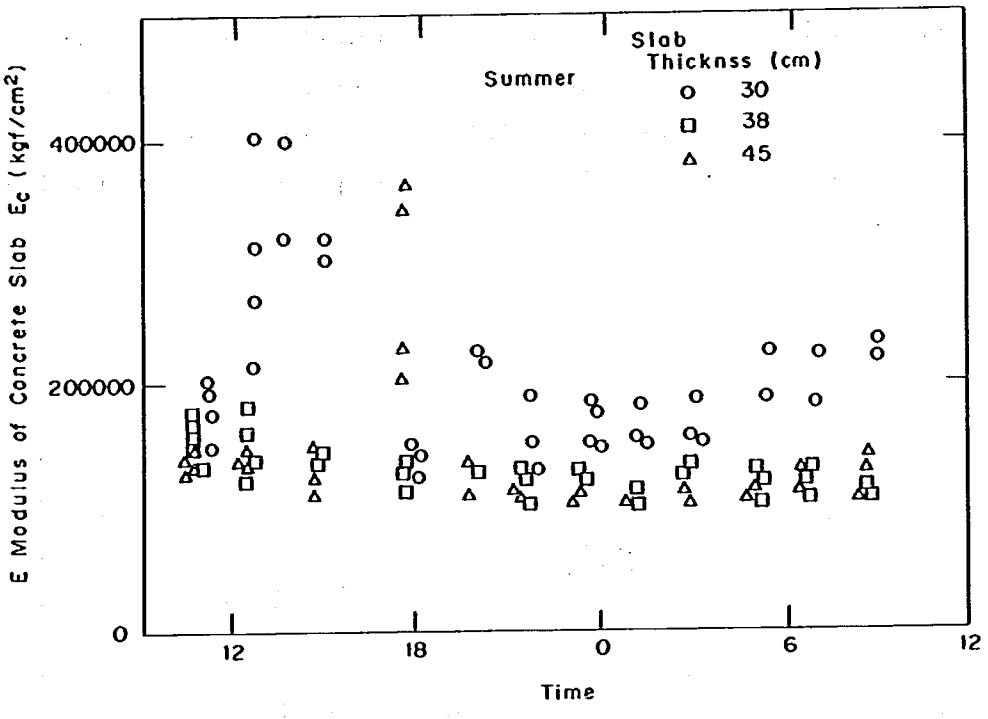


Fig.10 Backcalculated Ec

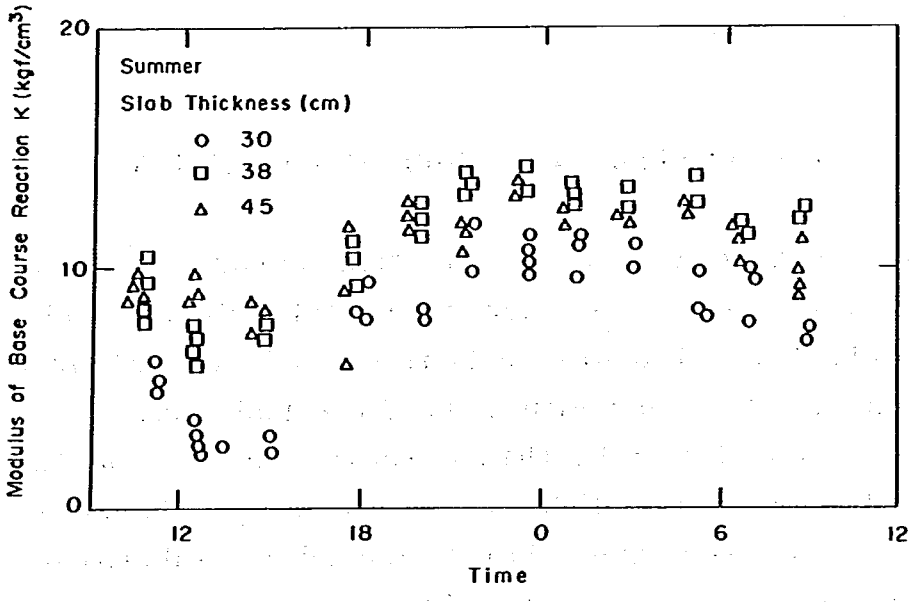


Fig.11 Backcalculated K

On the other hand, it is clear that backcalculated K varies with the time of day; that is, estimated K looks larger than measured K in the nighttime, and conversely, it seems smaller in the daytime. In view of this, the FWD deflections are surely affected by the warping of the concrete slab occurring from the vertical variation of temperature in it. However, the back calculated K coincides well with K measured on a crushed stone base which is 7.8kgf/cm^3 on an average. The concrete slab were placed on 4 cm thick asphalt stabilized layer over the crushed stone base.

4.2 QUANTIFICATION OF Eff

In the case of jointed concrete pavements at airports in Japan, joints have to be constructed at intervals of 4.5 - 7.5 m. The occurrence of cracking is often reported although load transfer devices such as dowel bars are generally installed.

In addition to the jointed concrete pavement, two types of concrete pavements without joints having load transfer devices have been constructed at airports in Japan. One is continuous reinforce concrete pavement (CRCP) in which cracks are closed tightly by reinforcing steel bars. The other is roller compacted concrete pavement (RCCP) in which dummy joints are constructed as installation of load transfer device is difficult. Therefore, the load transfer both at cracks in CRCP and at joints in RCCP depends on interlocking of aggregates.

Fig.12 shows the relationship between crack width and effectiveness of load transfer. It is found that the load transfer efficiency at cracks is more than 90 percent even if crack width is about 1 mm.

The load transfer efficiency at joints and cracks has been measured in Fukuoka airport as shown in Fig.13. Joints have much better load transfer function than cracks although both show good load transfer efficiency.

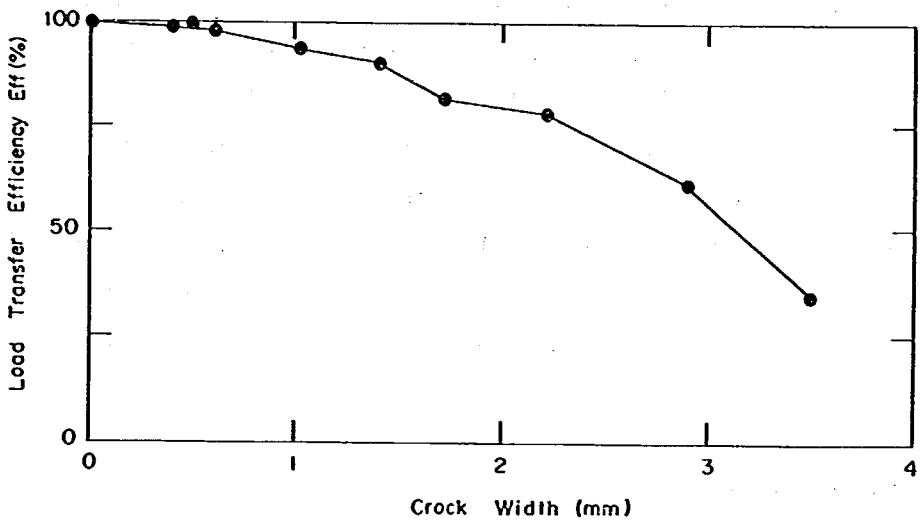


Fig.12 Relationship between Crack Width and Load Transfer Efficiency

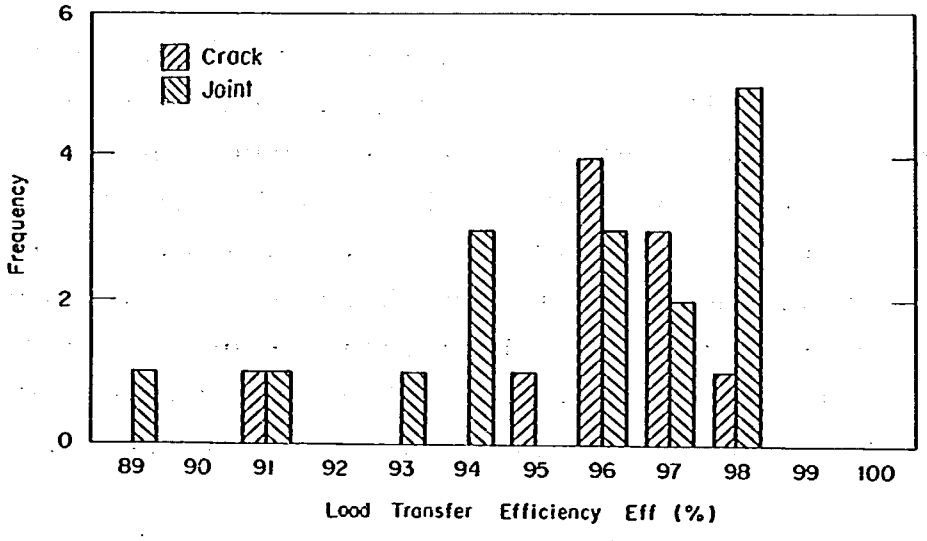


Fig.13 Measured Eff at Fukuoka Airport

Fig.14 shows measured load transfer efficiency at joints in jointed concrete pavements and at cracks which are dummy joints in RCCP at Tokyo International Airport. The load transfer efficiency is considered satisfactory at the joints, but it is insufficient at the cracks. It is supposed that joints in RCCP have a serious problem in load transfer.

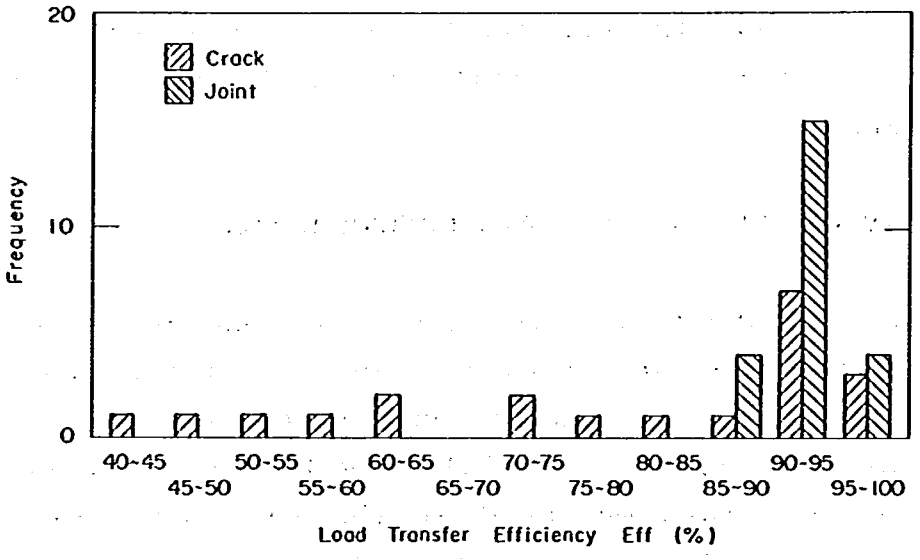


Fig.14 Measured Eff at Tokyo International Airport

Fig.15 shows the FWD deflections in at CRCP in New Tokyo International Airport. A loading cart with a main gear of B-747 aircraft have run 10000 times on this pavement. The load transfer efficiency at cracks is maintained good in spite of repeated loading.

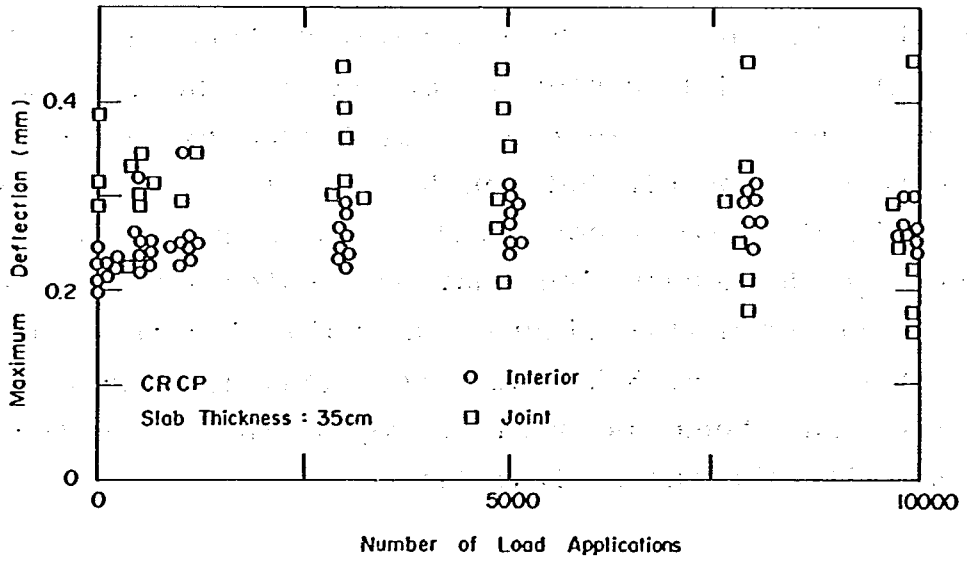


Fig.15 Influence of Load Applications on Deflections

The load transfer efficiency at cracks decrease gradually with increase of crack width although it is good as far as the crack is closed tightly. The durability of load transfer at cracks is verified in case of that crack width is kept approximately below 1 mm.

4.3 ESTIMATION OF VOID BENEATH CONCRETE SLAB

As mentioned above, a void beneath a concrete slab seems to be caused from the warping of the concrete slab. In addition to this, another type of void is possibly caused at the joint due to loss of base course material. Fig.16 shows the maximum FWD deflection at joints measured at the test pavement. This may suggest that a void can be detected and its size can also be estimated by the FWD deflection. In order to determine void size, if present, from the FWD deflections measured at a joint, the special FWD measurement method shown in Fig.6 was used; FWD was placed in the vicinity of a joint to obtain deflections d_{11}, d_{12} and d_{13} (Loading 1), and then, FWD was turned to obtain the deflections d_{21}, d_{22} and d_{23} (Loading 2). As resultant deflections, five deflections were obtained.

An apparent modulus of foundation reaction (K') has been introduced as mentioned above. If the void exists, K' will be smaller than K, which is backcalculated from the deflections measured at the slab center according to the aforementioned procedure. The void size can be estimated by the use of the theoretical relationship between K' and the void size such as shown in Fig.17. This theoretical relationship is obtained by using FEM on the assumption that the voids exists under the concrete slab. From this figure, the void size is estimated as 25 cm when K', K is 6, 8 kgf/cm^3, respectively. The validity of this method has been ascertained since the estimated values correspond with the theoretical line.

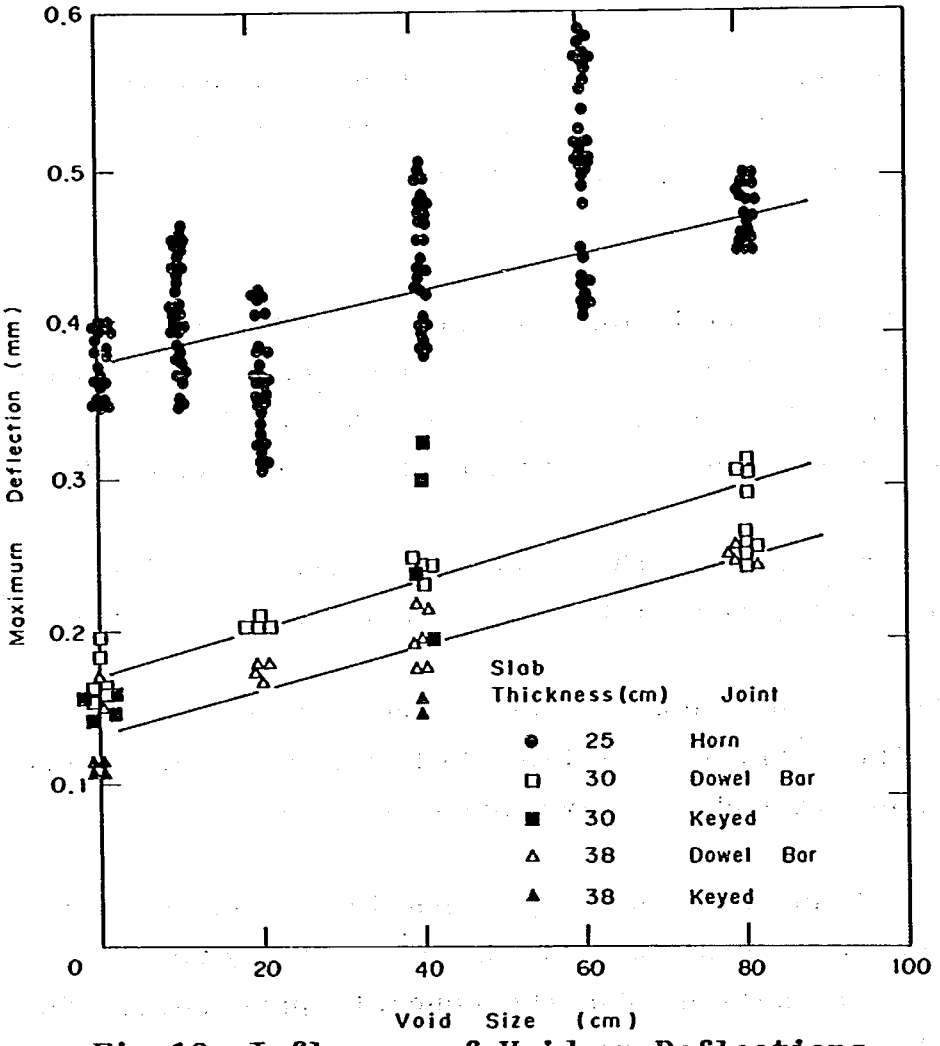


Fig.16 Influence of Void on Deflections

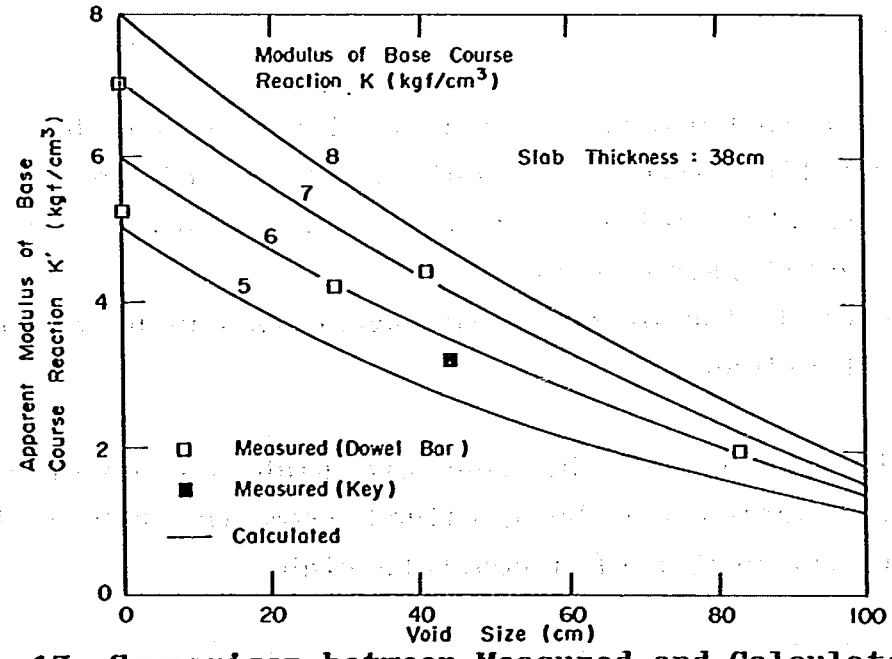


Fig.17 Comparison between Measured and Calculated K'

5. CONCLUSIONS

Airport Pavement Data Bank System has been developed to manage any kinds of information on airport pavements. Actual condition of the concrete pavements was analyzed using data stored in this system.

(1) The surface conditions of the concrete pavements are rated good because most of them are evaluated as that there is no need of rehabilitation. There seems no difference in types of facilities where the concrete pavements were constructed.

From the analyses on the surface conditions of airport pavements in U.S.A., the following findings are obtained.

(1) Surface conditions of the concrete pavements are kept good although they vary from Failed to Excellent. The concrete pavement in runways and taxiways are poorer than those in aprons.

(2) The dominant distress types are joint seal damage, longitudinal/transverse/diagonal cracking, scaling/map cracking/crazing and joint spalling for the concrete pavements. Both densities and severity of major distress types increase as PCI decreases.

(3) Relationship between PCI and service year can be expressed as a linear line with an acceptable level of coefficient of correlation. PCI varies with time most slowly for concrete pavements in all pavement types.
Structural condition of concrete pavements can be evaluated by the FWD deflections.

(1) Elastic modulus of the concrete slab and modulus of the foundation reaction can be obtained from the FWD deflections measured at the center of a concrete slab.

(2) The load transfer efficiency at joint can be quantified

using the deflections measured at joint.

(3) A void beneath the concrete slab can be detected and its size can be estimated analytically.

(4) It is also found from the collected FWD data that the warping of a concrete slab cannot be neglected. To minimize its influence regarding deflections, the FWD measurement at a slab center, at a joint, should be made during the nighttime and daytime, respectively.

Both surface conditions and structural conditions can be evaluated through the above-mentioned system. Researches on the reasonable maintenance and rehabilitation system for airport pavements, which is recognized as a next step of the evaluation system, has been just started. Therefore, a lot of items including prediction of the future pavement condition and an economic analysis should be made clear for the future.

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Rapid Monitoring of Flexible Pavement Deflections, Moduli and Roughness

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RAPID MONITORING OF FLEXIBLE PAVEMENT DEFLECTIONS, MODULI AND ROUGHNESS

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ABSTRACT

The appropriate selection of maintenance methods or rehabilitation overlay requirements, regardless of the pavement management system selected, depends on the reliable and cost effective determination of asphalt pavement performance (functional and/or structural) factors such as surface condition, roughness, deflections and moduli. The evaluation approach outlined involves the completion of a systematic pavement condition assessment (PAVER based parameters such as crack types and density) and localized roughness monitoring using a portable digital incremental profiler (Dipstick®), while the deflection/moduli parameters are automatically measured with a falling weight deflectometer (Dynatest 8000 FWD). To cover large flexible pavement networks (and rigid pavements if desired) and the time-dependent (load replications) degradation of pavements, the concept of prototype sections representative of the overall network, with regular Spring and Fall monitoring, is introduced.

1. INTRODUCTION

The serviceability of a pavement is generally based on its functional and structural condition. The functional condition of a pavement is its ability to carry out its function of providing a platform on which vehicles will be able to travel with reasonable comfort, and is generally evaluated by analyzing the type and severity of the pavement surface distresses along with the roughness and frictional properties of the pavement surface. The structural condition of the pavement is determined by correlating the response of the pavement under traffic loading to the ability of the pavement layers and subgrade to support the traffic. It is important that the environmental and loading related, time-dependent reduction in pavement component moduli (subgrade, granular layers and asphalt concrete layers) as reflected by increasing deflection/curvature (i.e. tensile fatigue strain at bottom of asphalt layers and vertical compressive strain at the unbound layer interfaces) is properly accounted for. The functional and structural condition of pavements are related in that, as the functional condition of a pavement deteriorates and the pavement surface begins to crack, moisture ingress can cause further cracking of the asphalt concrete surface and weakening of the granular base, subbase and subgrade layers.

2. FUNCTIONAL EVALUATION

The functional evaluation of a pavement may be of secondary importance for pavements having inadequate or marginal structural capacity. As the structural condition of the pavement deteriorates, the functional condition will also rapidly deteriorate as manifested by pavement surface distress and roughness. A pavement that is sound structurally will also deteriorate with age due to environment, construction or material related deficiencies.

2.1 Pavement Distress Survey

In order to rapidly record any pavement distresses, a relatively simple pavement condition evaluation form has been developed. The pavement distresses are categorizd according to the APWA PAVER System [1]. The pavement distress information is collected using a 'windshield' survey

approach. Distresses are qualified by type and severity and quantified in terms of the frequency of their occurrence. The type, severity and density of the distresses in a section are further reduced to a single pavement condition number on a scale of 1 through 10 with 10 representing a 'perfect' pavement. A sample pavement condition evaluation form is given in Figure 1.

General rehabilitation action warrants have been developed for corrective action as shown in Table 1.

TABLE 1 GENERAL PAVEMENT REHABILITATION WARRANTS BASED ON PAVEMENT CONDITION RATING

Pavement Condition Rating	General Rehabilitation Action
8-10	Minor Maintenance; Crack Sealing, etc.
6-8	General Maintenance; Localized Repairs, etc.
4-6	Major Maintenance; Overlay, Partial Reconstruction, Recycling, etc.
0-4	Reconstruction

2.2 Pavement Roughness

The characterization of pavement roughness is a very complex problem due to the differences in dynamic response of each vehicle traversing the pavement. Pavement roughness data must be reduced, through mathematical interpretation and simplifying assumptions, to meaningful statistics which relate roughness to serviceability. The devices that have been developed to quantify pavement roughness can generally be divided into those which directly measure the pavement profile and the response of the test vehicle to movements over the pavement surface. Many recent

PAVEMENT CONDITION EVALUATION FORM

SUBDIVISION _____ FILE _____

ROADWAY _____

LENGTH _____ SURFACE _____ CURB HEIGHT _____

WIDTH _____ BINDER _____ CURB CONDITION _____

YEAR CONSTRUCTED _____ BASE _____ DATE _____

TRAFFIC VOLUME _____ SUBBASE _____ SURVEY BY _____

BUS ROUTE _____ SUBGRADE _____

RIDING COMFORT RATING	EXCELLENT			GOOD		FAIR		POOR		VERY POOR
SPEED (kph) _____	9		8	7	6	5	4	3	2	1

PAVEMENT DISTRESS MANIFESTATIONS			SEVERITY OF PAVEMENT DISTRESS					DENSITY OF PAVEMENT DISTRESS					COMMENTS
			VERY SLIGHT	SLIGHT	MODERATE	SEVERE	VERY SEVERE	FEW	INTERMITTENT	FREQUENT	EXTENSIVE	THROUGHOUT	
								< 10%	10-20 %	20-50 %	50-80 %	80-100 %	
			1	2	3	4	5	6	7	8	9	10	
LOAD RELATED DISTRESS	ALLIGATOR CRACKING	1											
	EDGE CRACKING	2											
	RUTTING	3											
ENVIRONMENT RELATED DISTRESS	BLOCK CRACKING	4											
	LONGITUDINAL CRACKING	5											
	TRANSVERSE CRACKING	6											
	MEANDERING CRACKING	7											
CONSTRUCTION DEFICIENCIES	SETTLEMENTS	8											
	CONSTRUCTION JOINT CRACKING	9											
	JOINT REFLECTION CRACKING	10											
MATERIAL RELATED DEFICIENCIES	COARSE AGGREGATE LOSS	11											
	RAVELLING	12											
	FLUSHING	13											
	BLEEDING	14											
	SHOVING	15											
OTHER DISTRESSES	PATCHING/UTILITY CUTS	16											
	POTHOLES	17											
	SEALED CRACKS	18											

REMARKS:

FIGURE 1 PAVEMENT CONDITION EVALUATION FORM

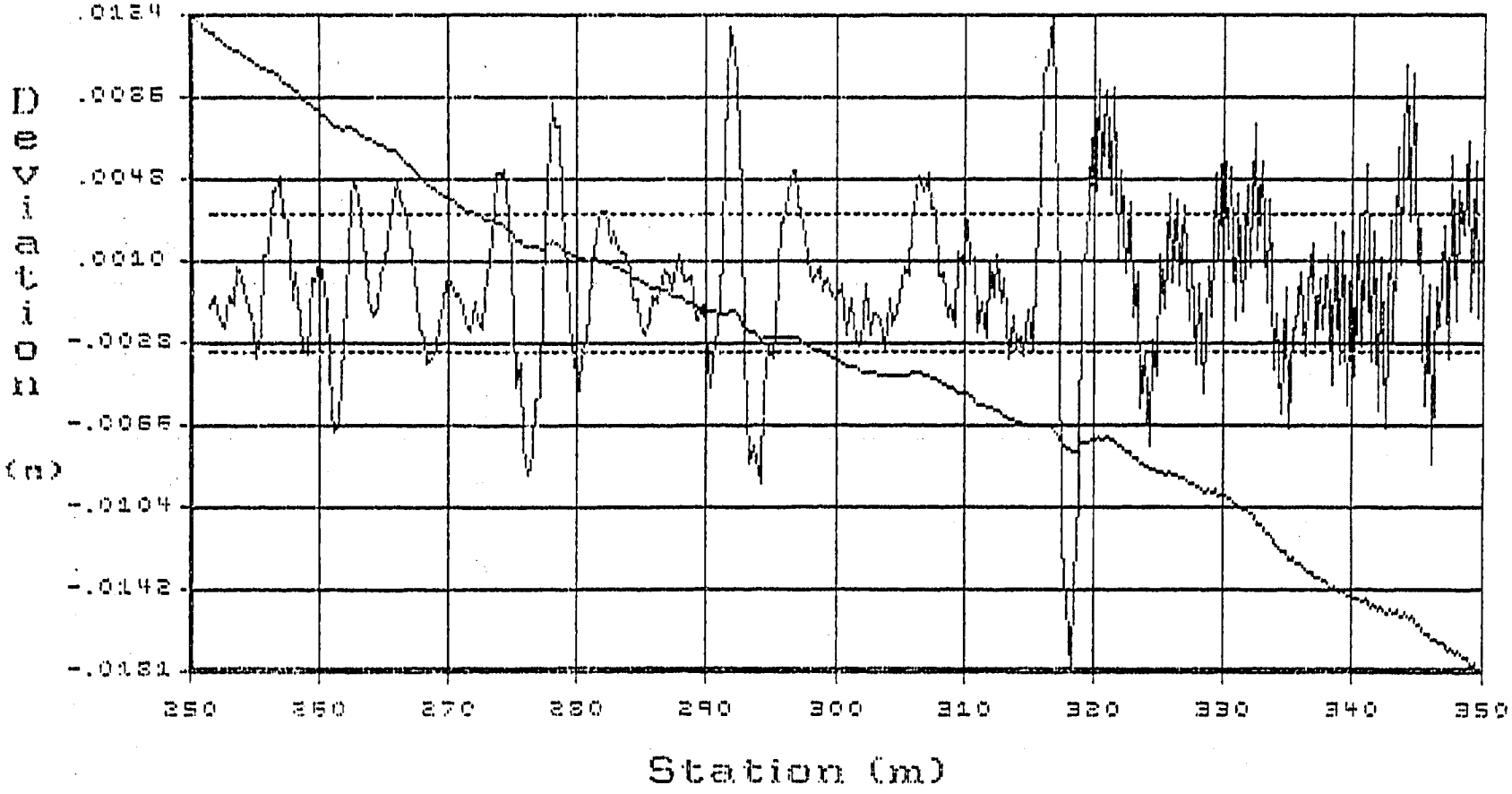
studies have documented the shortcomings of response type roughness measuring devices and recommend the use of devices that directly measure the pavement profile [2]. One such device that has been gaining rapid acceptance is the Digital Incremental Profiler. This portable computerized profiling device measures the pavement surface profile at 300 mm intervals to an accuracy of 0.1 mm and satisfies the requirements for an International Road Roughness Experiment (IRRE) Class 1 device [3]. Once the pavement surface profile has been characterized, the profile data can be analyzed to provide various roughness statistics. A comprehensive computer program (HRAP) has recently been developed to automatically accept input from the Digital Incremental Profiler and calculate roughness statistics such as the International Roughness Index (IRI), Surface Profile/Travelling Straight Edge (SP/TSE) and Root Mean Square Vertical Acceleration (RMSVA). Typical summary roughness statistics are given in Table 2.

TABLE 2 SUMMARY OF TYPICAL PROFILE ANALYSES

Section	Root Mean Square Vertical Acceleration (mm/m^2) Base Length			International Roughness Index (mm/m)	Profile Deviation Per Unit Length (mm/m) Straight Edge Length		
	1.2 m	1.8 m	3.0 m		3.0 m	10.0 m	30.0 m
1	4.3	2.7	1.4	4.2	9.0	21.7	48.3

A typical pavement profile and the profile deviation from the midpoint of a 3 metre straight edge is shown in Figure 2. The dashed lines in the figure represent the maximum allowable profile deviation with deviations above and below these lines indicating an unacceptably rough pavement surface.

Should the user wish to relate the profile based roughness indices calculated using the pavement profile to a subjective index such as the Riding Comfort Indices (RCI) or subjective indices such as the Portable Universal Roughness Device (PURD) or Maysmeter, a roughness measures correlation chart has been developed (Figure 3).



STRAIGHT EDGE LENGTH = 3.0
CRITICAL DEVIATION = .0032

FIGURE 2 EXAMPLE OF A TYPICAL PROFILE DETAIL

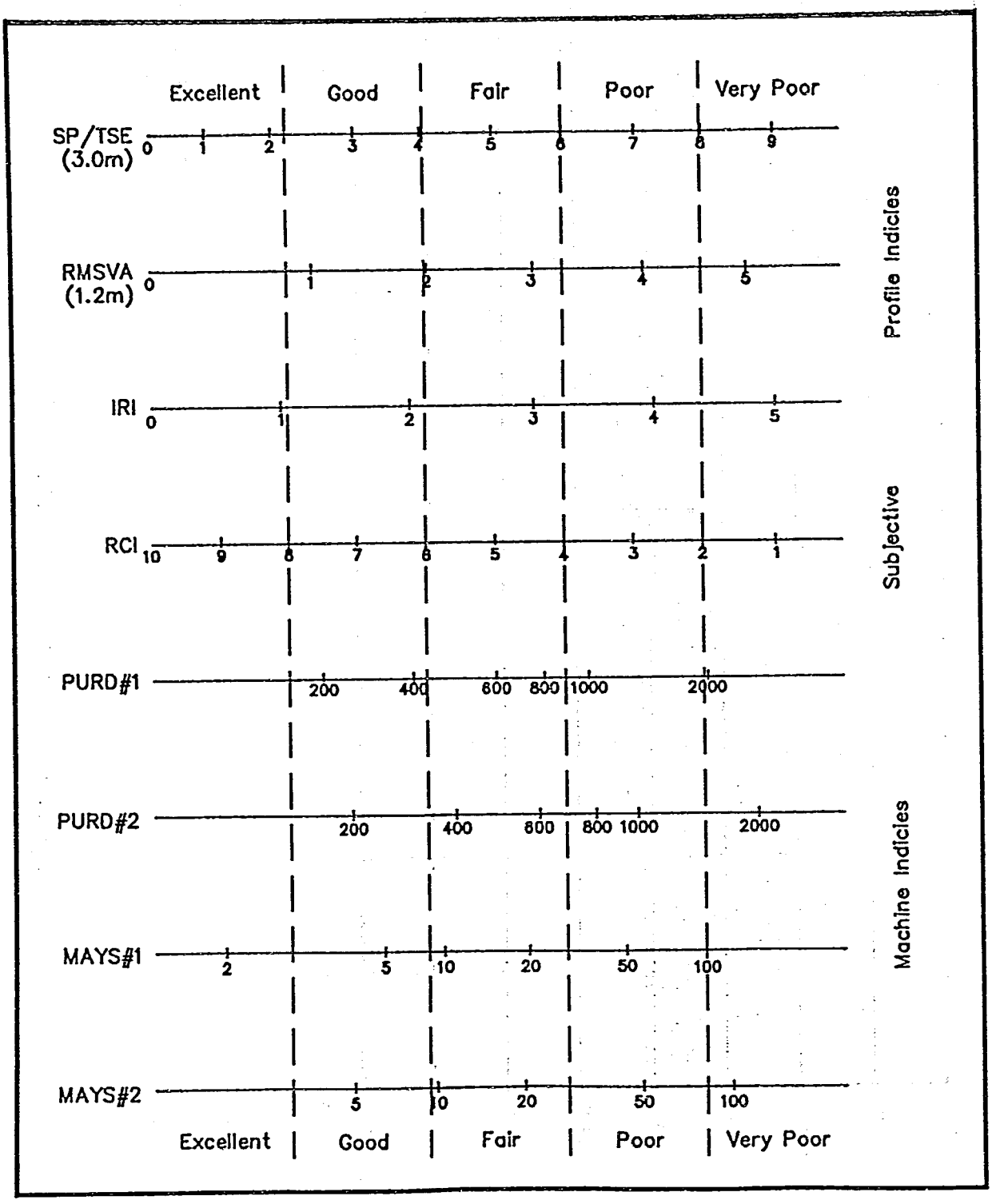


FIGURE 3 ROUGHNESS MEASURES CORRELATION CHART [1]

General rehabilitation action warrants have been developed for corrective action for roughness as shown in Table 3.

TABLE 3 GENERAL PAVEMENT REHABILITION WARRANTS BASED ON PAVEMENT ROUGHNESS

SP/TSE (3.0 m)	General Rehabilitation Action
0 - 4	No Action Necessary
4 - 6	Localized Repairs
6 - 8	Major Maintenance; Overlay, Recycling, etc.
8 - 10	Reconstruction

3. STRUCTURAL EVALUATION

The Falling Weight Deflectometer (FWD) is generally recognized as state-of-the-art equipment for rapid monitoring of pavement deflections. The FWD is an impulse type testing device that applies a load to the pavement surface similar in magnitude and duration to that of a moving wheel load. The deflection of the pavement surface is measured at seven predetermined locations from the point of load application. The measured deflections and load data are stored by on-board computer for later processing and analysis. The equipment is completely automatic and has the ability to collect pavement deflection data at several load levels for a particular test location within about one minute.

Pavement deflection testing is generally carried out at 100 metre intervals in alternating wheel paths. For general characterization and representative section delineation purposes, the maximum measured deflection is normalized to represent the equivalent deflection at a standard load and pavement temperature. Typical results of a deflection testing project are shown in Figure 4.

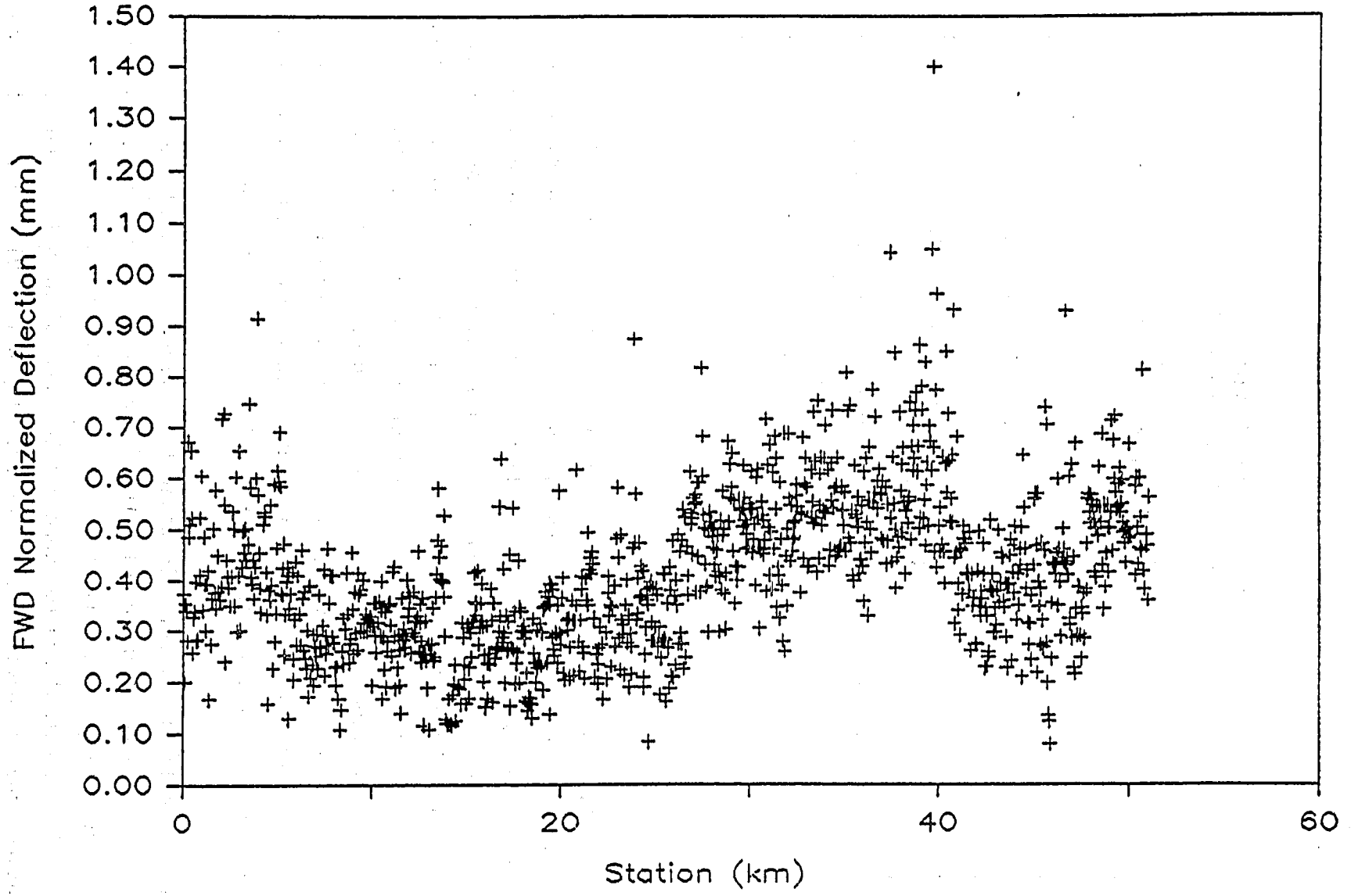


FIGURE 4 EXAMPLE OF A TYPICAL PAVEMENT DEFLECTION TEST RESULT SUMMARY FALLING WEIGHT DEFLECTOMETER

3.1 Elastic Layer Analysis Procedure

The load deflection data obtained using the FWD is analyzed using the Dynatest ELMOD (Evaluation of Layer Moduli and Overlay Design) computer program [4]. The ELMOD program uses the deflection data generated by the FWD along with information on pavement layer thickness, traffic and environment, to rapidly back-calculate the modulus or stiffness of each of the pavement layers. By using the pavement layer moduli and traffic loading information, the stresses and strains at the interface of each layer can be calculated for comparison with critical values assumed from general material characterization.

In order to back-calculate the stiffness of the individual pavement layers from surface deflection data, the thickness of the layers must be known. Although it is desirable to have pavement thickness information at each test location, this is impractical given that the FWD can test about 400 locations per day. It must be recognized that a 10 percent variation in the asphalt concrete thickness at any test location can result in a difference of up to 25 percent in the calculated modulus of the layer at that location. The significance of this variation is reduced when the 'modelled' pavement structure is used to predict its performance, i.e. if the asphalt concrete thickness used in the model is less than the actual thickness, this will result in an over-estimation of the asphalt concrete modulus; however, the stiffness of the layer will be accurately represented, and therefore the calculated stresses and strains at the interface should also be representative of the vertical pavement.

Traffic information (in the form of number of equivalent single axle loads (ESAL) per lane per year) is extrapolated to represent the total ESAL per lane over the design period.

Climatological data can be obtained from agencies such as Statistics Canada [5]. This information typically consists of 25 year average monthly air temperature and precipitation records. The climatological data are used to divide the year into three representative 'seasons' and characterize the variation in the magnitude of the layer moduli over the entire year, noting that the modulus of asphalt concrete is very

dependent upon its temperature. During the cold winter months, the asphalt concrete is very stiff (high modulus of elasticity), with the modulus of the asphalt concrete during the warm, summer months generally about 25 percent of the winter value. Similarly, the modulus of the subgrade is dependent on its moisture condition. A very wet, moisture susceptible subgrade will have a lower modulus of elasticity than the same subgrade in a dry condition. Precipitation and temperature records for environmental monitoring stations are generally provided in the format shown in Table 4.

TABLE 4 TYPICAL ELASTIC LAYER ANALYSIS PRECIPITATION/TEMPERATURE DATA (Southern Ontario)

Month	Average Rainfall (mm)	Snowfall (cm)	Temperature (^{0}C)
January	209	60	4
February	216	25	7
March	170	12	10
April	91	1	13
May	50	0	17
June	36	0	20
July	24	0	24
August	39	0	25
September	90	0	21
October	194	0	15
November	280	7	8
December	288	29	5

The above information is incorporated into the ELMOD program to calculate the modulus of elasticity for each of the pavement layers. Since the subgrade deflection is the most significant contributor to the total pavement deflection, it is important to consider any non-linear, elastic response of the subgrade. Three of the seven FWD deflection transducers are spaced at a distance sufficiently far from the point of loading that they measure, for all practical purposes, the response of the subgrade only. As such, the subgrade modulus under the actual load can be adjusted to account for any non-linearity in the pavement response. The modulus of the remaining layers of the pavement structure can be determined from the radius of curvature of the deflection basin and overall deflection response of the pavement.

Once the modulus of elasticity is determined for each layer, the ELMOD program computes an estimated modulus for each 'season' using the environmental and materials characterization data. The program simulates the traffic loading for each model pavement structure for each 'season' and determines the stresses and strains at the layer interface. The stresses and strains are summed for each season using Miner's Law to determine the overall 'damage' to the pavement due to the cumulative effects of traffic.

In order to estimate the cumulative 'damage' to the pavement structure, the stresses and strains are mathematically compared to empirical maximum allowable values calculated from various road tests and accelerated loading experiments. Failure of the asphalt concrete surfacing is defined by the maximum number of allowable repetitions of a critical horizontal strain at the base of the layer until cracking would be expected to occur. Failure of the unbound materials is determined by the maximum allowable repetitions of vertical compressive strain or stress at the top of the layer until excessive deformation occurs.

In order to determine the thickness of overlay required to ensure that the critical stresses and strains in the modelled pavement structure are not exceeded for the design period, the ELMOD program mathematically increases the thickness of the defined surfacing material in the modelled pavement structure until it is equivalent to the thickness necessary to prevent the theoretical failure of the pavement structure. A complete list of the design assumptions used in the ELMOD calculations is given in Table 5. An example output of the structural analysis program is shown in Tables 6 and 7. The mean plus one standard deviation of the calculated overlay is generally used as the overlay for the entire section.

TABLE 5 EXAMPLE OF ELMOD DESIGN PARAMETERS

1. Asphalt Modulus/Temperature Relationship

   Et/Ec = 1 - 2.5 Log (t/21)

   where Et = asphalt concrete modulus at desired temperature
   Ec = measured asphalt concrete modulus
   t = temperature at the time of testing

   Minimum Temperature = 5^0C Maximum Temperature = 30^0C

2. Seasonal Parameters

Season	Number of Weeks	Characteristic Week	Subgrade Reduction Factors
1	18	8	0.994
2	23	10	0.859
3	11	5	0.977

   Week of Maximum Temperature = 32
   Week of Minimum Subgrade Modulus = 12

3. Structural Design

   Standard Gear Load = 40 kN
   Dynamic/Static Ratio = 1.2
   Tire Pressure = 0.7 MPa
   Design Gear = Dual wheel
   Distance Between Wheels = 330 mm

4. Failure Criteria

   a) Asphalt Tensile Strain Relationship

   $S = 0.06311 * N^{(-0.304)} * (E/E^1)^{(-0.259)}$

   where S = critical strain
   N = number of load applications
   E = asphalt modulus (ksi)
   E^1 = 6.90

   b) Subgrade Compressive Strain

   $S = -0.03 * N^{(-0.250)} * E$

   where S = critical strain
   N = number of load applications
   E = subgrade modulus

5. Overlay Properties

Season	Modulus (MPa)
1	7000
2	3000
3	6500

TABLE 6 EXAMPLE OF ELASTIC LAYER ANALYSIS CALCULATIONS

Station	E1,MPa	E2,MPa	E3,MPa	CO	N
43.275	3936	317	127	73	-0.17
43.366	3636	374	120	42	-0.32
43.412	13058	367	119	108	-0.03
43.458	3952	346	128	128	+0.00
43.503	1769	683	172	33	-0.53
43.549	21091	361	124	72	-0.14
43.595	15399	457	115	115	+0.00
43.641	6291	349	128	76	-0.15
43.686	4507	382	171	113	-0.13
43.732	3143	269	97	97	+0.00
43.778	4714	374	107	50	-0.22
43.824	9660	294	124	124	+0.00
43.869	2914	510	147	45	-0.37
43.915	3797	511	138	40	-0.37
43.961	2628	421	136	46	-0.35
44.007	1772	309	159	111	-0.13
44.098	8355	281	95	95	+0.00
44.144	4661	237	129	128	-0.00
44.190	4625	677	122	23	-0.47
44.235	4908	305	158	141	-0.04
44.281	3653	322	125	63	-0.21

TABLE 7 EXAMPLE OF TYPICAL OVERLAY DESIGN OUTPUT USING ELMOD

Station	Life	layer	Mode	Overlay(mm)
43.275	9.47	1	strc	29.0
43.366	13.06	1	strc	16.0
43.412	19.56	1	strc	0.0
43.458	11.40	1	strc	21.0
43.503	>20.00	1	strc	0.0
43.549	>20.00	1	strc	0.0
43.595	>20.00	1	strc	0.0
43.641	13.84	1	strc	15.0
43.686	13.70	1	strc	14.0
43.732	6.21	1	strc	45.0
43.778	13.24	1	strc	16.0
43.824	12.30	1	strc	21.0
43.869	>20.00	1	strc	0.0
43.915	>20.00	1	strc	0.0
43.961	15.41	1	strc	10.0
44.007	7.35	1	strc	37.0
44.098	10.43	1	strc	27.0
44.144	5.08	1	strc	54.0
44.190	>20.00	1	strc	0.0
44.235	8.74	1	strc	32.0
44.281	9.55	1	strc	28.0
44.327	8.63	1	strc	31.0
44.373	>20.00	1	strc	0.0

4. PROTOTYPE SECTIONS

The rapid speed of the visual condition survey should allow even relatively large roadway networks to be routinely monitored on a regular basis. Pavement sections exhibiting poor visual condition can be further characterized using the pavement structural and roughness approaches outlined. The amount of testing that one can complete will depend on the size of the network. For relatively small networks, the visual condition survey can be completed for the entire network with the deflection and roughness testing completed only for pavements with pavement condition ratings between about 4 and 8. Below a rating of 4, the pavement would generally require reconstruction, and above 8, generally little or no maintenance is required. For large networks, it is suggested that the visual condition survey be completed for the entire network. Based on the condition survey, construction history and maintenance records, representative or prototype sections can be selected for regular monitoring. This concept is similar to that developed by the Canadian Good Roads Association (now the Roads and Transportation Association of Canada) and currently being used by the various Strategic Highway Research Programs. The performance of the prototype sections can be used to select the optimum timing and overlay thickness design (mechanistic) requirements for the overall network, check the effectiveness of new maintenance methods and the applicability of mechanistic design approaches (AASHTO, for instance).

5. CONCLUDING REMARKS

It must be recognized that pavement design and rehabilitation is not an exact science. A large number of variables are involved including the properties of the individual pavement layers (asphalt concrete, granular base/subbase and subgrade), layer thicknesses, the effects and variability of traffic types and loadings, and environmental factors such as rainfall, temperature and freeze thaw effects, etc. With recent rapid advancements in in-situ pavement performance monitoring equipment (Falling Weight Deflectometer and Digital Incremental Profiler, for example), pavement and materials characterization has been developed

into a more exact science. By coupling the concept of rapid, regular, seasonal monitoring of prototype pavement sections with conventional overall network monitoring and accounting for the 'degradation' of pavement materials (tensile strain - fatigue life of asphalt layers, for instance), a rational (mechanistic) basis for selection of maintenance and rehabilitation procedures can be developed.

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Seasonal Variations in Bearing Capacity of Asphalt Pavements

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SEASONAL VARIATIONS IN BEARING CAPACITY OF ASPHALT PAVEMENTS

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ABSTRACT

A test asphalt pavement was constructed to measure deflections by a falling weight deflectometer. Deflection measurements were made weekly at the same place on the pavement from November, 1989 through April, 1990. Eleven thermo-couples were buried in the pavement to measure a distribution of temperature and a depth of frost penetration.

Backcalculations were made to estimate the elastic moduli of the component layers of the pavement. In the backcalculations, the pavement was considered a four-layer structure consisting of an asphalt layer, a granular base, a subgrade, and a bedrock which is 10m below the upper surface of the subgrade. It was found that the seasonal variations in bearing capacity of the asphalt pavement could be expressed by the variation in the elastic moduli.

The main results from this study are summarized as follows:

1) Freezing of the granular base and subgrade decreases the tensile strain of the asphalt layer remarkably.
2) During the thawing season, the asphalt pavement sustains heavy damage, especially when the temperature in the asphalt layer is high.

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1. INTRODUCTION

Asphalt pavements in cold areas are often quickly damaged, because of frost heave in the coldest season, and deterioration in the bearing capacity during thawing. Studies of seasonal variations in the bearing capacity of asphalt pavements would be important for estimating the performance of asphalt pavements and establishing rational design method. A test asphalt pavement was constructed with an 80mm thick asphalt layer, and a 200mm thick granular base. Deflection measurements were made weekly at the same place on the pavement by a Falling Weight Deflectometer from November, 1989 through April, 1990. Eleven thermo-couples were buried in the pavement from the surface down to 980mm to measure the distribution of temperature and the depth of frost penetration.

Deflection data were collected 0, 300, 600, 900, 1200, and 2000mm from the center of the loading plate of the Falling Weight Deflectometer (FWD). Backcalculations were made by combining deflection data and elasticity theory for the layer structure, to estimate the elastic moduli of the component layers.

Nondestructive testing of pavements and backcalculations of elastic moduli are of great interest for pavement engineers [1], as they will be helpful to:

(1) Estimate the elastic moduli of component layers of pavements, and evaluate their bearing capacity.
(2) Determine design thicknesses of overlays to increase the bearing capacity of pavements.
(3) Assess the rate of deterioration of the bearing capacity, and predict the remaining service life of pavements.
(4) Determine optimum times for rehabilitation.

In backcalculations of elastic moduli, one problem is that elastic moduli estimates vary with calculation methods and models of pavement structure. We used elasticity theory for the layer structure, and the four-layer structure model of pavements consisting of an asphalt layer, a granular base, a subgrade, and a bedrock assumed to be 10m below the top of the subgrade.

The elastic modulus of the asphalt layer was known from the temperature of the asphalt layer, and the elastic moduli of the granular base and subgrade were estimated from deflection data by FWD. The purpose of this study is to show the seasonal variations in the bearing capacity of the pavement and its relation to variations in the elastic moduli.

2. TEST PAVEMENT

The test pavement was 5m wide and 60m long and constructed on the campus of Hokkaido Institute of Technology in 1988. The pavement structure was a 200mm thick granular base and an 80mm thick asphalt layer. The soil in the subgrade was silty sand. To measure the distribution of temperature and the depth of frost penetration in the pavement, eleven thermo-couples were buried at different depth. The vertical distribution of temperatures was measured simultaneously with deflection measurements by the FWD.

Five thermo-couples were buried 5, 20, 40, 60, and 80mm below the surface of the pavement for temperature measurements in the asphalt layer, and six were buried at 230, 380, 530, 680, 830, and 980mm for the depth measurements of the frost penetration. Figure-1 shows the pavement structure with the position of the thermo-couples.

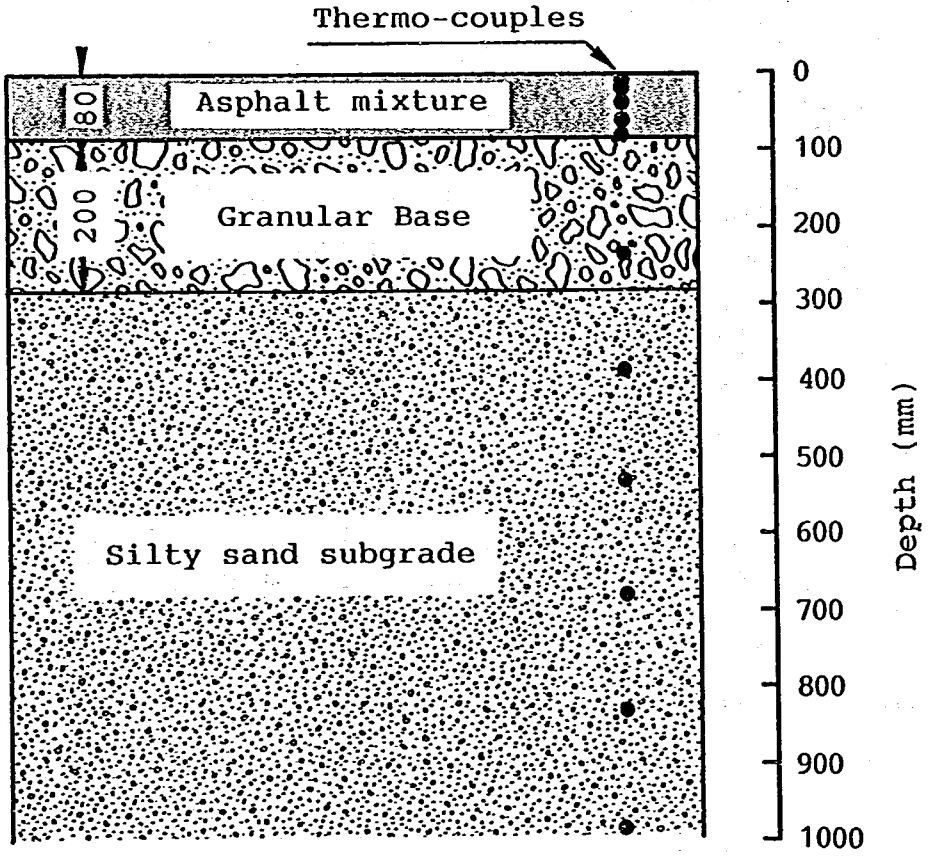


Figure 1 Pavement Structure with Vertical Position of Thermo-couples

3. FALLING WEIGHT DEFLECTOMETER

The Falling Weight Deflectometer in this study was Phonix FWD PT 5002 Model. The main specifications are:

Load: 50 kN

Radius of loading plate: 150mm

Range of deflection measurements: 0 - 2mm

Location of deflection measurements: 0, 300, 600, 900, 1200, and 2000 mm from the center of the loading plate.

Figure-2 shows the PT 5002 Model.

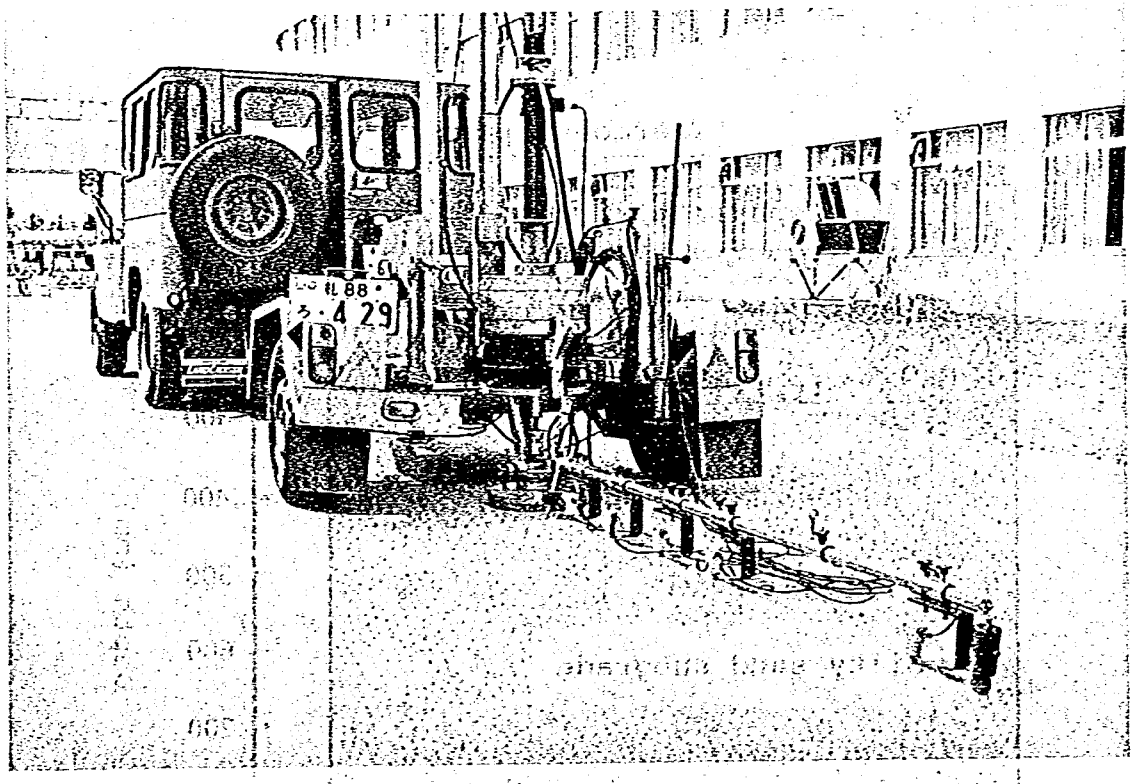


Figure 2 Phonix Falling Weight Deflectometer (Model PT5002)

4. MEASUREMENTS OF DEFLECTION AND TEMPERATURE

The deflection was measured weekly at one point on the test pavement by the FWD during five months from early winter 1989 through the spring of 1990, and the temperature distribution in the pavement was measured simultaneously by the thermo-couples. Table-1 shows the temperature and deflection at the different distances from the loading plate.

Figure-3 indicates the depth of frost penetration, the mean temperature of the asphalt layer and the deflections at the center (D_0) and 2000mm from the center (D_{2000}) of the loading plate.

Freezing started on December 11, 1989 and ended on March 12, 1990. Frost penetration reached 80mm on December 27, 1989, and the maximum depth was 980mm from the middle of February through early March. Figure-3 clearly shows that the variations in the deflection at the center of the loading plate (D_0) agree well with the changes in the depth of frost penetration.

The D_0 value was a maximum 1.463mm early in March during thawing, and it was 0.317mm on January 29, the coldest day. The difference in deflection by the FWD was found to be 1.1mm between the freezing and the thawing periods. The D_{2000} value, 2000mm from the center of the loading plate, was 0.058mm during freezing and 0.143mm during thawing, showing clear seasonal variations. The deflection during thawing is 3.5 - 4.5 times that during freezing. Deflections during thawing and freezing were compared to the baseline data, the mean deflection of measured values in early winter from November 20 through December 4. The relative values of D_0 and D_{2000} during thawing were 0.270 and 0.460, and 1.220 and 1.016 during freezing.

The mean temperature of the asphalt layer is given as the weighted mean of the layer thickness. As the deflection measurements were made during the day, the mean temperature was rather high, within ±5°C, even in the coldest period.

Table 1 Seasonal Variation of FWD Deflections and Temperatures of Pavement

Date	Deflections (mm)						Loads (kN)	Depth form surface (mm)										
	D0	D300	D600	D900	D1200	D2000		5	20	40	60	80	230	380	530	680	830	980
								Temperatures (°C)										
20 Mov. '89	1.214	0.892	0.540	0.378	0.245	0.129	4916			9.0	9.0	9.0						
27 Mov. '89	1.131	0.814	0.503	0.361	0.235	0.123	4916			8.9	8.9	8.9						
4 Dec. '89	1.180	0.881	0.532	0.374	0.240	0.126	4875			13.0	13.0	13.0						
11 Dec. '89	1.032	0.756	0.471	0.337	0.231	0.120	4916	-1.0	-0.8	-0.7	-0.6	-0.5	-0.4	-0.2	1.0	2.0	3.0	5.0
19 Dec. '89	0.509	0.407	0.299	0.254	0.183	0.088	4916	-3.5	-3.0	-3.0	-2.0	-2.0	-1.0	0.0	1.0	2.0	3.0	3.0
25 Dec. '89	0.632	0.469	0.332	0.278	0.198	0.097	4916	-2.0	-2.0	-1.0	-1.0	-1.0	-0.5	0.5	1.0	2.0	3.0	4.0
27 Dec. '89	1.173	0.874	0.548	0.397	0.258	0.131	4916	-1.0	-0.5	-0.5	-0.5	-0.5	1.0	1.0	1.0	2.0	2.5	3.5
8 Jan. '90	0.453	0.352	0.264	0.235	0.173	0.082	4875	-2.2	-2.1	-2.1	-2.1	-2.1	-2.0	-1.0	0.0	0.5	1.5	2.5
16 Jan. '90	0.344	0.276	0.206	0.187	0.142	0.072	4916	-4.0	-5.5	-5.0	-5.0	-5.0	-5.0	-2.0	-0.5	0.5	1.0	2.0
22 Jan. '90	0.324	0.243	0.180	0.163	0.123	0.069	4916	-6.5	-6.5	-5.0	-5.0	-5.0	-5.0	-2.0	-1.0	0.0	0.5	1.5
29 Jan. '90	0.317	0.227	0.157	0.139	0.102	0.058	4916	0.0	0.0	-0.5	-0.5	-1.0	-3.5	-3.5	-2.0	-1.0	0.0	1.0
2 Feb. '90	0.414	0.268	0.182	0.164	0.127	0.069	4957	1.2	1.3	1.8	1.2	0.3	-1.5	-1.5	-1.2	-1.0	0.0	1.0
13 Feb. '90	0.644	0.478	0.307	0.242	0.166	0.080	4916	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-0.5	0.0
19 Feb. '90	0.864	0.645	0.396	0.292	0.202	0.103	4957	5.0	5.0	3.0	2.0	0.5	-1.0	-1.0	-1.0	-1.0	0.0	0.5
26 Feb. '90	1.434	1.038	0.631	0.435	0.267	0.128	4916	2.0	1.0	0.0	0.0	0.0	-0.5	-1.0	-1.0	-1.0	-0.5	0.0
5 Mar. '90	1.463	0.942	0.559	0.396	0.259	0.138	4957	8.0	7.0	4.0	3.0	1.5	0.0	-1.0	-1.0	-1.0	-1.0	0.0
12 Mar. '90	1.377	1.051	0.641	0.452	0.286	0.143	4957	6.5	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.0	2.5	2.0
19 Mar. '90	1.319	0.984	0.583	0.406	0.260	0.139	4916	14.4	13.8	11.3	10.5	9.5	6.3	3.1	3.1	3.1	3.0	3.0
26 Mar. '90	1.094	0.759	0.469	0.345	0.229	0.123	4930	3.0	2.0	2.5	2.5	2.0	1.5	3.0	4.0	4.0	4.0	4.0
2 Apr. '90	1.236	0.807	0.466	0.332	0.224	0.123	4916	16.0	15.0	13.0	11.5	10.5	9.0	8.0	8.0	7.0	6.0	5.0
9 Apr. '90	1.228	0.892	0.510	0.360	0.236	0.127	4916	5.0	5.0	4.5	4.0	3.5	1.5	2.5	2.0	1.5	1.5	1.0
17 Apr. '90	1.427	0.924	0.487	0.335	0.228	0.126	4902	27.0	26.0	24.0	22.0	20.0	14.0	9.0	9.0	9.0	8.0	8.0
24 Apr. '90	1.351	0.953	0.543	0.371	0.245	0.132	4902	21.5	18.0	15.5	13.5	12.0	9.5	10.0	10.5	10.5	10.5	10.0

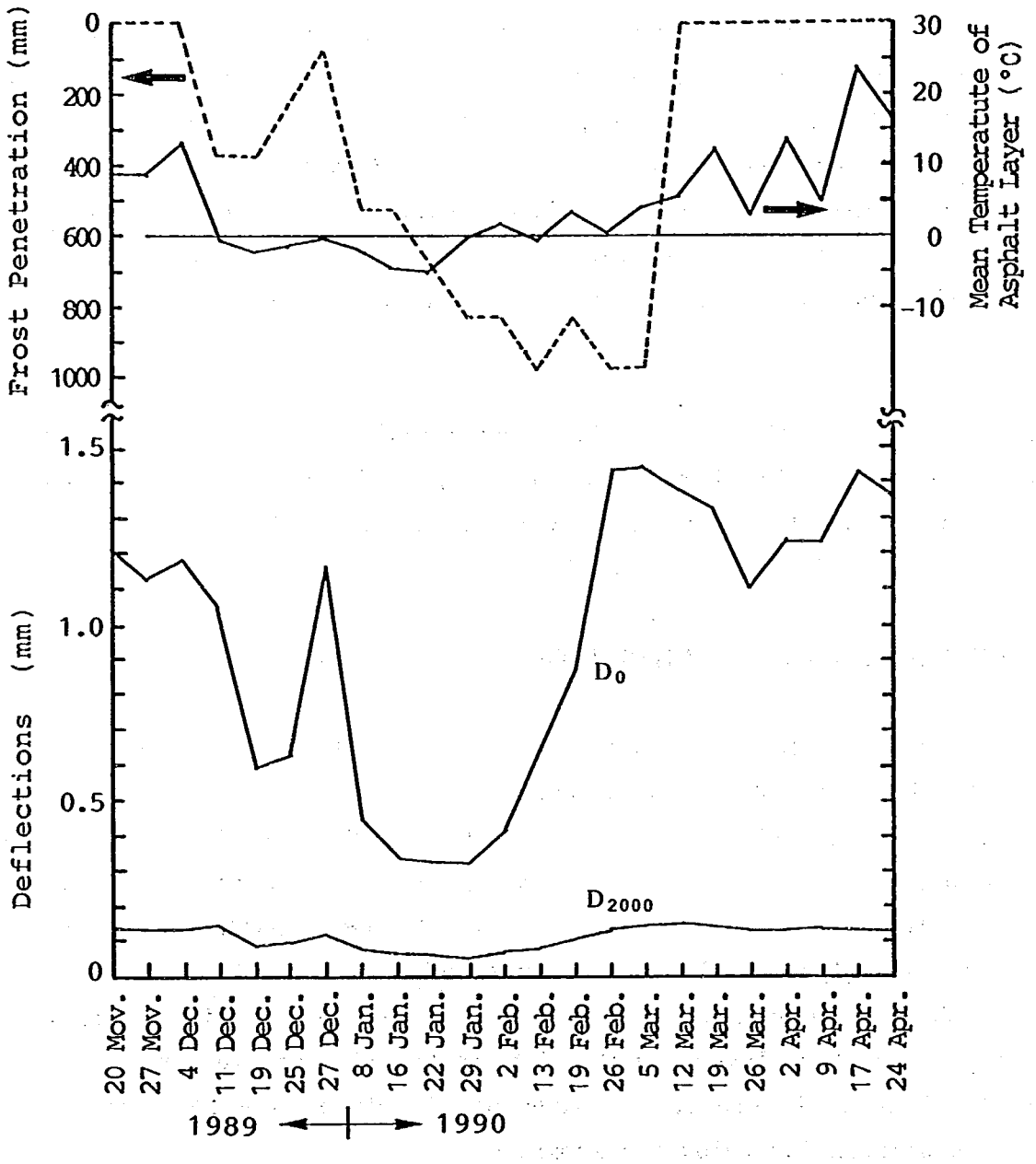


Figure 3 Seasonal Variations of FWD Defections, Depth of Frost Penetrations and Mean Temperatures of Asphalt Layer

5. ESTIMATING IN-SITU ELASTIC MODULI OF GRANULAR BASE AND SUBGRADE

5.1 BACKCALCULATIONS

Backcalculations are commonly used to estimate the elastic moduli of component layers of pavements using deflections and loads by the FWD as known amounts. The methods used in the backcalculations were elasticity theory for the layer structure, the finite element method, and visco-elasticity theory, including both static and dynamic analyses. Some cases assume an infinite depth of subgrade or bedrock under the subgrade [2]. No generally established procedures have been agreed upon however, because estimated values of elastic moduli vary widely with methods of calculation and assumptions.

The study uses LMBS (Layer Moduli Backcalculation System) developed by Himeno, one of the authors [3]. In LMBS, ELSA (Elastic Layer Structure Analysis Program) based on Chevron's analysis for use on personal computers, was added in a subroutine.

5.2 ESTIMATING IN-SITU ELASTIC MODULI

To estimate the elastic moduli of the component layers of the pavement using the LMBS, the following assumptions are made:

(1) The elastic modulus of the asphalt layer were given by the mean temperature of the asphalt layer, from the relation between the temperature and the elastic moduli of the asphalt layers shown in Figure-4. Figure-4 shows the results of dynamic indirect tension tests in the laboratory, and correspond well to the elastic moduli of the asphalt mixture layers by the FWD for the 30m sec. loading time [4].

(2) As shown in Figure-5, the pavement structure is a four-layer system with bedrock 10m below the subgrade.

(3) The elastic modulus of the bedrock is 100,000 MPa.

Input variables included six sets of deflection data measured by the FWD. The load of the FWD and the initial elastic moduli of the granular base and the subgrade, the elastic moduli of the granular base and the subgrade were successively varied. The root mean square error of the measured and

calculated deflections was considered to have converged when the change was below 10^{-6}.

Table-2 lists the estimated elastic moduli of the granular base and subgrade.

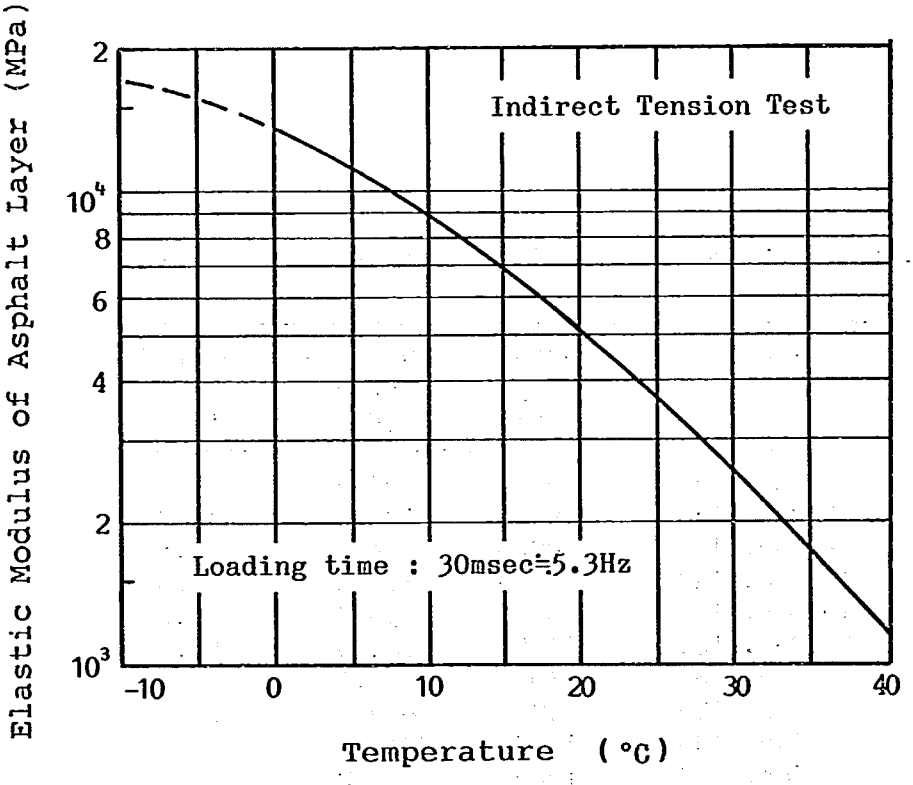


Figure 4 Elastic Modulus of Asphalt Layer coresponding to FWD Loading Time

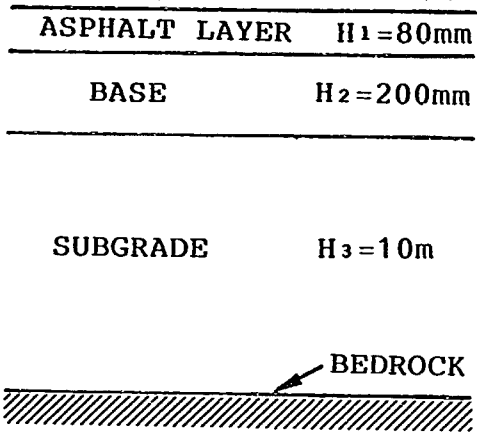


Figure 5 Pavement Structure for Backcaluculation

Table 2 Estmated Elastic Moduli of Asphalt Pavement (MPa)

Date	Asphalt layer (E1)	Granular subbase (E2)	Subgrade (E3)
20 Mov. '89	9500	93.9	39.1
27 Mov. '89	9500	124.5	41.2
4 Dec. '89	8000	134.3	38.5
11 Dec. '89	13000	110.8	43.9
19 Dec. '89	15000	136.7	56.2
25 Dec. '89	14000	744.0	53.3
27 Dec. '89	14000	65.1	38.7
8 Jan. '90	15000	1829.0	60.3
16 Jan. '90	15000	3249.2	73.9
22 Jan. '90	16000	2902.0	86.8
29 Jan. '90	14000	2008.9	107.5
2 Feb. '90	13000	1147.2	95.1
13 Feb. '90	14000	438.7	61.8
19 Feb. '90	14000	177.0	50.9
26 Feb. '90	13000	19.0	39.3
5 Mar. '90	11000	23.8	42.4
12 Mar. '90	10500	62.0	33.7
19 Mar. '90	8000	93.8	36.0
26 Mar. '90	11000	105.6	44.2
2 Apr. '90	6500	96.8	45.0
9 Apr. '90	11000	50.3	42.5
17 Apr. '90	4000	87.2	42.6
24 Apr. '90	6000	93.2	38.7

6. SEASONAL VARIATIONS IN THE ELASTIC MODULI OF GRANULAR BASE AND SUBGRADE

The seasonal variations in the elastic moduli of the granular base and subgrade are shown in Figure-6.

The elastic modulus of the granular base reached a maximum 3249.2 MPa on January 16, 1990. It was 100 - 200 MPa before the freezing of the granular base, and increased with the progress of freezing. In the thawing season it rapidly fell to 19.0 MPa, possibly due to the very high water content of the granular base. After the very low values it increased to 100 MPa.

The elastic modulus of the subgrade reached a maximum of 107.5 MPa on February 29, 1990. Before freezing the elastic modulus of the subgrade was 40 - 60 MPa and it increased with the progress of freezing. It fell to 40 - 60 MPa in the thawing season, and indicated higher values than the granular base from the end of February through early March.

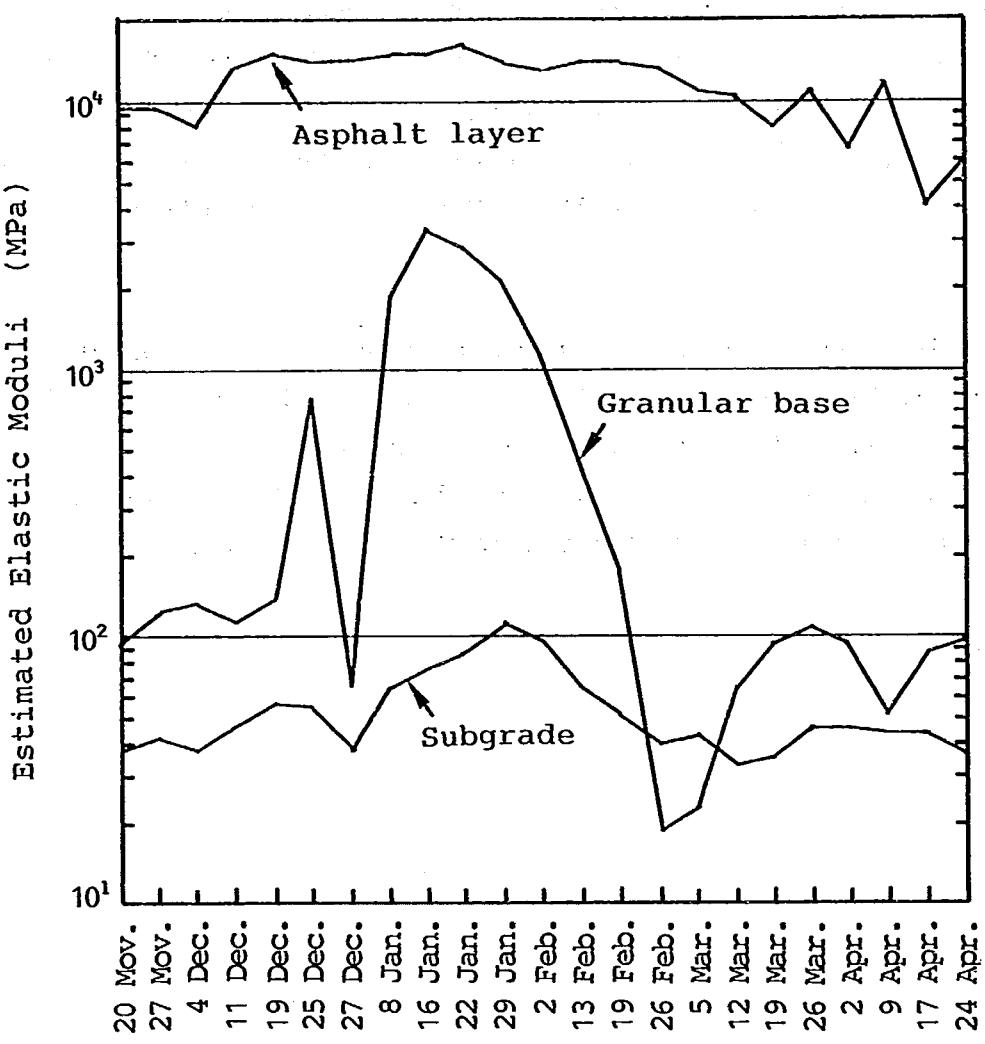


Figure 6 Seasonal Variations of Backcaluculated Elastic Moduli of Granular Base and Subgrade

7. TENSILE STRAIN AT BOTTOM OF ASPHALT LAYER

Since the elastic moduli of each layer of the pavement varies remarkably with the season, it is estimated that the tensile strain at the bottom of the asphalt layer (ε_a) also changes with standard loads. Typical elastic moduli of each component layer was selected for each season, and was calculated using the BISAR.

Table-3 shows estimated elastic moduli and calculated strain at the bottom of asphalt layer (ε_a) under a standard wheel load. The load conditions are: dual wheels each with a load of 25kN, the radius of contact area 132.5mm, the contact pressure 450 kPa, and the distance between wheel centers 340mm.

Table 3 Tensile Strain at Bottom of Asphalt Layer (MPa)

Season	Date	Estimated Elastic Moduli (MPa) E1	E2	E3	E4	Strain, ε_a (microstrain)
Early winter	20 Mov. '89	9500	93.9	39.1	100,000	282
Coldest season	16 Jan. '90	15000	3249.2	73.9	100,000	24.1
Early spring	26 Feb. '90	13000	19.0	39.3	100,000	293
		5000	19.0	39.3	100,000	584

The value of strain ε_a due to the standard load was 282×10^{-6} on November 20, 1989, and below 24.1×10^{-6} in the freezing season. This shows that the bearing capacity of an asphalt pavement with a frozen granular base is very high in winter. As pointed out by Huber et al. [5], the granular base and the subgrade may allow heavier loads than the standard during freezing. This is an important consideration for winter transportation by road in cold areas.

The tensile strain on February 26, 1990 was 293×10^{-6} , 4% higher than in early winter, despite the small elastic modulus of the granular base. However, when the temperature of the asphalt layer increased rapidly (e.g. 20°C) by solar radiation, the elastic modulus of the asphalt layer fell steeply (E_1=5000 MPa), and reached 584×10^{-6} . It is estimated that with

the small elastic modulus of the asphalt layer during thawing, the tensile strain becomes twice that in early winter. This is shown in the rapid failure of asphalt pavements during thawing. It also indicates that vehicles with overloads damage asphalt pavements heavily during thawing with the low bearing capacity of the granular base and the subgrade.

8. CONCLUSIONS

The results of measurements of seasonal variations in deflection of asphalt pavements using the FWD may be summarized as follows:

(1) The depth of frost penetration in the asphalt pavement influences deflection strongly.

(2) When the deflection at the center of the loading plate (D_0) in early winter without freezing of the granular base and subgrade is taken as a baseline (1.00), the deflections during freezing and thawing were 0.27 and 1.22.

(3) The seasonal variations in the bearing capacity of the asphalt pavement could be expressed by the variations in the elastic moduli of the granular base and the subgrade using backcalculations.

(4) When the elastic moduli of the granular base and subgrade in early winter estimated by back-calculations was the baseline, the maximum elastic moduli during freezing were 34.6 and 2.7, and the minimum elastic moduli during thawing 0.2 and 0.86.

(5) Freezing of the granular base and the subgrade decreases the tensile strain at the bottom of the asphalt layer extremely. During freezing, heavier loads than standard are permissible, an important consideration for winter transportation by road in cold areas.

(6) During thawing asphalt pavements suffer great damage, especially when the temperature of the asphalt layer is high.

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Integration of Rehabilitation, Maintenance and Upgrading in Saskatchewan's Pavement Management Information System

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ABSTRACT

For many years pavement management information systems have demonstrated the ability to show tradeoffs between rehabilitation options. One of the biggest problems of managing pavements, however, is making tradeoffs between maintenance, rehabilitation and upgrading treatments. How much of the total highway budget should be dedicated to each of these treatments? Do tradeoffs really exist?

Answers to the above questions are possible with Saskatchewan's Pavement Management Information System. This approach to PMIS was accomplished using a common set of benefits for each one of the treatments. PMIS generates a set of strategies for each pavement section that includes maintenance, rehabilitation and upgrading treatments.

An optimization routine is used to select the optimum strategies based on a flexible set of constraints such as maximizing benefits and minimizing budgets. A number of different future programs can be generated and evaluated in terms of total agency costs, total benefits, or future condition of the highway network.

INTRODUCTION

Most highway agencies utilize pavement management systems to help them optimize their pavement rehabilitation programs and expenditures. Generally these pavement management systems are stand alone systems devoted to pavement rehabilitation aspects of the highway system only, and do not consider the maintenance or upgrading types of interventions in the analysis. Therefore, they do not allow a true optimization of available scarce funds in that highway managers cannot evaluate the comparative merits of rehabilitating, routinely maintaining or upgrading various highway segments.

A considerable portion of Saskatchewan's highway system has gravel or thin-membrane asphalt surfacing which are quite maintenance intense and require upgrading to stronger pavement types. Therefore, the Pavement Management Information System (PMIS) designed by Saskatchewan Highways and Transportation expressly includes in its analysis the maintenance and upgrading alternatives in addition to the traditional pavement rehabilitation scenarios.

Details of the pavement rehabilitation aspects of Saskatchewan's PMIS have been previously published (1, 2). The purpose of this paper is to describe how the maintenance and upgrading alternatives are considered in Saskatchewan's PMIS to arrive at an overall optimization of the entire maintenance, rehabilitation and upgrading programs and budgets.

SALIENT FEATURES OF SASKATCHEWAN'S HIGHWAY SYSTEM AND PROGRAMS

Saskatchewan has a road and street network of about 200,000 kilometres. Of this length, 25,500 km are provincial highways managed by Saskatchewan Highways and Transportation; approximately 8,000 km are urban streets managed by cities, towns and villages; and the remaining 166,000 km are rural grid and farm-access roads and trails managed by rural municipalities.

The 25,500 km network of provincial highways has the following surface type characteristics.

Surface Type	Approximate Length (km)
Intermediate (Type A and B) and Standard Pavement	10,000
Thin-membrane (Type C and D) Asphalt Surfacing	9,500
Gravel	6,000

The standard and intermediate pavements require seal coats and overlays; while the thin membrane asphalt surfacing require seal coats but as a rule no overlays.

Saskatchewan's PMIS is concerned with approximately 19,500 km of provincial highways with standard or intermediate pavements or thin-membrane asphalt surfacing in terms of rehabilitation, upgrading and maintenance; and with the 6,000 km of gravel surfaced highways in terms of upgrading only.

Table 1 shows Saskatchewan Highways and Transportation's annual expenditures from 1971-72 to 1990-91 in the capital, maintenance and administration categories. The main conclusion from the table is that since 1980-81 maintenance expenditures have been increasing both in absolute terms and as a proportion of total department expenditures while capital expenditure have stayed relatively constant in absolute terms, and have decreased as a proportion of the total department expenditures.

Table 2 shows the capital expenditures for various work types since 1980-81. The main conclusion is that highway rehabilitation expenditures increased from $6.1 million in 1980-81 to $42 million in 1990-91. Since the total capital expenditures did not increase much, the higher allocation to highway rehabilitation was at the expense of allocation to construction (which comprises new construction, reconstruction, upgrading to higher standard, widening, etc.).

AN OVERVIEW OF THE PAVEMENT MANAGEMENT INFORMATION SYSTEM

The primary objective of the Pavement Management Information System (PMIS) is to provide a mechanism through which future rehabilitation, maintenance, and upgrading needs of the highway network in the province of Saskatchewan can be evaluated. Although the determination of need is usually a yearly activity, the system is designed so that the criteria that influence these needs can be specified as inputs (e.g., available budgets, intervention levels) which make it a valuable tool to analyze needs at any time.

More specifically, the objectives of PMIS are to facilitate a systematic analysis into various methodologies of:

- predicting future condition of highways by use of performance models for a variety of distresses such as roughness, rut depth, and cracking;
- determining the generic types of maintenance, rehabilitation, and upgrading treatments that are applicable for different types of road and different program activities;
- specifying the circumstances under which those treatments can be applied in terms of road conditions and intervention levels;
- developing costs and quantities for each treatment type;
- calculating the improvement that results out of applying any treatment to any road at any condition;
- determining the economic results of applying any treatment in any year in terms of road user cost savings, and future maintenance, upgrading and rehabilitation cost savings.

A simplified flowchart of the PMIS system is shown in Figure 1. Data from the Corporate Database is accessed by the PMIS Analysis System. The data is processed and ultimately produces a series of future "programs" for the highway network. Each of the major components is discussed briefly below. A detailed discussion of the design of PMIS is given in (1).

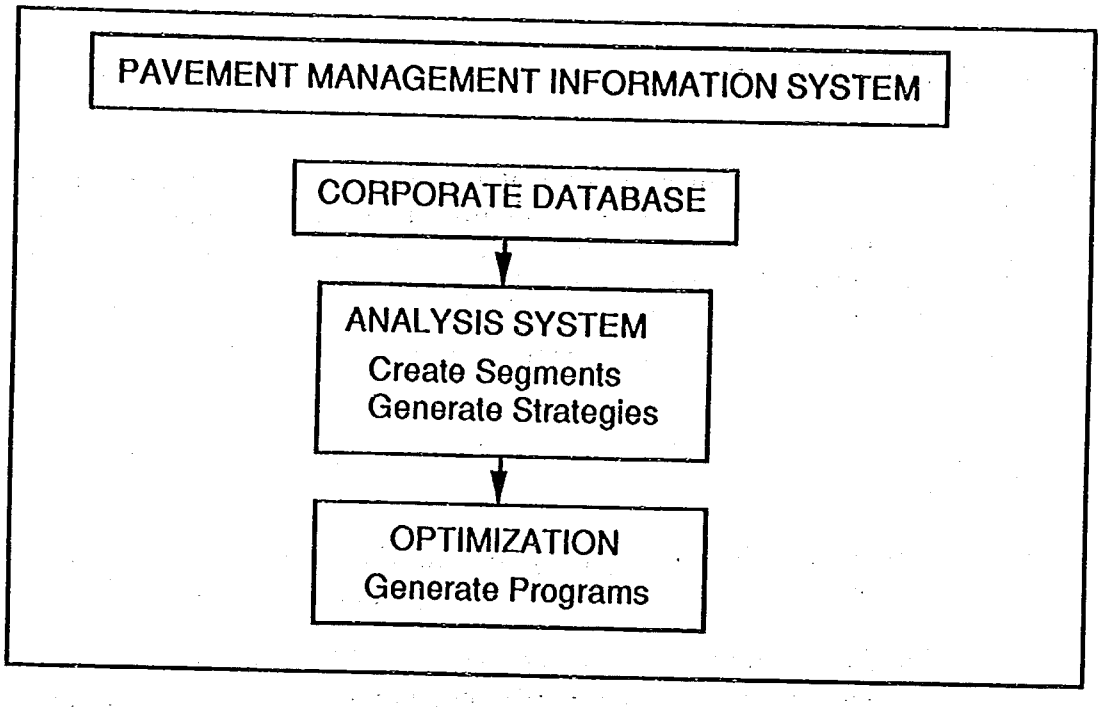


Figure 1: Flowchart of PMIS

Corporate Database

PMIS uses data in the Department as directly as possible. The only new data PMIS required which did not exist in the Department, was road condition data and road user cost data. The remaining data required by PMIS was available from the highway inventory system, traffic accident and traffic information systems, Riding Comfort Index system, and a maintenance cost information system. These systems were operating independent of one another and data from them was brought together into a corporate database for use by PMIS.

Analysis System

The heart of the PMIS is in the analysis system. This system has two specific and different functions. These are: 1) divide the highway network into homogeneous segments; 2) analyze each segment using performance models and assign a list of treatments

The first step of the analysis is to divide the highway network into a series of homogeneous analysis segments. "Homogeneous" refers to the fact that the segment is consistent throughout its length for any user defined characteristic. The process involved is to obtain the list of characteristics the user wishes to remain constant and then assemble the data into segments that at least keep those characteristics constant throughout their length.

After the segments have been created, the next step is to analyze each segment one at a time. During this analysis the future values of various features are predicted using performance models and are scrutinized for the application of treatments over a period of twenty years. During this step, the parameters from each of the performance models are obtained from user defined support files. A segment may be eligible to receive a number of treatments over the twenty years. A "strategy" is defined as a combination of treatments applied in different years that ensure a segment stays in reasonable condition for that time.

Optimization

Once all of the segments have been analyzed and have had a list of strategies generated for them, the next step is to perform the optimization. The basic premise of the optimization is to select one strategy for each segment so that the objective constraints are satisfied. This process is carried out using an incremental benefit-cost approach.

The resulting list of optimum strategies for each segment make up the "program", which contains the treatments, conditions, costs, and material quantities for each particular segment for a twenty year period. From the program, information about the highway system can be obtained for any of the twenty years, such as upgrading, rehabilitation, and maintenance costs, material quantities, roughness, etc.

PERFORMANCE MODELS

The PMIS analysis system uses nine performance prediction models. These are used to predict the future performance and condition of each highway segment. Based on the future value of these conditions, various intervention levels can then be specified to apply rehabilitation, maintenance or upgrading treatments. The nine performance models are briefly described below. A detailed description of the performance models, and their calibration is given in (2).

International Roughness Index

This model predicts roughness deterioration of the highway segments in terms of the International Roughness Index (IRI). This model is a function of segment age, cumulative equivalent single axle loads (ESAL's), and the current state of roughness. Although PMIS predicts future roughness using the IRI, the user inputs RCI and gets RCI back. This is done for two reasons. First, the performance model was developed using the IRI models of HDM III which were calibrated for local conditions. Second, the Department has used, and will continue to use, a COX road meter whose results are reported as RCI RCI is measured annually on the entire system.

Rut Depth

This model predicts rut depth deterioration of highway segments in terms of depth of rutting progression measured in millimetres. This model is a function of segment age, cumulative equivalent single axle loads (ESAL's), and the current rut depth. The current rut depth is measured on the highway system using a PURD unit. One third of the highway system is rated each year, thus covering the entire system on average once every three years.

Environmental Cracking

This model predicts cracking progression due to environmental conditions and includes temperature (transverse) cracking and longitudinal cracking measured in metres per kilometre. This model is a function of segment age and current state of cracking. The environmental cracking is measured using visual condition surveys by the operators of the PURD unit on a three year cycle.

Deep patching

This model predicts progression of deep patching requirements and is measured in square metres per kilometre This model is a function of segment age and current state of deep patching. The state of patching is measured using visual condition surveys with the PURD unit and annual maintenance effort on deep patching.

Spot Sealing

This model predicts progression of spot sealing and is measured in percent of travelled way area per kilometre This model is a function of segment age and current state of spot sealing. The state of spot sealing is measured using visual condition surveys with the PURD unit and annual maintenance effort on spot sealing.

Cracking and Patching

This model predicts progression of cracking and patching (includes fatigue and other load associated cracking) and is measured in percent of travelled way area per kilometre This model is a function of segment age and current state of cracking and patching. The state of patching is measured using visual condition surveys with the PURD unit and annual maintenance effort on patching.

Machine and Hand Patching

These two models predict progression of hand and machine patching and are measured in tonnes per kilometre These models are a function of segment age and current state of patching. The state of patching is measured using visual condition surveys with the PURD unit and annual maintenance effort on patching.

Other Maintenance Dollars

This model predicts other surface maintenance cost and is measured in dollars per kilometre This model is a function of segment age and current state of expenditure. The state is measured using annual maintenance costs.

TREATMENTS

The PMIS analysis system first predicts the condition of a highway segment for a twenty year period using the performance models discussed above. This situation is called the "Do-Nothing" strategy, since no treatments are applied and the segment is allowed to deteriorate with time with nothing other than routine maintenance being applied. This situation represents the absolute worst case the segment would deteriorate to and can be used to indicate what the maximum routine maintenance costs would be if there was no money available for rehabilitation or upgrading. The application of any treatment in the twenty years will result in an improvement to the roads condition, and may or may not result in less overall cost.

PMIS applies a rehabilitation, upgrading or maintenance treatment whenever the highway condition reaches a certain state. This state is called an intervention level, and the application of a treatment is "triggered" by a trigger model as shown in Figure 2 below. A trigger zone is defined to control when segments are eligible for a certain treatment. For example in Figure 2, two different trigger zones are shown for two different types of treatment. Treatment A can only be applied when the distress condition is in the trigger zone for Treatment A. When the treatment is applied, the nine performance conditions of the segment are then "reset" according to the treatment applied and the future conditions are recalculated from that point in time. This process of triggering and applying treatments continues until the end of the twenty year period.

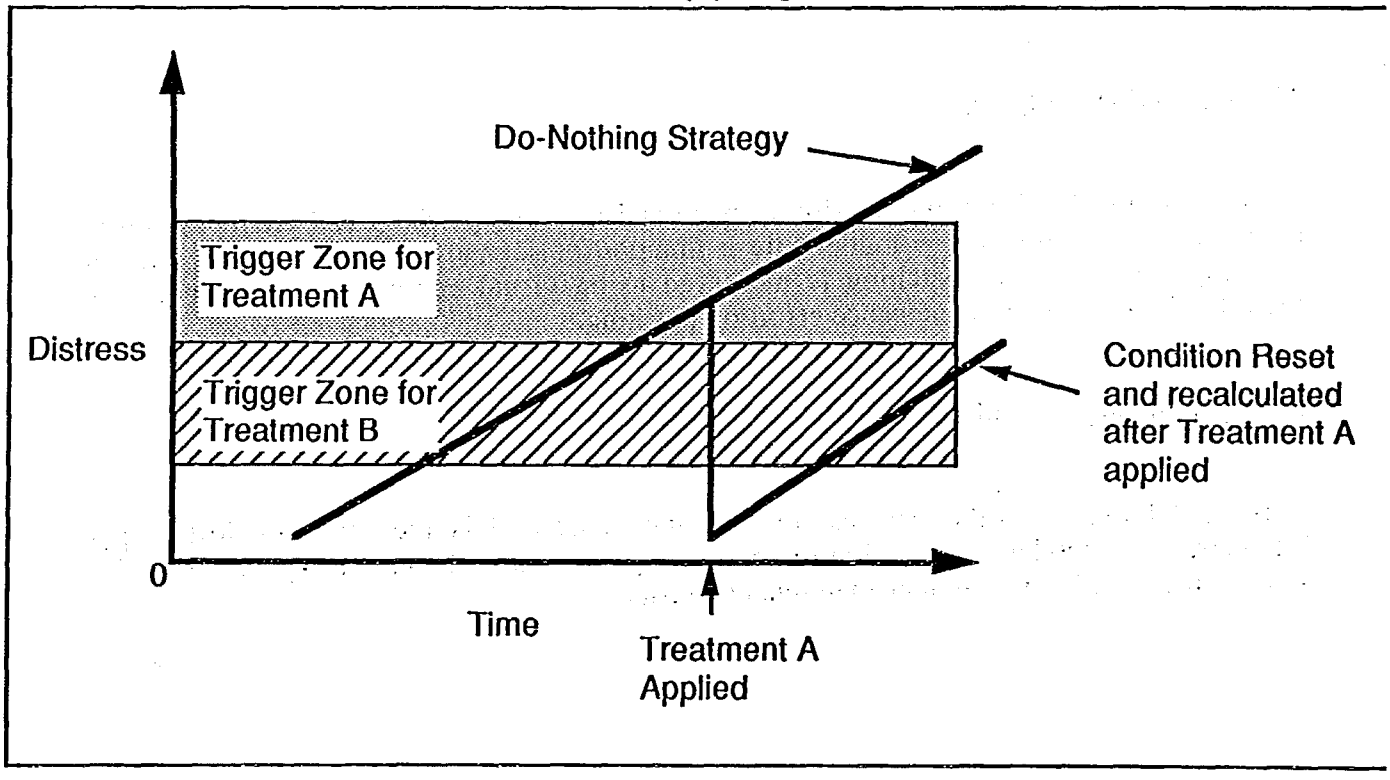


Figure 2: Trigger Zones

PMIS controls the manner in which treatments are applied by the use of strategies. A strategy is a specific pattern of treatments applied to a highway segment over time. This feature allows PMIS to integrate maintenance, rehabilitation and upgrading treatments.

PMIS creates a strategy by predicting the future performance conditions and "triggering" a treatment whenever any one of the conditions warrants. The list of PMIS treatments and the performance models which can trigger them are discussed below

Rehabilitation Treatments and Triggers

Rehabilitation treatments are triggered by the following performance models.

Rehabilitation Overlay Triggers	Riding Comfort Index (RCI) Rut Depth Deep Patching Cracking and Patching
Rehabilitation Seal Triggers	Cracking and Patching

For example, a Rehabilitation overlay can be applied on a segment when its RCI is between 5.4 and 6.0; when its rut depth exceeds 25 mm; when the deep patching area exceeds 10% of the travelled way area; or when the cracking and patching area exceeds 40% of the travelled way area. Even though the overlay can be triggered in any year for any of the conditions, the decision of which year is the best is left for the economics of the optimization step to decide. All the trigger parameters in PMIS are controlled by the user, which allows for evaluating different intervention policies.

The thickness of these structural overlays are designed by PMIS using future traffic volumes, modified structural number, and existing structure thicknesses. The segments condition is reset to newly rehabilitated highway conditions. Also, when an overlay is triggered, PMIS checks to see if the resulting roadway width would cause the section to be substandard. If this is the case, PMIS automatically switches the treatment to a recycle or an upgrade. In other words, PMIS will not apply a rehabilitation treatment which would result in the road segment becoming substandard. This feature ensures that all strategies keep the segments at or above current standards.

Rehabilitation seals have no structural contribution to the segment's performance and are applied to arrest fatigue cracking and postpone the application of an overlay. Conditions affected by seals such as fatigue cracking are reset by a certain percentage.

Maintenance Treatments and Triggers

There are two types of Maintenance treatments: thin overlays and routine maintenance activities. Maintenance treatments are triggered by the following performance models.

Maintenance Overlay Triggers	International Roughness Index Rut Depth Deep Patching Cracking and Patching
Routine Maintenance Triggers	Environmental Cracking Deep patching Machine Patching Hand Patching Spot Sealing Other Surface Maintenance

The thicknesses of these thin overlays are nominal 30 mm or 50 mm only, and are not "designed" per se in PMIS as are the rehabilitation overlays. The segments condition is reset to a percentage of a newly rehabilitated highway conditions.

The triggers for structural and maintenance overlays are shown in Figure 3. As the condition of distress on a segment increases, a segment will first be eligible for a thin maintenance overlay. If a thin overlay is not applied, the condition would get worse. When the condition does become worse, the segment will then become eligible for a structural overlay, and will no longer be eligible for a maintenance overlay. PMIS allows the user to overlap trigger zones so a segment will be eligible for both types of overlays at any given condition state. This allows for a cost-benefit evaluation of the overlay types which again, is handled in the optimization step.

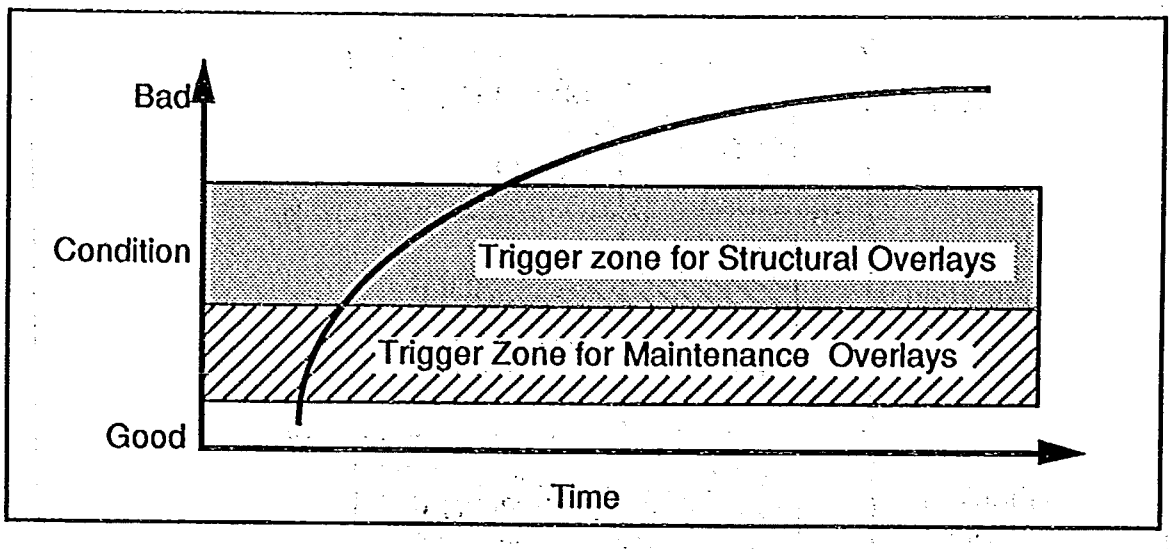


Figure 3: Rehabilitation and Maintenance Overlay Triggers

Routine maintenance activities are applied to the segment every year of the analysis. The amount of maintenance activity is determined by the various condition states of the segment. Routine maintenance models calculate costs and quantities only and do not reset any condition. In fact, some maintenance activities can have a negative effect on the segment's condition. For instance, roughness can increase when spot sealing is applied.

UPGRADE TREATMENTS AND TRIGGERS

Upgrade treatments are triggered by the following performance models.

Upgrade Triggers	International Roughness Index
	Traffic Volume
	Maintenance Costs
	Shoulder Width

PMIS expresses the upgrade trigger relationships in the form of a relationship between RCI and AADT, shoulder width and AADT, and maintenance cost and AADT as shown in Figure 4. These graphs show there are three possible zones a particular segment would fall into at any point in time: Sole, Marginal or None. Which zone it falls into depends on its current condition and traffic level. Any time a segment falls into any of the three sole zones, an upgrade treatment would be triggered by that factor alone, that is any one of RCI, maintenance costs or shoulder width. Or if the segment falls into the marginal zone on any two, an upgrade would also be triggered. When an upgrade is triggered, the built in design standard selection matrix in PMIS calculates what standard of road will result from the upgrade.

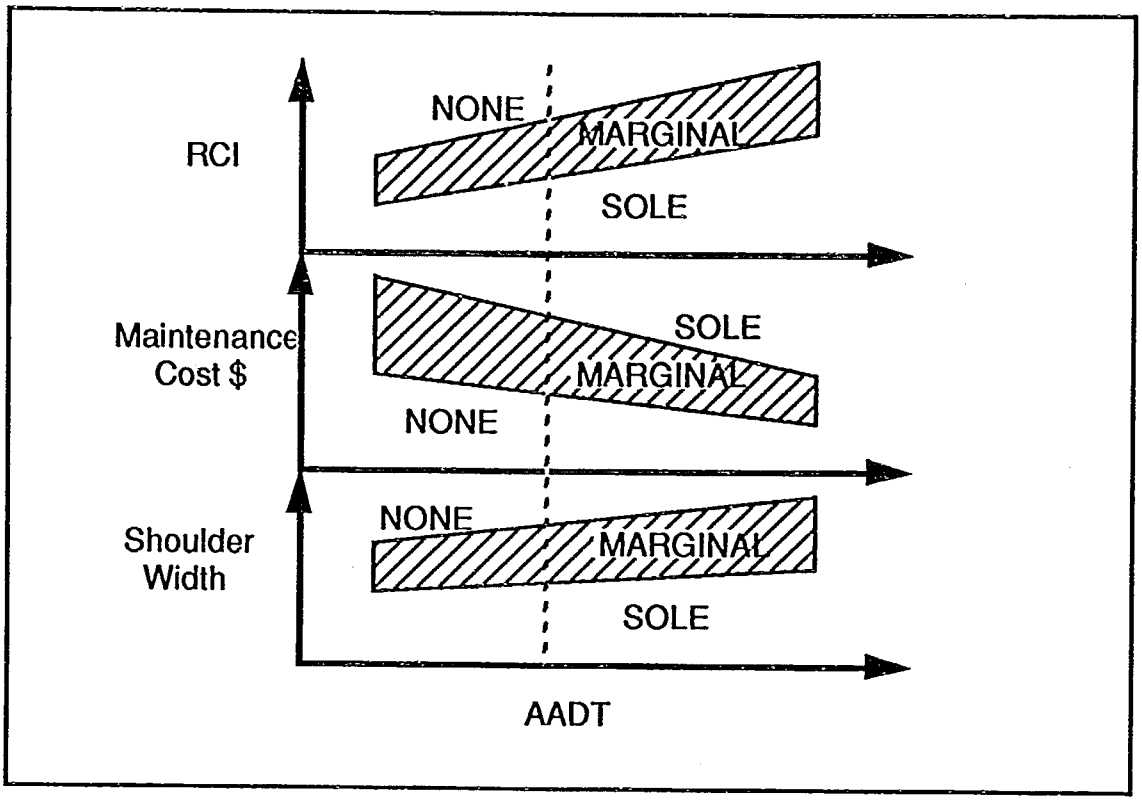


Figure 4: Upgrade Triggers

The thickness of the upgraded pavement structure and width is designed by PMIS using future traffic loadings and the new design standard of the segment. The segment's condition is reset to newly constructed highway conditions.

INITIAL TREATMENT MATRIX

During the analysis stage, PMIS checks the triggers for each segment and generates a treatment matrix which indicates the year in which that treatment could be applied. Each initial treatment will then become part of a unique strategy for that particular segment. After the initial treatment is applied, conditions are reset and then predicted for the remaining years. The conditions in these remaining years are checked to see if any other treatments should be applied before the end of the twenty years.

The allowable subsequent treatments in that strategy are determined from a user defined strategy pattern. For example, after an initial treatment of a rehabilitation overlay the next treatment may be a rehabilitation seal followed by an upgrade. The concept of strategy patterns is used to keep the number of strategies generated for each initial treatment both to a minimum and within practical boundaries. For example, an upgrade followed by an upgrade may not be practical.

An example of an initial treatment matrix is shown in Table 3 below. The matrix defines when and what type of initial treatments can be applied over the twenty year horizon using the performance models and trigger models.

Table 3: Initial Treatment Matrix

Initial Treatment	Year Applied 1	2	3	4	5	...	19	20
Rehabilitation								
Overlay	N	N	Y	Y	Y	N	N	N
Seal	N	Y	N	N	N	N	N	N
Maintenance								
Overlay	Y	Y	N	N	N	N	N	N
Annual	Y	Y	Y	Y	Y	Y	Y	Y
Upgrade	N	N	N	N	N	Y	Y	Y

The number of unique strategies that PMIS will generate for a road segment can range anywhere from 1 (Do Nothing Strategy) to a maximum of about 100. If strategy patterns were not used, the maximum number of potential unique strategies could reach into the thousands for each segment.

GENERATION OF STRATEGIES

The generation of strategies is the most complex portion of PMIS. PMIS determines the following information for each strategy during the analysis and stores the information in a strategy file:

- strategy number;
- types of treatments and years applied;
- condition of the segment determined by each of the performance models for each of the twenty years resulting from the treatments applied;
- quantities, costs and material quantities for each year for each of the maintenance activities;
- costs and material quantities for each year for each of rehabilitation, maintenance and upgrading treatments applied;
- road user costs for each year; and
- all economic factors necessary for the optimization process.

PMIS keeps track of the costs for each of the maintenance, rehabilitation, and upgrading budgets separately. All strategies, for example, have a yearly maintenance cost associated with them. A strategy will only have a rehabilitation cost for the year in which a rehabilitation treatment is applied. The same is true for an upgrade. The maintenance cost would be reduced for a strategy in the years during and following the application of a rehabilitation or an upgrade treatment. If the rehabilitation or upgrade treatments are delayed, the maintenance costs would increase during the years which the treatment has been delayed.

This information gives a complete future picture of the highway segment for each unique strategy. All of the strategies for a particular segment have different costs, benefits and result in different conditions at different years. What is left now, is to select a strategy for each segment that is the best. The constraints that determine which strategy is the best, and how it can be selected is discussed in the section below.

OPTIMIZATION

Costs

As previously discussed costs are calculated in PMIS for each strategy applied to a segment. The costs include routine annual maintenance costs; construction and quantity costs associated with application of any rehabilitation, upgrading or maintenance treatment; and road user costs. A study was conducted which calibrated the work done by the World Bank to Saskatchewan conditions. This study used the HDM III road user cost model as its basis (2). Road user costs are determined from vehicle operating costs according to vehicle type, road roughness, segment length and traffic volume. All costs are discounted to time zero using a user defined discount factor.

Benefits

On the benefit side of the equation, PMIS offers two alternatives. First, the user may elect to perform optimization using the "area-under-the-curve" benefits. Second, he may perform optimization using the savings in road user costs. The study into road user costs calibrated the work done using the world bank in developing the HDM III model for Canadian conditions (2). These two alternatives are discussed below.

Figure 5 shows the RCI performance curves for two strategies. The first strategy, which also yields the lowest RCI, is called the Do-Nothing strategy as previously discussed. PMIS assumes that each segment of road in the network has a Do-Nothing strategy which represents what the RCI value would decrease to in the event that only routine maintenance were performed on that segment. The second strategy shows what the RCI condition curve would look like if two treatments were applied during the twenty year analysis period. The first is an upgrade treatment applied in year five, and the second is a rehabilitation treatment applied in year fifteen. The remaining discussion will use the assumptions of this example.

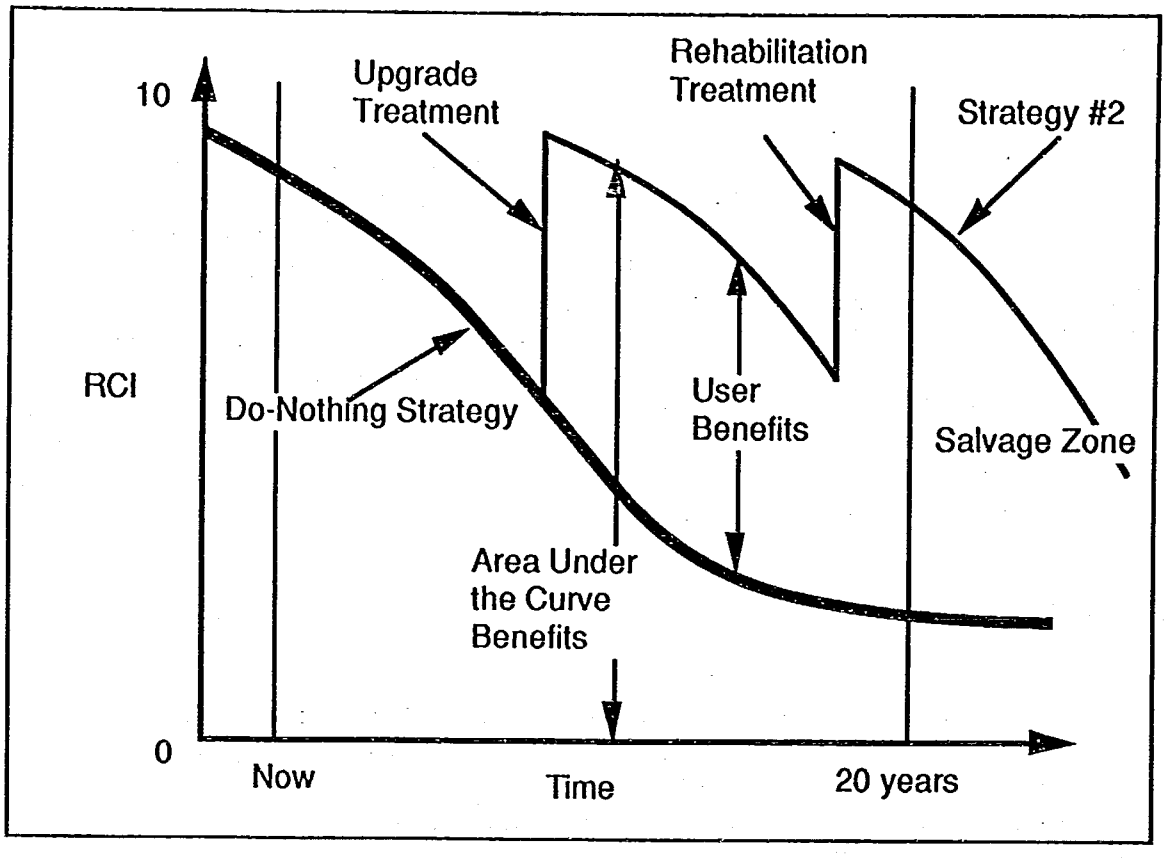


Figure 5: PMIS Analysis

PMIS will support the calculation and use of two types of benefits: dollars and "area under the curve". The second type is used by several highway agencies and is said to act as a surrogate for the first type. In the above example, the area-under-the-curve benefit of strategy number 2 is the total area under the RCI curve multiplied by the length of the segment. In this method, the benefit for a single year is the RCI of that year times the length of the segment. The area is then the sum of the benefits for all the years. In order to calculate the total benefits of strategy number two, all the yearly values are discounted to time zero using a user defined discount rate.

The two components involved in the calculation of dollar benefits are: savings in maintenance costs and savings in vehicle operating costs. The savings in maintenance costs are calculated by subtracting the maintenance costs of strategy two from the maintenance costs of the Do-Nothing strategy. The savings in vehicle operating costs are calculated in the same manner and are based on the conclusion that vehicle operating costs are inversely proportional to the RCI of the road. As with the previous benefits, the dollar benefits are calculated on a yearly basis and discounted to time zero with the same discount factor.

Incremental Benefit Cost

Optimization is performed using an incremental benefit-cost approach. The basis of this approach is that there is a single strategy in the network that yields the highest benefit for the least cost. Then, there is a second strategy that yields a higher benefit but for a higher cost, and a third highest and so on. Each strategy has a twenty year maintenance, rehabilitation and upgrading cost associated with it. PMIS checks to see if the costs for the strategy do not exceed the budgets for each of the three programs in any of the years. If the costs are less, the strategy is selected for that section of road. If there is still money left over, the next strategy is examined. And, just like the first, if there is enough budget for it, it too is selected. This is continued until the budgets are exhausted. The end result is that most benefits are achieved for the least cost, whether the benefits are in road user costs or in area under the curve.

CONCLUSION

This paper has described the salient features of Saskatchewan's PMIS which allows the evaluation of rehabilitation, upgrading and maintenance alternatives. PMIS uses nine performance prediction models to calculate the future performance and condition of the highway system. The system applies a rehabilitation, upgrading or maintenance treatment whenever a condition reaches a certain state. This state is called an intervention level and the treatment application is triggered by a trigger model.

Costs are calculated for each strategy and consists of routine maintenance costs, construction and quantity costs associated with the application of a treatment. Benefits are also calculated using "area-under-the-curve" benefits and user benefits sin terms of road user cost savings. Selection of the most optimum strategy is determined using an incremental benefit-cost approach.

The combining of rehabilitation, upgrading and maintenance allows Saskatchewan Highways and Transportation to evaluate the needs of the entire paved highway system and determines the resources required to satisfy those needs. Integration of maintenance, rehabilitation and upgrading allows a true optimization of available scarce funds and allow managers to evaluate the comparative merits of each.

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Evaluation of the Performance of Asphalt Pavement in Consideration of Reliability

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EVALUATION OF THE PERFORMANCE OF ASPHALT PAVEMENT IN CONSIDERATION OF RELIABILITY

by

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ABSTRACT

Tohoku Regional Construction Bureau has conducted a survey of pavement surface condition of national highway in every 3 to 4 years. This paper describes the evaluation of the performance of asphalt pavements by means of the Markovian method developed by the authors. The number of the data used in this study is 10,450 sites. The main results are as follows:

(1) The authors developed a mathematical method of deciding an annual transition probability matrix.

(2) Transition probability matrices of the asphalt pavement of each route were analyzed. Using these data, service life and its reliability were obtained.

(3) Repair methods were evaluated by a method based on the transition probability matrices.

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1/ INTRODUCTION

A method of evaluation based on a Markov chain model developed by the authors [1] was applied in the study to data from measurements of pavement performances made by the Tohoku Regional Construction Bureau of the Ministry of Construction to examine the performance of asphalt pavements of national highway in the Tohoku Region.

Almost every area in the Tohoku Region is cold and receives snow cover, and pavement surfaces suffer extreme damage from wearing caused by studded tires and tire chains in the wintertime. In such case, in order to carry out rehabilitation operations rationally, it is necessary to prepare a model expressing the time-series variations in performance of pavements. Normally, regression models are used as time-series models concerning performance of asphalt pavements, but only behaviors on average can be expressed by regression models. In contrast, the Markov chain model is capable of expressing the scatter in damage to asphalt pavements in terms of probability of the performance of an asphalt pavement by means of this model.

2/ MARKOV CHAIN MODEL

The surface conditions of an asphalt pavement are ranked in terms of an n number of stages. Here, a simple Markov property is assumed for the transitions in the damaged conditions of the asphalt pavement. That is, it is hypothesized that the condition at a certain stage depends solely on the condition of the immediately preceding stage. First of all, the case of repairs not performed is considered. In this case, the pavement reaching rank n which is the ultimate damaged state does not move on to another state, and rank n corresponds to the absorbing state in the Markov chain.

Letting the transition probability when the pavement in a condition of rank i moves on to rank j be p_{ij}, the transition probability for the group of the various ranks of road surface conditions will be as follows according to expression by matrix.

$$P = \begin{bmatrix} P_{11} & P_{12} & \cdots\cdots & P_{1n} \\ P_{21} & P_{22} & \cdots\cdots & P_{2n} \\ \vdots & \vdots & & \vdots \\ \vdots & \vdots & & \vdots \\ P_{n1} & P_{n2} & \cdots\cdots & P_{nn} \end{bmatrix} \quad \cdots\cdots(1)$$

where, $0 \leqq p_{ij} \leqq 1$

Here,since the individual transition phenomena are mutually exclusive and collectively exhaustive, the sum of the elements of individual rows will be 1.

$$\sum_{j=1}^{n} p_{ij} = 1 \quad \cdots\cdots(2)$$

If the initial condition of the pavement is expressed by a vector such as Eq. (3), the condition of the pavement after the m-th step may be expressed by Eq. (4). Furthermore, the probability that the pavement in a certain condition will not reach ultimate failure after the m-th step, that is, the reliability, is expressed by Eq. (5).

$$X(0) = [\, x_1(0) \;\; x_2(0) \;\; \cdots\cdots \;\; x_n(0) \,] \quad \cdots\cdots(3)$$

where, $x_i(0)$: probability of the pavement being of rank i in the initial condition

$$X(1) = X(0)P$$
$$X(2) = X(1)P = X(0)PP = X(0)P^2$$
$$\vdots$$
$$X(m) = X(m-1)P = \ldots\ldots = X(0)P^m \quad \ldots\ldots(4)$$

$$R(m) = 1 - x_n(m) \quad \ldots\ldots(5)$$

where, $R(m)$: reliability of pavement after m-th step

$x_n(m)$: probability of pavement after m-th step being of rank n, and obtained by putting initial condition probability in Eq. (4) as $X(0) = [\ 1\ 0\ \ldots\ 0\ ]$

Let the expected number of steps that a pavement of rank i in the initial condition being retained at rank j until reaching ultimate failure be m_{ij}. The matrix M according to this m_{ij} is expressed by Eq. (6). The sums of the individual rows are the expected numbers of steps until the pavements in the respective initial condition reach ultimate failures. In accordance with this, the expected number of absorbing steps of the pavement of initial state of rank 1 is obtained, with which it is possible to evaluate the expected service life of a newly constructed pavement.

$$M = [\ I - P'\ ]^{-1} \quad \ldots\ldots(6)$$

where, M : matrix of expected number of the retention steps

I : unit matrix of (n-1)-th order

P' : square matrix of (n-1)-th order deleting n-th row and n-th column from transition probability matrix P

Next, the absorbing point will disappear in case repairs are made periodically, and this is an ergodic Markov chain model which converges at a certain stationary probability X^*. This stationary probability X^* may be expressed by Eq. (7).

$$X^* = T\ [\ I - Q\ ]^{-1} \quad \ldots\ldots(7)$$

where, Q : matrix deducting 1 from elements of column n of transition matrix

I : unit matrix of n-th order

T : row vector of n-th order when value of n-th element is 1 and values of other elements are 0

In the transition process of pavement, the probability r_{ik} of repairs by the method k being carried out on a pavement of rank i and the probability f_{kj} of the pavement moving on to rank j in the next step through this repair k are derived in the manner of the following equations:

$$r_k = \begin{bmatrix} r_{1k} \\ r_{2k} \\ : \\ : \\ r_{nk} \end{bmatrix} = {}^t[\, r_{1k} \quad r_{2k} \,\cdots\cdots\, r_{nk} \,] \qquad \cdots\cdots(8)$$

where, $0 \leqq r_{ik} \leqq 1$, and the superscript t stands for the matrix transpose

$$f_k = [\, f_{k1} \quad f_{k2} \,\cdots\cdots\, f_{kn} \,] \qquad \cdots\cdots(9)$$

where, $0 \leqq f_{kj} \leqq 1$, $\sum_{j=1}^{n} f_{kj} = 1$

By employing these r_k and f_k, the transition probability p_{ij}' in case of executing L types of repair methods on a certain pavement section may be expressed employing the transition probability p_{ij} in case repairs are not made may be expressed by the equation below.

$$p_{ij}' = p_{ij} \left(1 - \sum_{k=1}^{L} r_{ik} \right) + \sum_{k=1}^{L} r_{ik} f_{kj} \qquad \cdots\cdots(10)$$

where, r_{ik} : probability of repair k being carried out on pavement of rank i, providing $\sum_{k=1}^{L} r_{ik} \leqq 1$

f_{kj} : probability of pavement subjected to repair k moving on to rank j in the next step

Further, the ratio R_k of method k from among the L varieties of repair methods being executed may be expressed by the following equation using the state probability vector and the repair probability vector:

$$R_k = [\, x_1 \quad x_2 \,\cdots\cdots\, x_n \,]\, {}^t[\, r_{1k} \quad r_{2k} \,\cdots\cdots\, r_{nk} \,] = X\, r_k \qquad \cdots\cdots(11)$$

3/ DATA FILE

Data from surveys of road surface conditions carried out by the Tohoku Regional Construction Bureau of the Ministry of Construction were used in this study. These surveys were made once every 3 to 4 years on the same unit sections with lengths of 100 m based on kilometer posts as unit survey sections on national highway. Cracking ratios, rutting, evennesses in longitudinal directions are recorded among the survey data. In the present study, data of 10,450 sites excluding sections of structures such as tunnels and bridges were extracted from among these survey data, and a data file was prepared supplementing these with traffic data obtained in the highway traffic census for 1985.

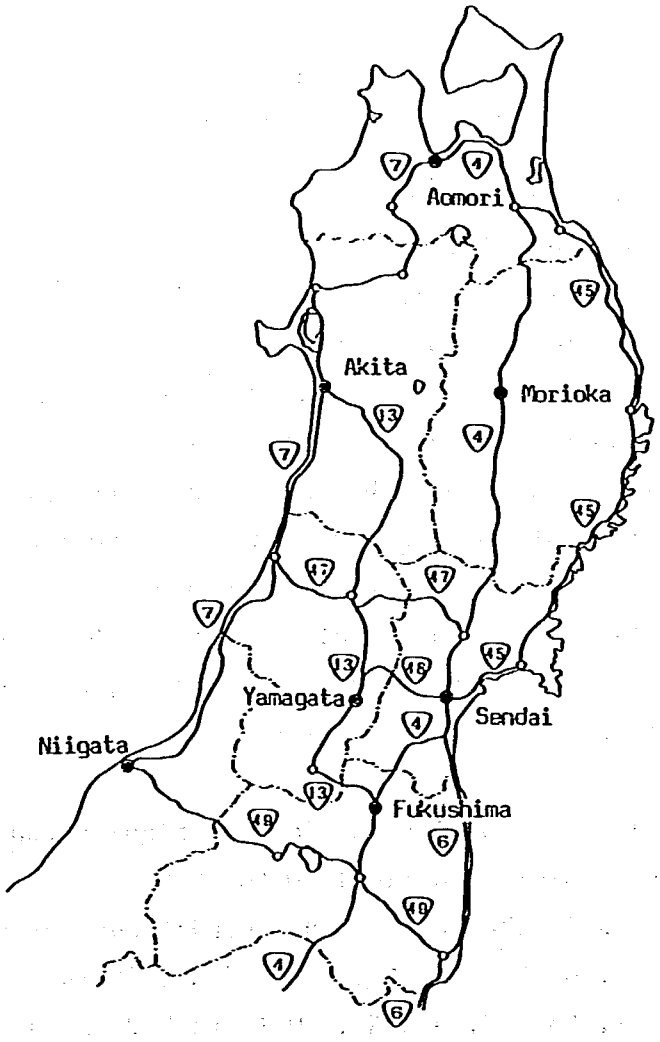


Fig. 1 National Highway Routes in Administrative Area of Tohoku Regional Construction Bureau

4/ TRANSITION PROBABILITY MATRIX WHEN REPAIRS NOT MADE

The PSI (Present Serviceability Index) given in the Manual for Road Maintenance and Repair [2] was employed as the serviceability index of asphalt pavements. This is as expressed by the following equation:

$$PSI = 4.53 - 0.518 \log \sigma - 0.371 \sqrt{C} - 0.174 D^2 \quad \cdots\cdots\cdots\cdots\cdots\cdots (12)$$

where, σ : standard deviation of evenness in longitudinal direction (mm)

C : cracking ratio (%)

D : average depth of rutting (cm)

The conditions of pavements were evaluated according to the five ranks below using this PSI.

Rank	1	2	3	4	5
PSI	4 -	3 - 4	2 - 3	1 - 2	- 1

Since the data employed in this study were obtained at intervals of 3 to 4 years, annual transition probability matrices were prepared by the following method. The initial value of the transition probability matrix P is given in Eq. (4), the sum of the rows in matrix $N^{(M)}$ of the number of times of transition during M number of years of data when repairs are not made is allotted to each row and each column by the matrix of the M-th power of P, and the matrix $N^{(M)'}$ of the estimated number of times of transition is prepared. Then, the individual elements of P are varied by minute increments while satisfying the condition of Eq. (2), the matrices by which the sum of squares of the differences between the individual elements of $N^{(M)}$ and $N^{(M)'}$ become least squares are obtained by repetitive calculations, and these are taken to be the annual transition probability matrices.

Table 1 gives transition probability matrices by highway route for the case of Class C traffic (large vehicle traffic 1000 to 3000 / day, one way). According to this, the transitions in the road surface conditions may be classified as two groups of routes No. 6, No. 45, No, 47, and No. 48 in the range of rank 1, and routes No. 4, No. 7, No. 13, and No. 49 in rank 2 and higher. Of the former, No. 6 and No. 45, especially, are routes on the Pacific Ocean side of comparatively mild climate, while the latter are routes located in cold areas

where snow cover occurs. These routes in snow-coverd, cold areas indicate the influences of the effects of wearing by studded tires and chained tires.

Table 1 Annual Transition Probability Matrices

No. 4

$$\begin{bmatrix} 0.113 & 0.250 & 0.462 & 0.153 & 0.022 \\ 0 & 0.557 & 0.254 & 0.143 & 0.046 \\ 0 & 0 & 0.698 & 0.302 & 0.000 \\ 0 & 0 & 0 & 0.666 & 0.334 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No. 6

$$\begin{bmatrix} 0.388 & 0.612 & 0 & 0 & 0 \\ 0 & 0.629 & 0.371 & 0 & 0 \\ 0 & 0 & 0.833 & 0.135 & 0.032 \\ 0 & 0 & 0 & 0.870 & 0.130 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No. 7

$$\begin{bmatrix} 0.458 & 0.342 & 0.200 & 0 & 0 \\ 0 & 0.514 & 0.385 & 0.101 & 0 \\ 0 & 0 & 0.629 & 0.371 & 0 \\ 0 & 0 & 0 & 0.234 & 0.766 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No.13

$$\begin{bmatrix} 0.441 & 0.319 & 0.213 & 0.027 & 0 \\ 0 & 0.667 & 0.271 & 0.000 & 0.062 \\ 0 & 0 & 0.743 & 0.245 & 0.012 \\ 0 & 0 & 0 & 0.890 & 0.110 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No.45

$$\begin{bmatrix} 0.473 & 0.526 & 0.000 & 0.001 & 0 \\ 0 & 0.753 & 0.221 & 0.018 & 0.008 \\ 0 & 0 & 0.787 & 0.213 & 0.000 \\ 0 & 0 & 0 & 0.773 & 0.227 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No.47

$$\begin{bmatrix} 0.392 & 0.608 & 0 & 0 & 0 \\ 0 & 0.597 & 0.326 & 0.000 & 0.077 \\ 0 & 0 & 0.625 & 0.357 & 0.018 \\ 0 & 0 & 0 & 0.723 & 0.277 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No.48

$$\begin{bmatrix} 0.410 & 0.591 & 0 & 0 & 0 \\ 0 & 0.641 & 0.359 & 0 & 0 \\ 0 & 0 & 0.594 & 0.406 & 0 \\ 0 & 0 & 0 & 0.440 & 0.560 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

No.49

$$\begin{bmatrix} 0.361 & 0.491 & 0.110 & 0.000 & 0.038 \\ 0 & 0.446 & 0.515 & 0.000 & 0.039 \\ 0 & 0 & 0.464 & 0.536 & 0.000 \\ 0 & 0 & 0 & 0.646 & 0.354 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

5/ RELIABILITY OF PAVEMENT AND EXPECTATION OF PERFORMANCE SERVICE LIFE

From Eq. (4), since the transition probabilities of m years later can be determined by multiplying to the m-th power the transition probability matrix of Table 1, it is possible to obtain the probability that rank 5 (ultimate damaged state) will not be reached, namely, reliability. Table 2 gives the reliability at 5 and 10 years of service in case of Class C traffic.

Table 2 Reliability

Years of Service	Route No.							
	4	6	7	13	45	47	48	49
5	0.562	0.948	0.609	0.852	0.948	0.767	0.851	0.670
10	0.160	0.712	0.108	0.594	0.677	0.326	0.249	0.143

Regarding the same data, when the number of years that the condition of a pavement remains at a single rank, and the expectation of service life determined as the aggregate values are obtained by Eq. (6), the results will be as given in Table 3. Whereas the service lives of routes No. 6 and No. 45 located at the Pacific Ocean side were around 15 years, the pavements of other routes except for No. 13 were all 10 years or under.

Table 3 Expectation of Pavement Service Lives

No. 4 (years)

$$\begin{bmatrix} 1.13 & 0.64 & 2.26 & 2.83 \\ 0 & 2.26 & 1.90 & 2.68 \\ 0 & 0 & 3.31 & 2.99 \\ 0 & 0 & 0 & 2.99 \end{bmatrix} \quad \begin{matrix} 6.86 \\ 6.84 \\ 6.31 \\ 2.99 \end{matrix}$$

No. 45 (years)

$$\begin{bmatrix} 1.90 & 4.04 & 4.20 & 4.26 \\ 0 & 4.05 & 4.21 & 4.26 \\ 0 & 0 & 4.71 & 4.40 \\ 0 & 0 & 0 & 4.40 \end{bmatrix} \quad \begin{matrix} 14.40 \\ 12.52 \\ 9.11 \\ 4.40 \end{matrix}$$

No. 6 (years)

$$\begin{bmatrix} 1.64 & 2.69 & 6.00 & 6.22 \\ 0 & 2.69 & 6.00 & 6.22 \\ 0 & 0 & 6.00 & 6.22 \\ 0 & 0 & 0 & 7.69 \end{bmatrix} \quad \begin{matrix} 16.55 \\ 14.92 \\ 12.23 \\ 7.69 \end{matrix}$$

No. 47 (years)

$$\begin{bmatrix} 1.65 & 2.48 & 2.15 & 2.78 \\ 0 & 2.48 & 2.15 & 2.78 \\ 0 & 0 & 2.67 & 3.44 \\ 0 & 0 & 0 & 3.61 \end{bmatrix} \quad \begin{matrix} 9.06 \\ 7.42 \\ 6.10 \\ 3.61 \end{matrix}$$

No. 7 (years)

$$\begin{bmatrix} 1.85 & 1.30 & 2.34 & 1.31 \\ 0 & 2.06 & 2.14 & 1.31 \\ 0 & 0 & 2.69 & 1.31 \\ 0 & 0 & 0 & 1.31 \end{bmatrix} \quad \begin{matrix} 6.79 \\ 5.50 \\ 4.00 \\ 1.31 \end{matrix}$$

No.48 (years)

$$\begin{bmatrix} 1.69 & 2.78 & 2.47 & 1.78 \\ 0 & 2.78 & 2.47 & 1.78 \\ 0 & 0 & 2.47 & 1.78 \\ 0 & 0 & 0 & 1.78 \end{bmatrix} \quad \begin{matrix} 8.73 \\ 7.03 \\ 4.25 \\ 1.78 \end{matrix}$$

No.13 (years)

$$\begin{bmatrix} 1.79 & 1.71 & 3.29 & 7.74 \\ 0 & 3.00 & 3.17 & 7.03 \\ 0 & 0 & 3.89 & 8.64 \\ 0 & 0 & 0 & 9.06 \end{bmatrix} \quad \begin{matrix} 14.53 \\ 13.20 \\ 12.53 \\ 9.06 \end{matrix}$$

No.49 (years)

$$\begin{bmatrix} 1.57 & 1.39 & 1.65 & 2.50 \\ 0 & 1.81 & 1.73 & 2.62 \\ 0 & 0 & 1.87 & 2.82 \\ 0 & 0 & 0 & 2.82 \end{bmatrix} \quad \begin{matrix} 7.11 \\ 6.17 \\ 4.69 \\ 2.82 \end{matrix}$$

6/ TRANSITION PROBABILITY MATRICES CONSIDERING REPAIRS

Up to this point, cases of pavements not undergoing repairs were considered. Actually, however, repairs are carried out as pavements approach failure and the pavements are made to recover to ranks higher than their current conditions. Seeking the transient probability matrices considering the repair records of the various routes in case of Class C traffic (Table 4) by Eq. 10, the results are as given in Table 5. The repair probability r_{ik} and repair effect f_{kj} were obtained by analyses from the data file. The repair effects by route are as shown in Table 6.

The stationary probability vectors obtained by Eq. (7) from transition probability matrices considering the repair records in Table 3 are as given in Table 7. On the other hand, obtaining the condition probabilities of pavements at the various routes based on the data file and comparing the two, the results are as shown in Fig. 2. Although there are slight differences between the two, they are more or less in agreement.

Table 4 Ratios of Repairs Performed

Repair Method	Route No.							
	4	6	7	13	45	47	48	49
Overlay	0.106	0.101	0.074	0.105	0.102	0.062	0.054	0.107
Surface Treatment	0.110	0.028	0.129	0.150	0.055	0.119	0.116	0.046
Total	0.216	0.129	0.203	0.255	0.157	0.181	0.170	0.153

Table 5 Transition Probability Matrices Considering Repair Performance

No. 4

$$\begin{bmatrix} 0.116 & 0.254 & 0.457 & 0.152 & 0.021 \\ 0.031 & 0.553 & 0.240 & 0.133 & 0.043 \\ 0.072 & 0.108 & 0.573 & 0.245 & 0.002 \\ 0.121 & 0.165 & 0.029 & 0.456 & 0.229 \\ 0.257 & 0.305 & 0.052 & 0.010 & 0.376 \end{bmatrix}$$

No.45

$$\begin{bmatrix} 0.474 & 0.525 & 0.000 & 0.001 & 0 \\ 0.046 & 0.724 & 0.205 & 0.018 & 0.007 \\ 0.108 & 0.072 & 0.645 & 0.174 & 0.001 \\ 0.175 & 0.124 & 0.008 & 0.535 & 0.158 \\ 0.354 & 0.269 & 0.015 & 0.006 & 0.356 \end{bmatrix}$$

No. 6

$$\begin{bmatrix} 0.390 & 0.610 & 0 & 0 & 0 \\ 0.031 & 0.615 & 0.351 & 0.001 & 0.002 \\ 0.069 & 0.052 & 0.726 & 0.119 & 0.034 \\ 0.127 & 0.083 & 0.006 & 0.675 & 0.109 \\ 0.299 & 0.162 & 0.015 & 0.008 & 0.516 \end{bmatrix}$$

No. 7

$$\begin{bmatrix} 0.459 & 0.343 & 0.198 & 0 & 0 \\ 0.039 & 0.506 & 0.359 & 0.094 & 0.002 \\ 0.094 & 0.083 & 0.517 & 0.302 & 0.004 \\ 0.144 & 0.126 & 0.016 & 0.170 & 0.544 \\ 0.269 & 0.230 & 0.027 & 0.009 & 0.465 \end{bmatrix}$$

No.13

$$\begin{bmatrix} 0.436 & 0.325 & 0.212 & 0.027 & 0 \\ 0.027 & 0.658 & 0.258 & 0.002 & 0.055 \\ 0.066 & 0.184 & 0.564 & 0.175 & 0.011 \\ 0.103 & 0.286 & 0.079 & 0.470 & 0.062 \\ 0.196 & 0.544 & 0.145 & 0.013 & 0.102 \end{bmatrix}$$

No.47

$$\begin{bmatrix} 0.395 & 0.603 & 0.001 & 0.001 & 0 \\ 0.065 & 0.574 & 0.289 & 0.004 & 0.068 \\ 0.155 & 0.127 & 0.446 & 0.259 & 0.013 \\ 0.245 & 0.188 & 0.014 & 0.404 & 0.149 \\ 0.475 & 0.330 & 0.023 & 0.023 & 0.149 \end{bmatrix}$$

No.48

$$\begin{bmatrix} 0.412 & 0.587 & 0.001 & 0 & 0 \\ 0.057 & 0.617 & 0.326 & 0 & 0 \\ 0.138 & 0.101 & 0.456 & 0.305 & 0 \\ 0.213 & 0.150 & 0.011 & 0.275 & 0.351 \\ 0.404 & 0.265 & 0.018 & 0.000 & 0.313 \end{bmatrix}$$

No.49

$$\begin{bmatrix} 0.362 & 0.489 & 0.111 & 0 & 0.038 \\ 0.028 & 0.438 & 0.495 & 0.002 & 0.037 \\ 0.062 & 0.044 & 0.430 & 0.464 & 0 \\ 0.114 & 0.075 & 0.046 & 0.497 & 0.268 \\ 0.268 & 0.161 & 0.072 & 0.013 & 0.486 \end{bmatrix}$$

Table 6 Repair Effects

Route No.	Repair Method k	Rank i				
		1	2	3	4	5
4	Overlay	0.548	0.381	0.053	0.000	0.018
	Surface Treatment	0.212	0.625	0.125	0.038	0.000
6	Overlay	0.714	0.232	0.036	0.018	0.000
	Surface Treatment	0.000	0.833	0.000	0.000	0.167
7	Overlay	0.547	0.413	0.033	0.007	0.000
	Surface Treatment	0.456	0.428	0.062	0.023	0.031
13	Overlay	0.229	0.625	0.146	0.000	0.000
	Surface Treatment	0.204	0.578	0.176	0.028	0.014
45	Overlay	0.505	0.475	0.010	0.010	0.000
	Surface Treatment	0.666	0.257	0.057	0.010	0.010
47	Overlay	0.722	0.278	0.000	0.000	0.000
	Surface Treatment	0.429	0.475	0.048	0.048	0.000
48	Overlay	0.708	0.292	0.000	0.000	0.000
	Surface Treatment	0.500	0.455	0.045	0.000	0.000
49	Overlay	0.676	0.324	0.000	0.000	0.000
	Surface Treatment	0.000	0.278	0.611	0.111	0.000

Table 7 Stationary Probabilities of Pavements

Route No.	4	X^* = [	0.096	0.281	0.289	0.228	0.106]
	No. 6	X^* = [	0.118	0.294	0.385	0.143	0.060]
	No. 7	X^* = [	0.174	0.268	0.282	0.135	0.141]
	No. 13	X^* = [	0.094	0.423	0.329	0.116	0.038]
	No. 45	X^* = [	0.189	0.425	0.227	0.104	0.055]
	No. 47	X^* = [	0.152	0.440	0.259	0.115	0.034]
	No. 48	X^* = [	0.174	0.414	0.252	0.106	0.054]
	No. 49	X^* = [	0.140	0.217	0.253	0.239	0.151]

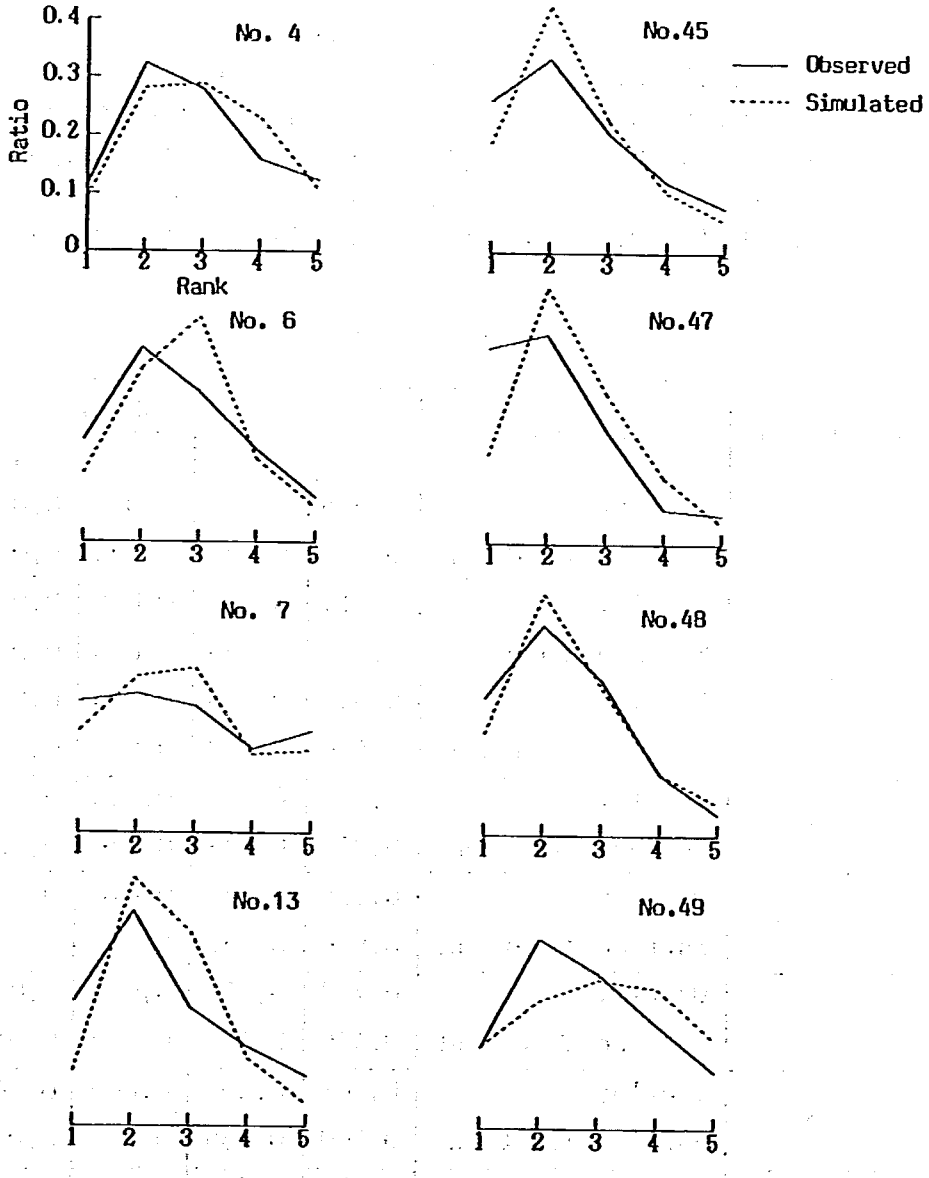


Fig. 2 Stationary Probabilities

7/ CONCLUSION

In order to maintain a pavement in an agreeable condition economically, it is necessary to predict the transition of the pavement surface condition resulting from use in traffic, and further, to construct a system by which the various characteristic values concerning damage to the pavement can be evaluated. In the present research, a data file was prepared based on the data of pavement performance surveys periodically carried out by the Ministry of Construction, and evaluations of performance of asphalt pavements were attempted by applying a Markov chain model to this file. Topics remaining for the future study are making the data file more complete and constructing a dynamic system for analyses of the economics.

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A Generic Framework for Pavement Management and Staging Modules for Implementation

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A GENERIC FRAMEWORK FOR PAVEMENT MANAGEMENT AND STAGING MODULES FOR IMPLEMENTATION

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ABSTRACT

Two decades of pavement management implementation experience suggest that a generic framework exists within which the technology can be organized and which allows complete flexibility for tailoring different models, methods and procedures to individual authorities. As well, this experience suggests that effective application requires a good data base and staging of the implementation.

Five major modules, for the network level of pavement management and based on a generic framework, are identified in the paper. They are: (a) data base, (b) needs analysis, (c) rehabilitation priority programming, (d) maintenance planning and programming and (e) combined maintenance and rehabilitation programming. These modules have been designed for staged implementation and to have useable products at the end of each stage.

The paper presents several example products associated with the staging modules. First, a present status distribution of Pavement Quality Index for a Canadian city is shown for the first module. Then a needs distribution for the same city is shown as a typical product. Alberta's Municipal Pavement Management System (MPMS), which is very comprehensive and completely flexible for tailoring to particular requirements and resources, is used as the context for most of the examples. Finally, rehabilitation and maintenance priority program examples are presented.

The paper concludes that the provision for staging, and the associated flexibility of timing, are important in tailoring a pavement management system to the specific needs and resources of individual authorities.

1. INTRODUCTION

Pavement management has progressed from a concept to a widely accepted process during the past two decades. The principles have been formulated and much has been learned from implementation experience at federal, provincial/state and local levels in various countries.

Several basic reasons exist for the extensive interest in pavement management systems, including the following: such systems offer the most efficient way of getting value for money spent, they explicitly recognize the tremendous transport infrastructure investment represented by pavements, and it has been demonstrated that they can work, be understandable and assist the decision making of their users.

This paper builds on two decades of progress in pavement management system (PMS) development and implementation by describing the generic structure of the process and the staging modules for implementation. It also provides example applications from Canadian experience. More specifically, the objectives of the paper are as follows:

1. To summarize some of the background experience of two decades of pavement management implementation.

2. To identify the requirements for an effective, operational PMS, and the generic structure for application of the process.

3. To define a set of modules from that generic structure which are amenable to staged implementation.

4. To provide examples of typical products or outputs from each stage.

The intent of the paper is to show how the foregoing can be used to tailor a PMS to the specific needs and resources of individual authorities. It is limited in scope to applications, rather than detailed description of the underlying methodology. Some of those details can be found in the references cited.

2. BASIC REQUIREMENTS AND GENERIC STRUCTURE FOR AN EFFECTIVE AND COMPREHENSIVE PMS

2.1 Background Experience from Development and Implementation

It is useful to briefly review what has been learned from two decades of pavement management before identifying the basic requirements. The first is that the original concept of a comprehensive, systematic process is quite valid. That is, it incorporates in an organized way all the activities that go into providing and operating pavements, ranging from the collection, processing and analysis of field and other data on various pavement sections, the identification of current and future needs, and the development of rehabilitation and maintenance, programs to the implementation of these programs through design, construction and maintenance (1).

What perhaps has not been so well learned is that just because an authority has carried out all these activities does not mean they have a PMS. To have a proper PMS requires a coordinated execution of these activities, and most importantly the incorporation of a number of key elements (such as performance or deterioration models, life-cycle economic evaluation, etc., as subsequently described).

Table 1 summarizes, in more specific terms, some of the key things we have learned about the P.M. process itself and about its application.

2.2 Requirements for an Effective, Operational PMS

Pavement management is no different in its general requirements than any other area of management. It involves the coordinated direction of resources and labor to achieve a desired end. Decision making at various levels is therefore a primary activity. This can only be effective if good information is available.

TABLE 1

SOME KEY THINGS LEARNED FROM TWO DECADES
OF PAVEMENT MANAGEMENT EXPERIENCE

1. A generic framework exists for pavement management. The technology can be effectively organized within this framework; moreover it allows complete flexibility for tailoring different models, methods and procedures to specific needs.

2. A sound technological base is critical to the effective application of pavement management, as is a good data base.

3. Pavement management operates at two basic levels: network and project.

4. Staging is an essential requisite in the development and implementation of a PMS, and it provides for better understanding and acceptance by the people in the authority.

5. Pavement management can make effective use of available funds but it cannot rescue a network if the funding is below some threshold level.

6. Pavement management can be equally applicable and effective in small authorities as in those of medium or large size.

Several basic requirements exist for the effective application of a PMS. They include the following (2):

1. Serving different types of users in the organization, (i.e., elected level, senior administrative and technical),

2. Making good decisions regarding network programs and individual projects, and executing these decisions in a timely manner,

3. Making good use of the existing technology, and new technology as it becomes available.

An essential part of fulfilling the foregoing requirements is to have a structure or framework for the various activities of pavement management. Such a structure should be generically based so that it could be used to organize the PMS activities of any individual authority. Moreover, it should be comprehensive enough to incorporate all P.M. activities at both network and project levels.

2.3 Generic Structure for Pavement Management

The major activities and/or decisions made within the total spectrum of pavement management are listed in Table 2 (3).

The actual system used by any particular authority will cover part or all of the components of Table 2. As well, such actual systems incorporate the specific methods, models and procedures, plus the sequence of and relationships between activities, which make them operational. Examples will subsequently be described.

Staging the development and implementation of a PMS represents a modular, sequential coverage of the components of Table 2. Such staging makes good sense for most authorities esource considerations, gaining familiarity with the system in a progressive, step by step way, and improving the chances for acceptability by the people involved.

3. STAGING MODULES FOR PMS DEVELOPMENT AND IMPLEMENTATION

The rationale behind staging PMS development and implementation seems to have been accepted by many authorities. Given that, the question then rises as to what modules make sense for such staging, particularly so that a useable product results at the end of each one.

An example of staging modules, typical products and their relationship to the framework and activities of Table 2 is given in Table 3 for the network

TABLE 2

A GENERIC STRUCTURE FOR PAVEMENT MANAGEMENT WHICH SUMMARIZES THE MAJOR COMPONENT ACTIVITIES AND DECISIONS

Basic Blocks of Activities	Network Management Level (Administrative and Technical Decisions)	Project Management Level (Technical Management Decisions)
A. Data	1 Sectioning 2 Data Acquisition a) Field inventory (roughness, surface distress, friction, deflect., geom. b) Other (traffic costs) 3 Data Processing	1 Detailed data (structural, materials, traffic, climate, and unit costs) 2 Subsectioning 3 Data Processing
B. Criteria	1 Min. serviceability, friction, and structural adequacy, max. distress 2 Max. user costs, maint. costs 3 Max. program costs 4 Selection criteria (benefit max.; cost-effectiveness)	1 Max. as-built roughness; min. structural adequacy and friction 2 Max. project costs 3 Max. traffic interruption 4 Selection criteria (least total costs)
C. Analyses	1 Network needs (now) 2 Perf. predictions and future needs 3 Maint. and rehab. altern. 4 Tech. and econ. eval. 5 Priority analysis 6 Eval. of altern. budgets	1 Within-project alternatives 2 Testing and 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 Maint. treatments for sections in networks
E. Implementation	1 Schedule, contracts 2 Program monitoring 3 Budget and financial planning updates	1 Construction activities, contract control, and as-built records 2 Maint. activities, maint. management and records

TABLE 3

STAGING MODULES (NETWORK LEVEL) FOR PMS DEVELOPMENT AND IMPLEMENTATION

Module	Typical End Products	Coverage in Table 2
• Data base	• Present status reports (listings, histograms, sorts, etc.) of surface distress, structural adequacy, Riding Comfort Index (RCI), Pavement Quality Index (PQI), curb and gutter condition, sidewalk condition, drainage, geometrics, etc. • History of accidents, maintenance, traffic, etc.	A1 to A3
• Needs analysis	• Identification of present needs and future needs (by section, histogram summaries, etc.) • Performance curves by section or for groups of similar sections/ pavements	B1, C1 and C2 (plus A1 to A3)
• Rehabilitation Priority Programming	• Identification of feasible rehabilitation alternatives • Priority programs of rehabilitation projects (by year, by road class, by district, by road link, etc.) • Budget levels analysis (effects of different funding levels) • Financial plan (for specified level of service, maximum amount of deficient mileage, etc.)	B2 to B4, C3 to C6, and D1 (plus above)
• Maintenance Planning and Programming	• Identification of feasible maintenance alternatives • Budget level analysis	D1 (plus A1 to A3, B1 to B4, C3 to C6)
• Combined Maintenance and Rehabilitation Programming	• Identification of feasible alternatives • Integrated priority programs of maintenance and rehabilitation • Budget level analysis	All of the above

level of pavement management. This is drawn from experience. The rate at which these modules are developed and implemented varies widely, depending largely on resources, size of the authority and its needs, and the people involved. Generally, however, the time period for each is between a half to one and a half years.

With the availability of modules such as those summarized in Table 3, a road authority is able to tailor both the extent and timing to their needs and resources. For example, a small authority may for some period of time decide to only implement the first two modules: data base and needs analysis. That would give them the necessary information capabilities to identify which sections are deficient in which years. Then the appropriate maintenance and rehabilitation might be scheduled on a local experience and judgment basis rather than based on a priority analysis. Except for quite small networks, however, this can result in programs of work that differ significantly from what would be optimum.

An example of staged implementation is Alberta's "Pavement Information and Needs System" (PINS), which covered the first two modules of Table 3, and their "Rehabilitation Information and Priority Programming System" (RIPPS), which added the third module of Table 3 (4,5).

After the Provincial PMS was implemented, Alberta also developed a very comprehensive "Municipal Pavement Management System" (MPMS) which is available for all their cities (6). These range in population from about 10,000 to 600,000. The system covers all the modules of Table 3; as well, there is a project level design and analysis system included in MPMS. Figure 1 gives an overview of the system. Examples to follow are largely drawn from the application of MPMS to various cities, and they illustrate the products listed in Table 3.

4. EXAMPLES OF END PRODUCTS FROM THE MODULES

Most of the following examples will be drawn from application of Alberta's MPMS, largely because it is comprehensive, flexible and incorporates current state of technology methods and procedures. The examples start with the first module and show how the data base can be used to generate

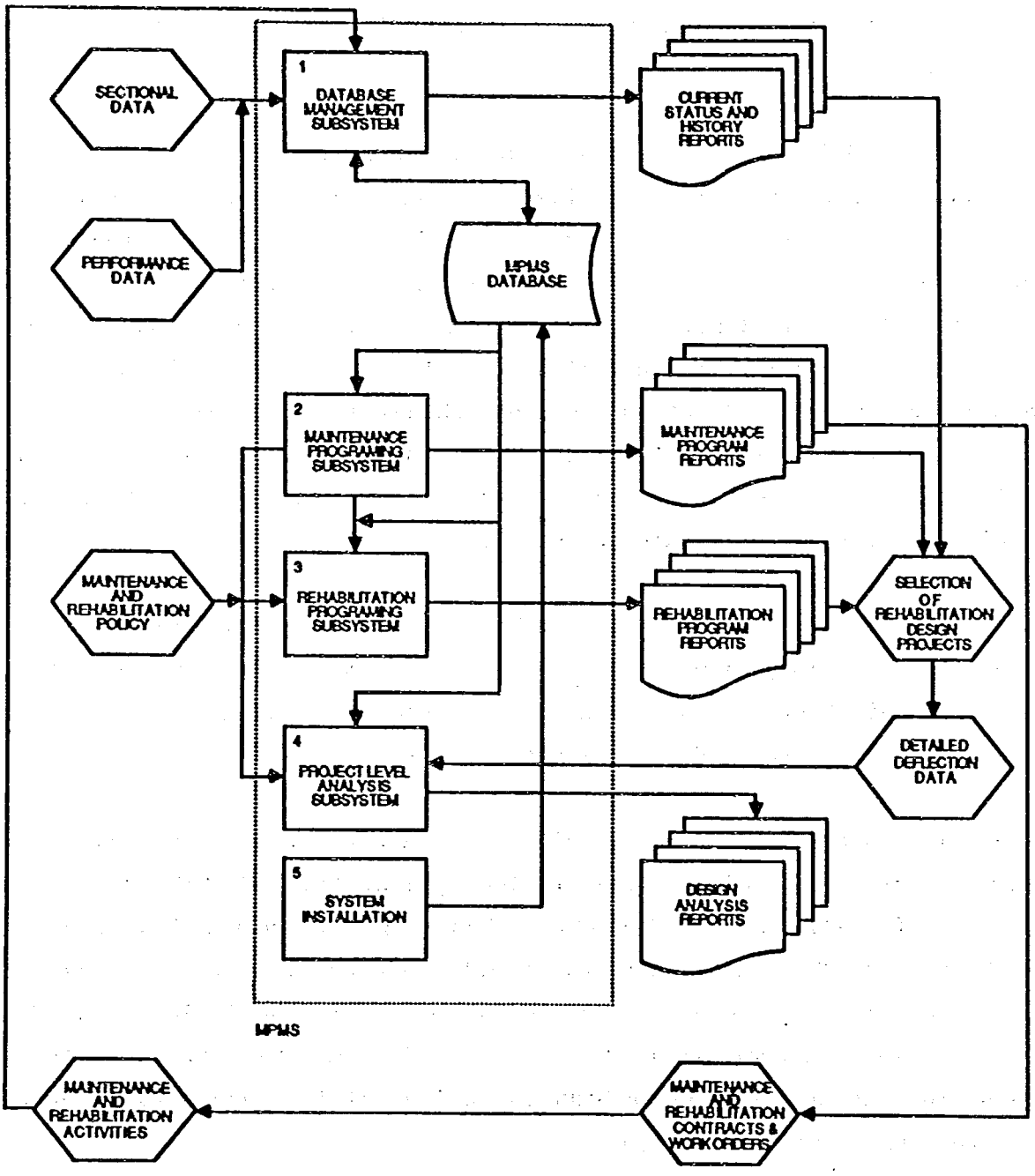


Figure 1 Overview of Alberta's Municipal Pavement Management System (MPMS)

useful products such as present status reports. Subsequent examples cover the progressive addition of modules.

4.1 Example Use of a Data Base to Generate Present Status Reports

The detailed data on the various sections in a pavement management data base form the basis for the widely used summary measures of Riding Comfort Index (RCI), Structural Adequacy Index (SAI), Visual Condition Index (VCI) and a combined Pavement Quality Index (PQI), all on a scale of 0 to 10 (4,5). Detailed records of these measures for each section, in tabular form, along with such other data as traffic volumes, would be normally generated. As well, summaries in histogram form are very useful, such as the example for PQI distribution of one city with 476 lane-km of arterial streets, composed of 390 sections, shown in Figure 2. Similar distributions for the components of PQI (RCI, SAI and VCI) would also normally be generated.

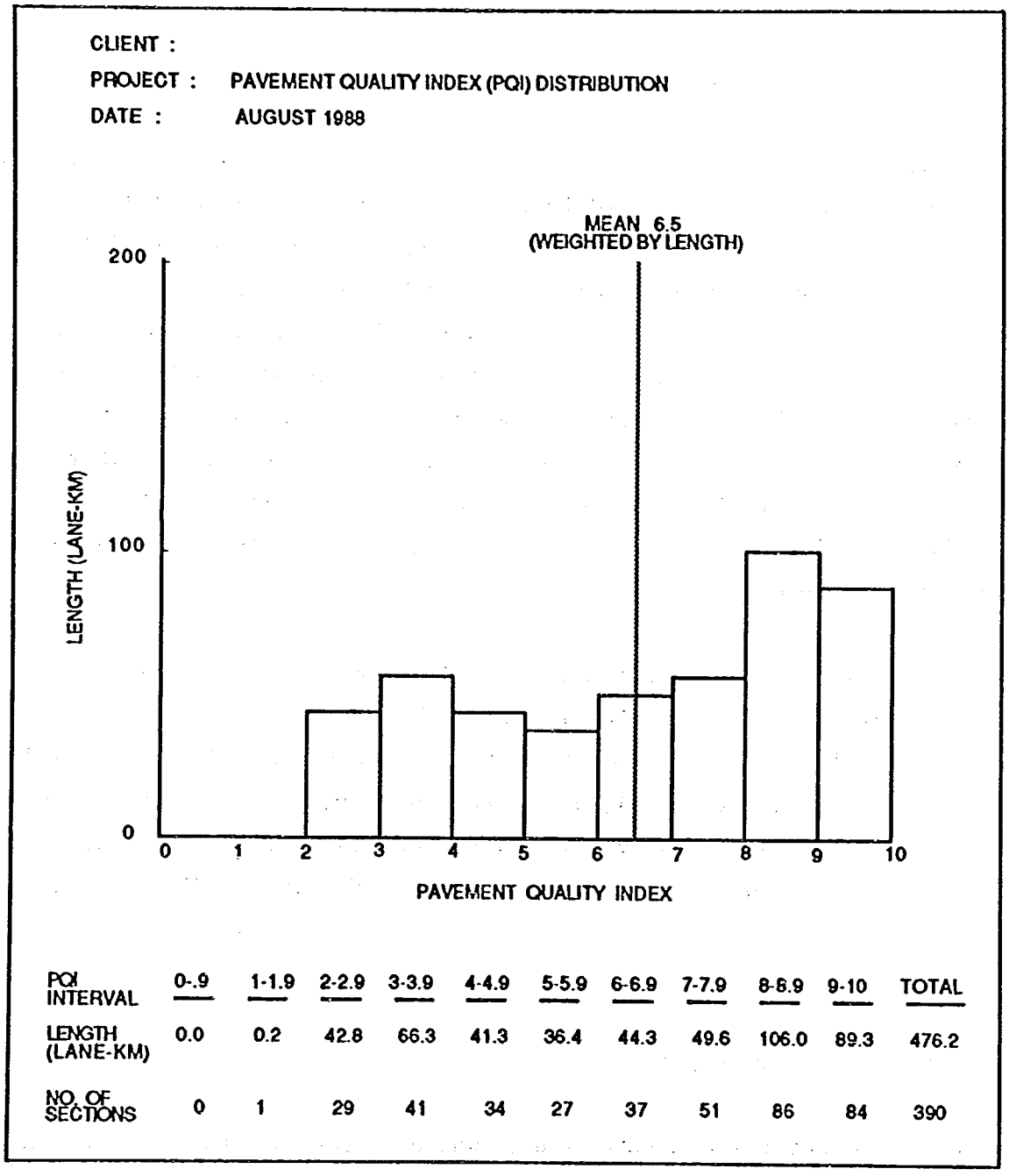
A major reason for the relatively flat PQI distribution of this particular city is a lot of sections with low SAI (i.e., weak subgrades exist). It should be emphasized, however, that the component distributions, and the composite PQI, can vary widely from city to city, depending on age of the network, the amount of money previously put into rehabilitation, traffic conditions, degree of maintenance performed, subgrade conditions, climate, etc.

4.2 Example Needs Analysis

In order to determine needs—i.e., which sections are or will become deficient—the following is required:

1. Decision on what measure(s) to base needs (RCI, SAI, VCI or PQI).

2. Designation of the minimum acceptable level(s) for these measures (i.e., B1 of Table 2).



PQI INTERVAL	0-.9	1-1.9	2-2.9	3-3.9	4-4.9	5-5.9	6-6.9	7-7.9	8-8.9	9-10	TOTAL
LENGTH (LANE-KM)	0.0	0.2	42.8	66.3	41.3	36.4	44.3	49.6	106.0	89.3	476.2
NO. OF SECTIONS	0	1	29	41	34	27	37	51	86	84	390

Figure 2 Example summary of PQI distribution

3. Selection of a planning or programming period over which the needs will be identified (i.e., 5 years, 10 years, etc.).

4. Performance prediction models for RCI, SAI, VCI and PQI so that future needs can be identified.

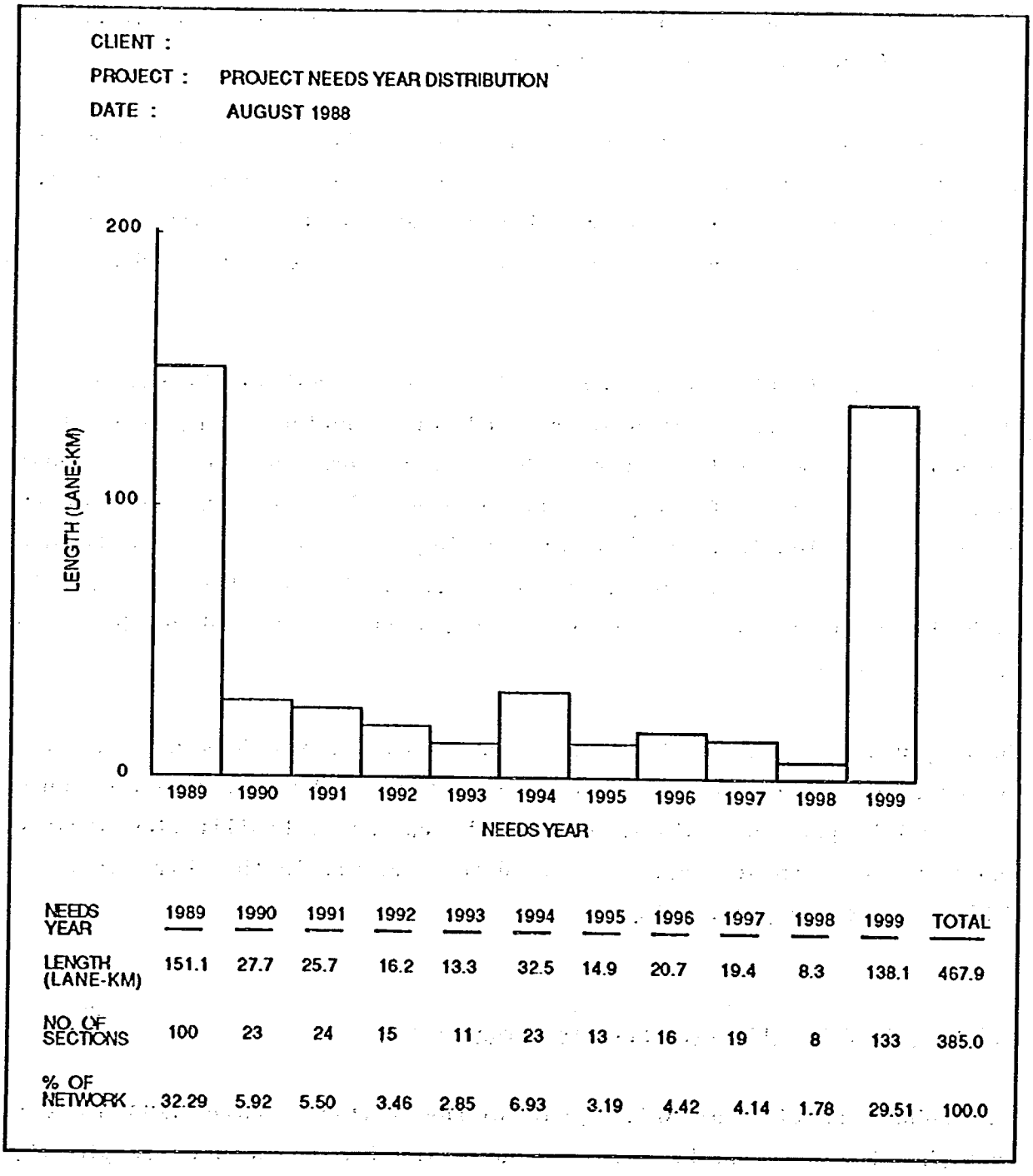
The first three requirements are usually met on the basis of past practice and judgment as to what is appropriate for the particular municipality. Development of performance prediction models is a most important requirement because the identification of future needs years depends on them.

An example application of needs analysis to the network of the same city of the previous example is given by the needs distribution of Figure 3. It is based on a minimum PQI level of 4.5. The performance prediction models used are not shown. The high amount of needs for the first year of the programming period is typical of the backlog that exists for many Canadian cities. Unless these needs can all be financed in that first year, which is highly unlikely, those that are not met will be deferred to the second year, and so on.

Knowing what sections will become needs in what years does not, however, indicate which sections should be rehabilitated and which deferred, under the usual budget constraints, or what the optimum rehabilitation strategy should be for each section. That is determined within the third module of Table 3, priority programming.

4.3 Example Rehabilitation Priority Programming

The rehabilitation priority programming system within MPMS is very comprehensive and can be used as the basis for an actual example (7). First, the overall structure of this subsystem (3 in Fig. 1) is given in Figure 4. It uses the MPMS data base for input plus maintenance needs analysis (subsequently described in more detail). As well, the subsystem incorporates a rehabilitation needs analysis (3.4 in Fig. 4); an example end product of this was described in the previous section.



NEEDS YEAR	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	TOTAL
LENGTH (LANE-KM)	151.1	27.7	25.7	16.2	13.3	32.5	14.9	20.7	19.4	8.3	138.1	467.9
NO. OF SECTIONS	100	23	24	15	11	23	13	16	19	8	133	385.0
% OF NETWORK	32.29	5.92	5.50	3.46	2.85	6.93	3.19	4.42	4.14	1.78	29.51	100.0

Figure 3 Example summary of a needs distribution

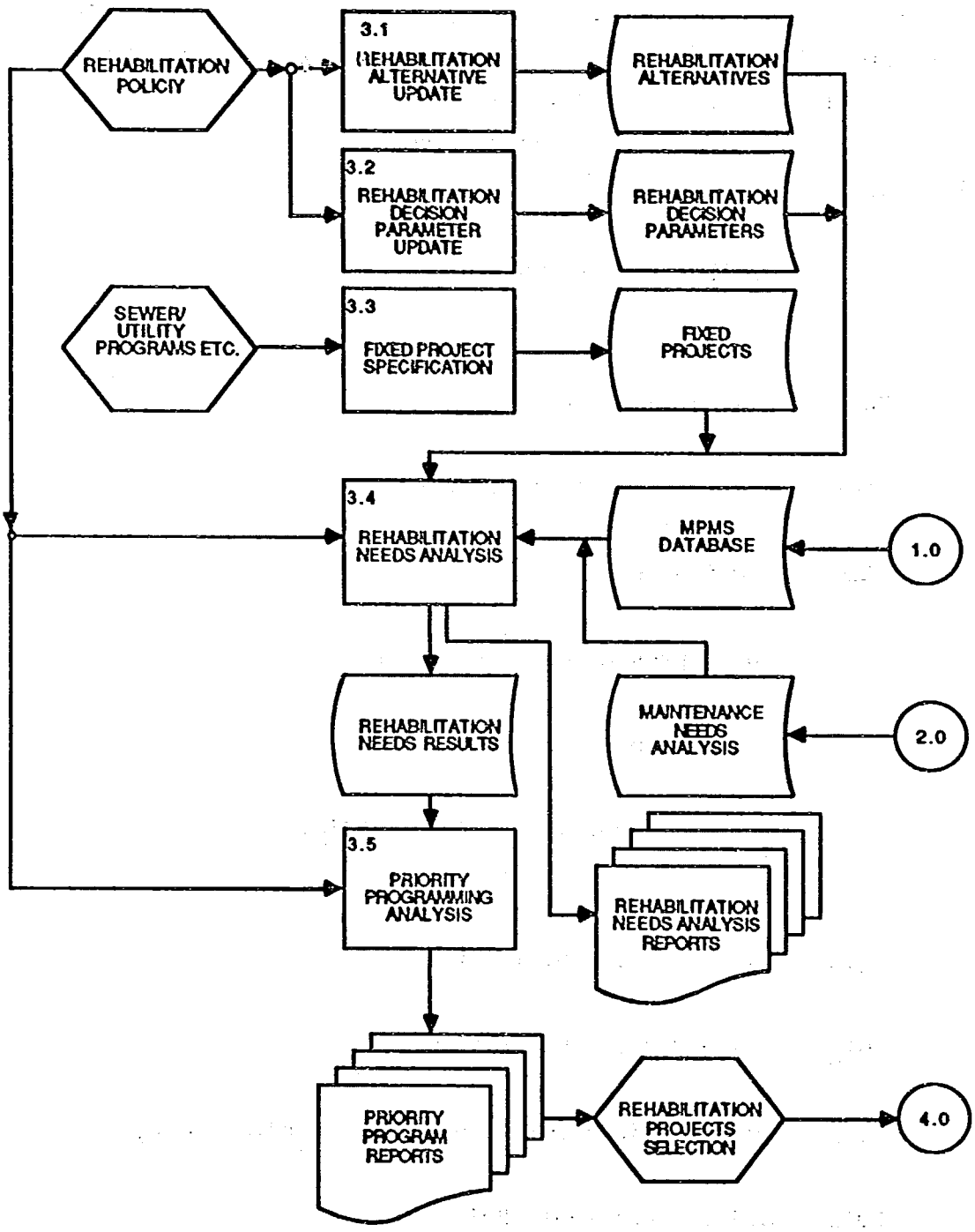


Figure 4 General structure of the rehabilitation programming subsystem in MPMS

At the heart of the subsystem is the priority programming analysis (3.5 in Fig. 4). It is quite similar to that described in Ref. (5) and includes performance models (see the example PQI curves in Figure 5 for seven rehabilitation alternatives on a soil cement base pavement), present worth of costs calculations for the alternatives, a maintenance cost model and a cost-effectiveness model. The latter model is based on the following calculation:

$$CE = 0.001 \cdot AADT \cdot LEN \cdot E/C_p$$

where

CE = cost effectiveness

AADT = average annual daily traffic on the section

LEN = section length

C_p = total present worth of costs

E = effectiveness (see Figure 6)

$$= \sum_i (PQI_R - PQI_L) - \sum_j (PQI_M - PQI_N)$$

PQI_N = PQI before rehabilitation

PQI_R = PQI after rehabilitation

PQI_L = PQI_N if $PQI_N > PQI_M$ and PQI_M if $PQI_M > PQI_N$

PQI_M = minimum acceptable PQI

i = years from implementation of the rehabilitation until $PQI_R \leq PQI_M$, and

j = years from need year to implementation year

The foregoing calculation is carried out for the alternatives for the needs year plus years in advance and years in deferral. In Figure 6, the alternative shown has been deferred.

For true optimization, all the possible combinations of section vs rehabilitation alternative vs implementation year should be compared. However, a near optimization technique, which has been shown to not be significantly different from true optimization (8), can be computationally much more efficient and is used in Alberta's MPMS. It starts with the strategy of highest cost-effectiveness (q) and then calculates the marginal cost-effectiveness of each strategy (1) which is mutually exclusive to q, as follows:

$$CE_{(1/q)} = \frac{E_1 - E_q}{C_{p1} - C_{pq}}$$

The marginal cost-effectiveness, $CE_{(1/q)}$, is then used for further comparisons and becomes the means for identifying the best strategy.

Application of the foregoing technique for priority analysis can be used to generate various reports, ranging from a listing of those streets programmed for a particular year (such as the example of Table 4) to a summary report (such as the example of Figure 7). In the particular example of Fig. 7, funds have only been programmed for the first 5 years, and the result for that period is clearly a substantial increase in the average PQI of the network (from 6.8 to 8.2) and decrease in the percent of the network mileage less than the minimum acceptable PQI (from 27% to 1%). Also shown, by dotted lines on the graphs, are the extreme results of no rehabilitation funding during the entire 10-year program period. Even for the case where funding is removed for the final five years, the effects can be seen in that the average PQI drops to 6.0 and the percent of network mileage less than the minimum acceptable rises to 19%.

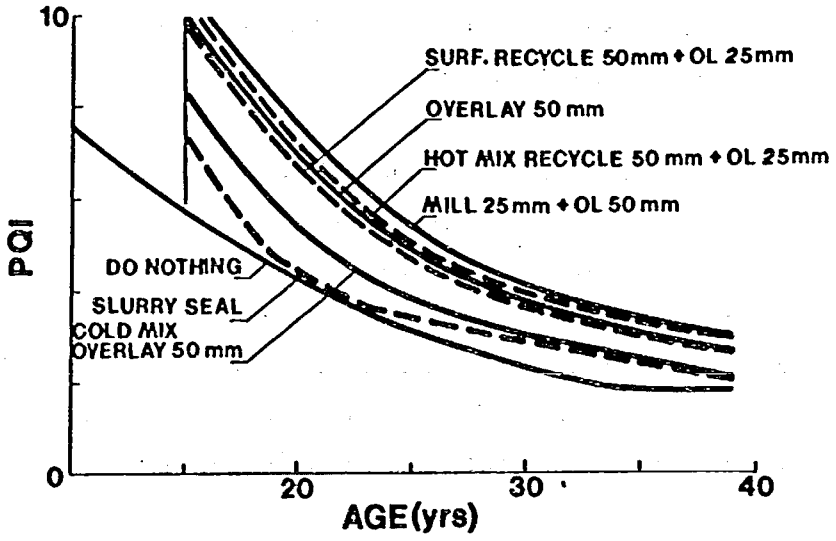


Figure 5 Example performance curves for soil-cement base pavements, from Alberta's MPMS

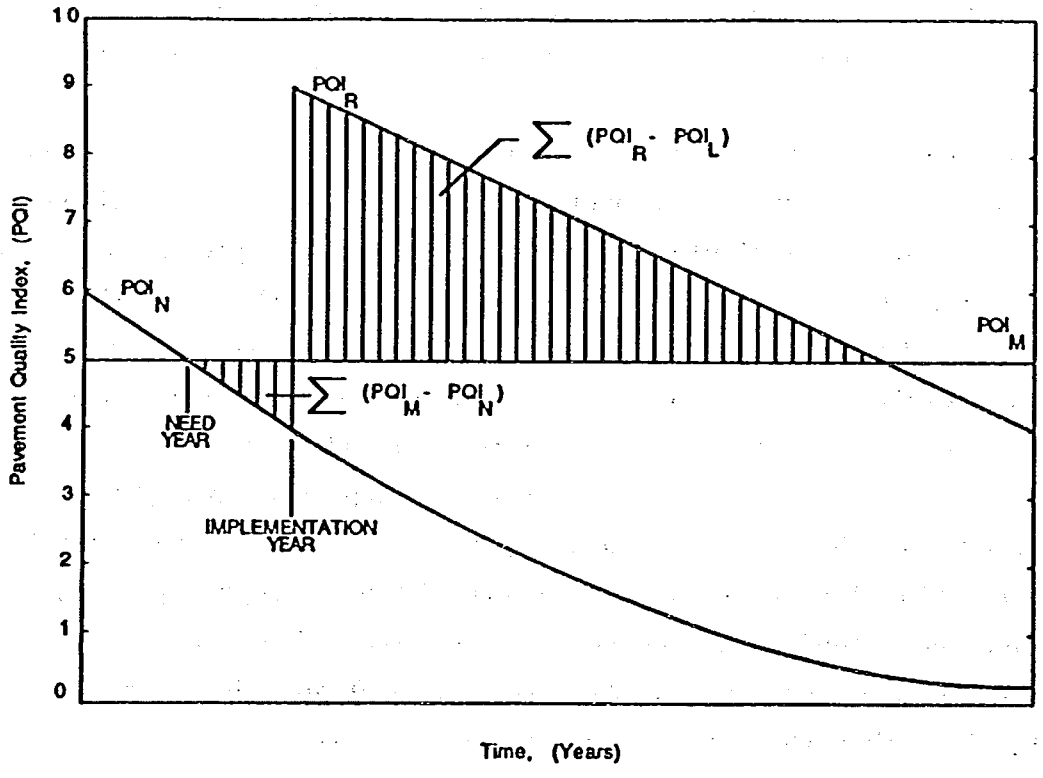


Figure 6 Calculation of effectiveness

TABLE 4

EXAMPLE OF A PRIORITY PROGRAM LIST BY YEAR

CITY OF — MPMS Rehabilitation Programming Subsystem

PRIORITY PROGRAM REPORT BY YEAR FOR 1989 — 09/01/1989

FUNCTIONAL CLASS : ARTERIAL — Page 1

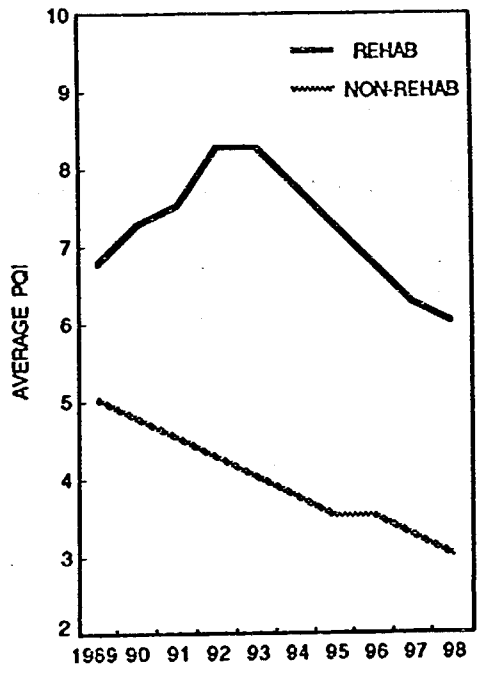
STREET	SECT. I.D.	LENGTH	PAVE TYPE	MAINT DIST	PQI NEED YEAR	RCI NEED YEAR	VCI NEED YEAR	SAI NEED YEAR	REHAB YEAR	REHABILITATION ALTERNATIVE	COST	EFFECT	COST-EFF	PQI NEED YEAR
Bellerose Dr. Boudreau Rd. (N) E.O.P.	10042	780	GB		1989	1898	1989	1992	1989	Overlay 60 mm	16380	447719	27.33	2009
Gervais Rd. Levasseur Rd. Hwy #2	10280	919	SC		1990	1991	1989	1994	1989	Overlay 60 mm	21780	204726	9.40	2000
Gervais Rd. Hwy #2 Levasseur Rd.	10290	919	FC		1989	1993	1989	1989	1989	Overlay 60 mm	21780	257446	11.82	2003
Hebert Rd. Arlin Akins Dr.	10400	870	FD		1989	1991	1989	1989	1989	Overlay 60 + Crb Rep	24133	190634	7.90	2003
Perron St. SW St. Anne St. Mission Ave.	10560	185	FD		1989	1989	2002	1989	1989	Overlay 50 mm	6937	56082	8.08	2003

CITY OF MPMS REHABILITATION PROGRAMMING SUBSYSTEM

PRIORITY PROGRAM PERFORMANCE SUMMARY REPORT 09/01/1989

FUNCTIONAL CLASS : ARTERIAL

YEAR	1989	1990	1991	1992	1993	1994	1995	1996	1997	1998
FUNDS PRORAMMED	162399	133148	67762	192745	87916	0	0	0	0	0
PROGRAMMED AVERAGE PCI	6.8	7.3	7.5	8.3	8.2	7.7	7.2	6.8	6.4	6.0
STD. DEV.	2.6	2.9	2.9	2.4	1.7	1.7	1.6	1.5	1.4	1.4
NON-REHAB AVERAGE PCI	5.0	4.7	4.4	4.2	4.0	3.7	3.6	3.4	3.2	3.0
STD. DEV.	1.5	1.5	1.4	1.4	1.3	1.2	1.2	1.1	1.0	1.0
PROGRAMMED % PCI < MIN	27	25	22	4	1	9	19	19	19	19
NON-REHAB % PCI < MIN	50	60	61	73	77	86	96	96	96	96



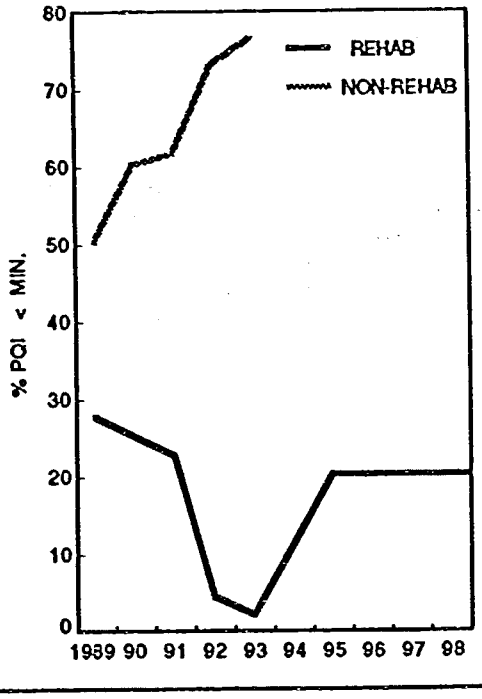


Figure 7 Summary report for a priority analysis, from Alberta's MPMS

4.4 Example Application of Maintenance Programming

A maintenance programming subsystem is included in Alberta's MPMS (2 in Fig. 1). It not only provides input to the rehabilitation programming subsystem, but can also be used on a stand-alone basis. This provides an additional degree of flexibility to the user municipalities. Figure 8 provides a flow chart and the overall structure of the subsystem.

It can be used to generate either maintenance needs reports, based on what maintenance treatments should be carried out to correct surface distresses, and/or to generate maintenance strategies under budget restraints. The latter utilizes a cost-effectiveness model based on the improvement in Visual Condition Index (VCI) resulting from the application of the treatment. Details are contained in Ref. (7).

Decision trees are also incorporated to identify, for cost-effectiveness analysis, those treatments which are feasible for any given type, severity and density of distress (9).

The products available from the maintenance programming subsystem are similar to those generated from the rehabilitation programming subsystem, and range from a listing of the streets programmed for rehabilitation (such as the example of Table 5) to performance and summary reports for the entire network maintenance program (such as the example of Figure 9).

The example of Fig. 9 illustrates that the distribution of Visual Condition Index (VCI) changes quite significantly from the situation of no maintenance budget to the unconstrained annual budget level of $83,836 (where all needs are satisfied). A summary report of maintenance activities and associated costs for a lower budget level (60N) of $59,881 is also shown in Fig. 9.

The approach used in the Alberta MPMS maintenance programming subsystem makes it very amenable to tailoring for individual municipalities. In effect, the decision trees, the cost-effectiveness analysis, the degree of improvement in VCI for various treatments and the provision to add or delete maintenance treatments mean that it is calibrated to local conditions. Moreover, the ability to evaluate different budget levels

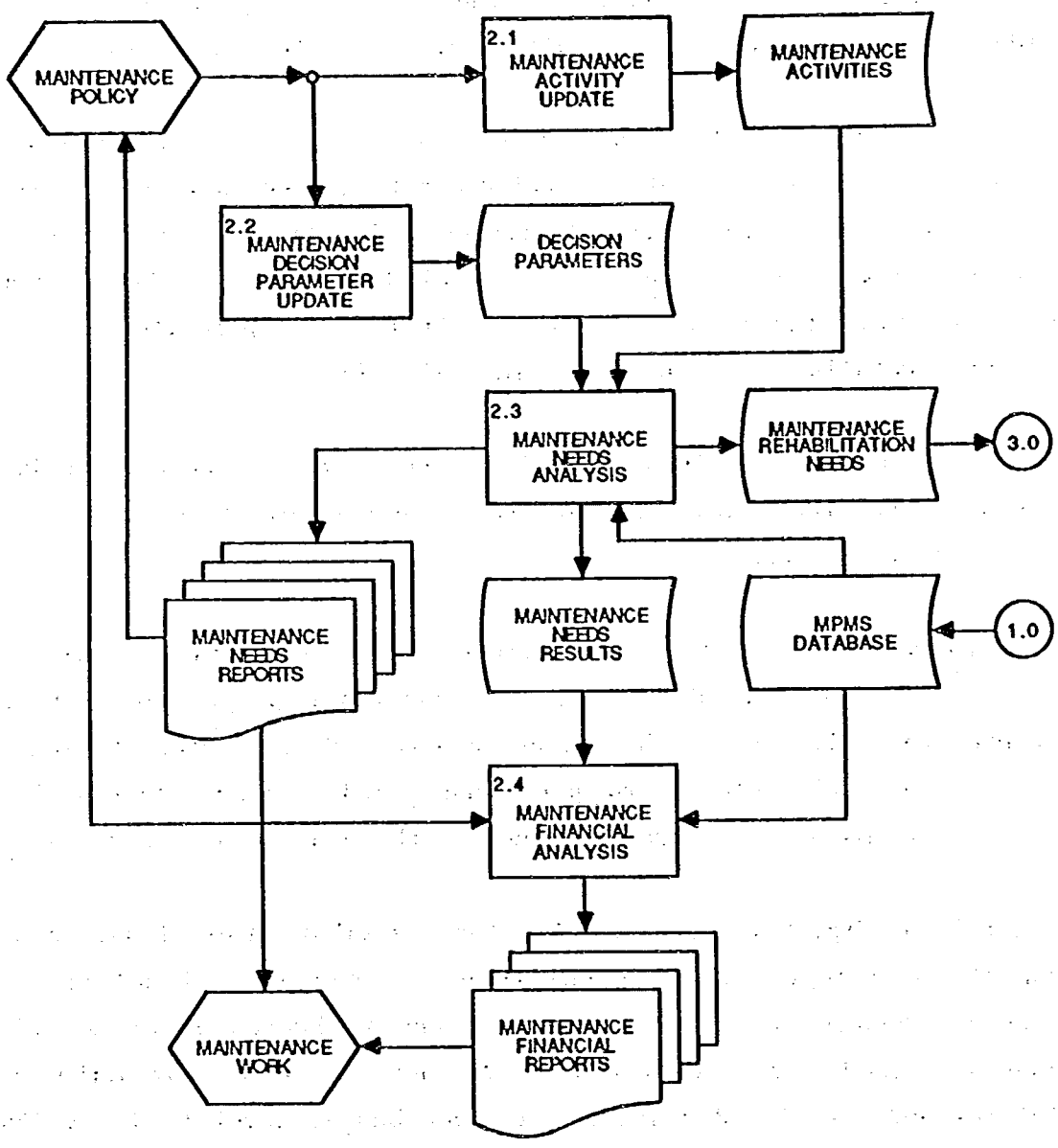


Figure 8 Overall structure of the maintenance programming subsystem in Alberta's MPMS

Table 5

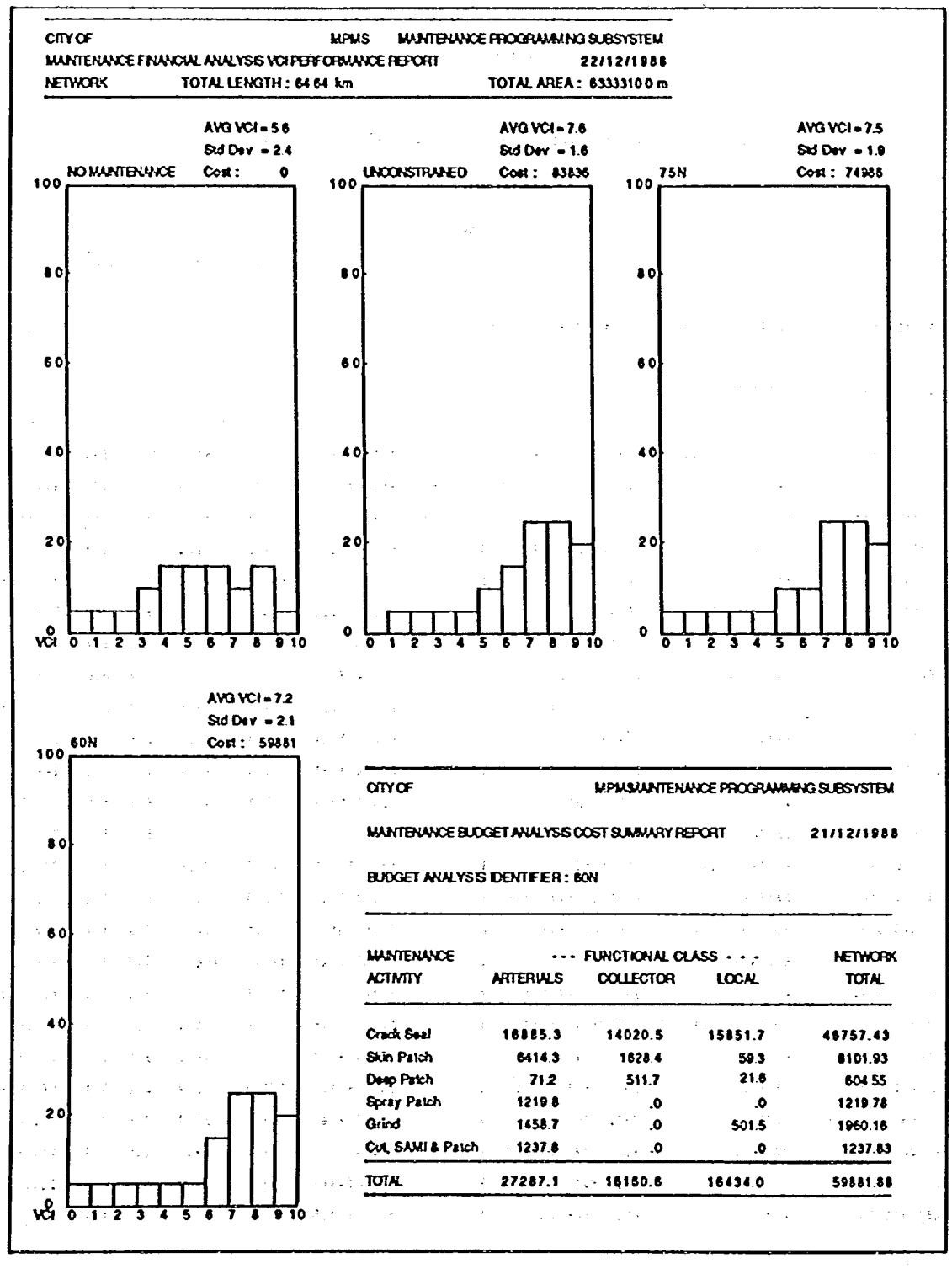
EXAMPLE OF A MAINTENANCE PROGRAM LIST

CITY OF — MPMS MAINTENANCE PROGRAMMING SUBSYSTEM

MAINTENANCE BUDGET ANALYSIS MAINTENANCE PROGRAM REPORT — 21/12/1988

BUDGET ANALYSIS IDENTIFIER: 60M

SECTION			FUNC	MAINT	MAINTENANCE		AREA	PREV	INC.	COST-
STREET	FROM	TO	CLAS	DIST	ACTIVITY	COST	(m2)	VCI	VCI	EFFECT
FINCH CR	FOREST DR (S)	FOREST DR (N)	3		Crack Seal	7.2	9.6	3.6	3.7	314.688
POIRIER AVE	PINEVIEW DR	CARNAGIE DR	1		Grind	160.6	44.6	6.6	7.5	303.053
POIRIER AVE	PINEVIEW DR	CARNAGIE DR	1		Crack Seal	66.6	88.6	7.5	7.8	292.879
SW CHURCHILL AVE	GRESHAM BL	PERRON ST	1		Grind	134.2	37.3	5.1	5.7	267.984
PERRON ST	SW CHURCHILL AVE	ST. ANNE ST	1		Crack Seal	62.4	83.3	6.2	6.6	265.900
PERRON ST	SW CHURCHILL AVE	ST. ANNE ST	1		Skin Patch	21.5	11.3	6.6	6.7	255.187
GATE AVE	GRANGE DR	HWY #2	2		Crack Seal	101.1	134.9	7.5	8.3	253.969
BOUDREAU RD	TEMP	CAMPBELL RD	1		Crack Seal	8.1	10.7	7.3	7.5	241.026
CHATELAIN DR	BOUDREAU RD	CALDER PL	2		Crack Seal	7.5	10.0	8.7	8.9	237.220
SW CHURCHILL AVE	GRASHAM BL	PERRON ST	1		Skin Patch	47.2	24.9	5.7	5.9	235.150
POIRIER AVE	PINEVIEW DR	CARNAGIE DR	1		Skin Patch	41.8	22.0	7.8	8.0	220.957
GRANDIN RD	GATE AVE	SW CHURCHILL AVE	1		Crack Seal	1772.5	2363.3	6.0	8.8	192.607
GATEWOOD AVE	SW CHURCHILL AVE	GRENFELL AVE	2		Skin Patch	30.8	16.2	1.5	1.7	192.055
SW CHURCHILL AVE	GRESHAM BL	PERRON ST	1		Crack Seal	573.2	764.3	5.9	7.7	189.581
STURGEON RD	BOUDREAU RD	SW CHURCHILL AVE	1		Grind	147.4	41.0	6.2	7.0	188.452
STURGEON RD	SW CHURCHILL AVE	(N) CITY LIMIT	3		Skin Patch	46.8	24.6	.1	1.3	182.475
CAMPBELL RD	BOUDREAU RD	(S) CITY LIMIT	2		Crack Seal	9.4	12.6	3.8	4.0	179.131
GRANDIN RD	GATE AVE	SW CHURCHILL AVE	1		Skin Patch	178.3	93.8	8.8	9.1	176.989
SW CHURCHILL AVE	GRESHAM BL	GATEWOOD AVE	1		Crack Seal	22.6	30.1	8.6	8.9	165.943
STURGEON RD	BOUDREAU RD	SW CHURCHILL AVE	1		Crack Seal	77.9	103.8	7.0	7.3	164.897
BEAVERBROOK CR	BOUDREAU RD	BEAVERBROOK CR	3		Deep Patch	21.6	4.8	5.4	5.6	160.711
GERVAIS RD	HWY #2	LEVASSEUR RD	1		Skin Patch	22.1	11.6	4.8	4.9	157.523



MAINTENANCE ACTIVITY	--- FUNCTIONAL CLASS - - - ARTERIALS	COLLECTOR	LOCAL	NETWORK TOTAL
Crack Seal	16885.3	14020.5	15851.7	46757.43
Skin Patch	6414.3	1628.4	59.3	8101.93
Deep Patch	71.2	511.7	21.6	604.55
Spray Patch	1219.8	.0	.0	1219.78
Grind	1458.7	.0	501.5	1960.16
Cut, SAMI & Patch	1237.8	.0	.0	1237.83
TOTAL	27287.1	16160.6	16434.0	59881.88

Figure 9 Example network distribution of Visual Condition Index (VCI) for various maintenance budget levels, and Summary Report for the selected budget level (60N)

makes it very flexible.

5. CONCLUSIONS

The major points of this paper can be summarized as follows:

1. Two decades of pavement management experience suggest that: (a) a generic framework exists, with flexibility for tailoring different methods and procedures to individual agency needs, (b) effective application requires a sound technology base for both the network and project levels, (c) staging is almost always necessary in implementation, (d) below some threshold level of funding, the most effective pavement management system can not prevent deterioration of a network, and (e) applicability does not depend on size of the authority.

2. Five major modules have been identified for staging a network level pavement management system; these are:

   a) Data base
   b) Needs analysis
   c) Rehabilitation priority programming
   d) Maintenance planning and programming
   e) Combined maintenance and rehabilitation programming.

   Each module should have useable end products.

3. The ability to develop and implement by stages inherently represents tailoring to specific needs and resources of individual authorities. Tailoring is also represented by the flexibility of timing for these stages and by the products specified for each stage.

4. Examples are given in the paper from the application of Alberta's Municipal Pavement Management System (MPMS), which is both very comprehensive and highly flexible so that it can be used by a wide range of very small to large authorities.

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