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REHABILITATION OF FULL-DEPTH ASPHALT CONCRETE PAVEMENTS IN ALBERTA

by

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MOTTO

"minimum of expense is, of course, highly desirable; but the road which is truly the cheapest is not the one which has cost the least money, but the one which makes the most profitable returns in proportion to the amount expended upon it." (Gillespie)

To my wife Maria and daughters Hanna and Ewa.

ABSTRACT

Full-depth asphalt concrete pavements developed by the Asphalt Institute once gained widespread acceptance in the Province of Alberta, Canada. However, premature failures of some of these pavements caused the Province to cease their construction in 1982. This investigation is aimed at developing guidelines for rehabilitation measures appropriate for full-depth pavements existing in the Province. In the course of the work, experiences were gathered regarding full-depth pavements in seasonal frost areas of North America. The study revealed that full-depth pavement performance is generally satisfactory, however with some exceptions. Poor performance has been experienced with thin pavements, non-uniform and/or weak subgrades and construction with little or no quality control.

In the second phase of this research mechanistic-empirical methods of full-depth pavement rehabilitation design was investigated. Two backcalculation techniques of pavement moduli determination were employed, using FWD deflection basin information. These two method were: ELMOD and FWDUT1S of the MAPCON computer system. It was found that the FWDUT1S asphalt concrete moduli were approximately nine percent lower, and the FWDUT1S subgrade moduli were 17 percent higher than the appropriate ELMOD moduli.

Five full-depth selected sections were analyzed using three semi-analytical design procedures. The methods

used were: ELMOD and two DAMA based procedures. In the DAMA methods two fatigue criteria were employed. Calculated overlay thicknesses using the three procedures and the RTAC Benkelman beam deflection based were compared for the selected pavements.

The DAMA based overlay method was used to developed three rehabilitation measures for a thick full-d[illegible] pavement. Three selected rehabilitation alternatives[illegible] overlay, reconstruction and partial cold-milling followed by overlay were considered. The third method appears to be a promising one. However, further work is required to evaluate costs, construction techniques and expected performance to obtain realistic life-cycle costs.

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INTRODUCTION

1.1 General

A pavement structure is a system, usually composed of layers of various materials, which should provide a means to distribute traffic loads to the underlying subgrade. To properly serve the public any pavement should be safe, provide reasonable comfort of ride, be durable and have minimal maintenance costs.

Because of the well developed existing highway network construction of new highway pavements will be limited due to various reasons. Some reasons are:

- high cost of the right-of-way acquisition
- increasing cost of highway materials and
- necessity for the energy conservation

In view of this the existing highway network will have to serve adequately for the expected future traffic loadings. This means that existing pavements will have to be periodically rehabilitated.

Out of many rehabilitation measures, overlays are most commonly utilized because of their simplicity of construction. In the case of the asphalt concrete overlays, there is the possibility to open the highway to the traffic immediately after construction. However, because of scarceness of quality overlay materials and necessity for the energy conservation, rehabilitation measures now include the utilization of already built-in layers by their recycling.

Since analytical-empirical design of new pavements has now gained worldwide acceptance, current research efforts should be directed toward using this concept for design of overlays and other rehabilitation procedures. As indicated by the name the analytical-empirical method of design (or mechanistic-empirical) contains two parts. The first part is an analytical model employed for calculating the pavement critical stresses and strains in each pavement layer. The second part consists an empirical relationship between the pavement responses, calculated using the first part, and the rate of pavement deterioration. In the case of overlay design a new factor absent in the case of design of new pavements appears. This is the necessity for an adequate evaluation of the existing structure. The structural evaluation has been performed utilizing non-destructive methods of testing. Up to the present time the Benkelman beam procedure has been the one most commonly applied. The last few years have brought into use devices able to determine not only one central deflection near the loaded area, as with the beam procedure, but many deflections with a possibility for the shape of deflection basin determination. Knowing the basin's shape, pavement layer thicknesses and applying the theory of elasticity with backcalculation techniques the elastic moduli of each layer may be determined. The moduli may then be used to estimate remaining life of the evaluated pavement. In this way the structural capability of the existing pavement may be more rationally determined. In this study considerable effort has been directed toward the analytical-empirical design of asphalt concrete overlays of full-depth asphalt concrete pavements.

1.2 Purpose and Objectives of the Investigation

The general purpose of this study is to establish guidelines for the rehabilitation of full-depth asphalt concrete pavements in the Province of Alberta. The more detailed objectives are:

1) To review experiences with full-depth asphalt concrete pavements in seasonal frost areas including Alberta.

2) To introduce the pavement moduli backcalculation procedures and to review some concepts of mechanistic-empirical methods of pavement rehabilitation.

3) To apply the introduced backcalculation approach and mechanistic-empirical design method to selected full-depth pavements.

4) To provide strategies for full-depth pavements rehabilitation measures.

1.3 Scope of the Thesis

The scope of this research is limited to full-depth asphalt concrete pavements situated in seasonal frost areas of North America and particularly in the Province of Alberta, Canada.

Two pavement moduli backcalculation techniques have been selected out of a variety of available procedures for detailed consideration. These two procedures are, namely, the Dynatest ELMOD (1, 2, 3) and the MAPCON based FWDUT1S (4, 5, 6) computer models. Four mechanistic-empirical overlay design procedures are described in this work. They are:

1) Dynatest ELMOD model (1, 2, 3)
2) MAPCON based model (4, 5, 6)
3) The University of Nottingham model (7) and
4) DAMA computer model of The Asphalt Institute (8)

For more detailed investigation five full-depth asphalt concrete pavements located in the central region of Alberta were selected. The structural properties of the pavements were evaluated using the Dynatest Model 8000 Falling Weight Deflectometer and ELMOD backcalculation techniques. The estimated pavement moduli were then incorporated into a DAMA based overlay design procedure.

1.4 Organization of the Thesis

This study is divided into eight chapters and five appendices.

Chapter 1 - Introduction, contains a general introduction to the topic as well as scope, objectives and the work organization.

Chapter 2 reviews rehabilitation concepts and pavement evaluation techniques. A part regarding economic considerations for pavement rehabilitation is also included.

Chapter 3 summarizes experiences with full-depth asphalt concrete pavements in seasonal frost areas other than the Province of Alberta, whereas Chapter 4 reviews the performance of this type of pavement structure in Alberta.

In Chapter 5, the concept of establishing the pavement layer moduli based on the Falling Weight Deflectometer measured deflection basins and backcalculation techniques is introduced. Further some mechanistic-empirical models of overlay design are described.

Chapter 6 describes a comparison of two selected backcalculation techniques. Also a description of analyses performed on some selected full-depth pavement sections using a selected design approach is included. The selected approach is based on the DAMA computer program with ELMOD calculated pavement moduli incorporated. Two sets of DAMA computer analysis are presented, each utilizing different asphalt concrete fatigue criteria. The results of these two analyses are compared with each other, the Dynatest ELMOD mechanistic procedure accomplished by JEGEL (3) and with the RTAC Benkelman beam deflection based procedure (9).

Chapter 7 presents proposed guidelines for rehabilitation strategies applicable to full-depth asphalt concrete pavements. This Chapter also contains initial construction cost comparisons of three rehabilitation methods.

Chapter 8 is the terminal section of the study providing a summary of the Thesis, conclusions and major recommendations for further research.

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CHAPTER 2

REHABILITATION CONCEPTS

2.1 General

The purpose of any highway is to provide the public with safe, comfortable, convenient and economical methods for transporting people and goods. However highways, as well as all other engineering structures, lose their strength or bearing capacity so they are less able to provide quality services to the public. With time the highway surfaces become less skid resistant and rougher and the climatic and loading type of distress show up. Because of all these reasons, some means of restoring the as-designed conditions of the highways is necessary.

2.2 Current practices and trends in rehabilitation

2.2.1 Introduction

Very many different factors might make a pavement rehabilitation necessary (1,2,3,4). Some of these are:

a) safety

b) quality of ride

c) bearing quality of the structure

d) surface distress or condition

a. Because each pavement should provide a safe ride to its users, the deterioration of safety standards is one reason why rehabilitation has to be considered. This type of rehabilitation usually restores pavement skid resistance.

b. Quality of ride can be determined subjectively as well as by means of objectively measuring the pavement roughness. For any given road, a certain level of quality can be specified. Quality of ride depends on the expectancy of the potential user. Different quality will be expected on a freeway than on a low-volume rural road. From this point of view, it seems to be uneconomical to restore pavement riding quality beyond that expected by the user.

c. Bearing capacity, which expresses a pavement's ability to carry present and future traffic, is a factor which often calls for some form of rehabilitation. Structural capability for present loads may be fulfilled but, for the expected loads it might not be adequate. From this perspective it may be better to upgrade the pavement capability sooner than to allow for severe deterioration under the heavier traffic expected.

d. Surface conditions like cracks, distortions, patches and potholes can be good reasons for pavement rehabilitation. Although these signs of distress might be non-loading associated and at the time of evaluation do not influence bearing capacity of the pavement, their presence sooner or later will have some negative influence on the pavement structural adequacy. It may be reasonable to eliminate this distress before the structural adequacy is severely affected.

All of the above factors should be looked at in the light of the cost associated with them, cost effectiveness and life-cycle times. It is usually more effective to keep a pavement in good condition by employing proper and timely maintenance procedures than to allow the pavement to deteriorate to an extent where more extensive and expensive means of restoration must be applied.

Restoration of pavement conditions can be achieved by means of several different methods which are dependant on:

a) type of pavement

b) type and extent of distress

c) loads imposed by traffic

d) availability of pavement materials

e) policy of a highway agency

f) available funds

g) available contractors and their expertise

h) expectancy of the facility users

Of these factors, the available funds is the one which most often limits the agency. Therefore, any restoration technique should be compatible not only with a particular pavement but also with the road network as a whole.

Generally, three groups of methods of restoration to original or close to original quality are used (1).

These are:

1. methods using overlays
2. methods other than overlays
3. reconstruction

2.2.2 Overlays

Overlays are a widely accepted method by which pavement conditions can be restored (1,4,5). An overlay can be applied in order to:

- strengthen the existing pavement structure
- eliminate surface distress
- improve skid resistance
- improve riding quality

Depending on the reason for an overlay application, its design will be different e.g. an overlay required to restore the structural adequacy of a pavement will be designed differently than an overlay aimed at improving the pavement riding quality.

Commonly, uniform overlay thickness is applied throughout a considered project. However, a much better approach is to divide the project into homogeneous sections having similar structural, distress, riding quality and skid resistance characteristics, and differentiate the thicknesses accordingly (1,4).

2.2.3 Methods Other than Overlays

Methods of rehabilitation other than overlays can include (1):

a) full-depth repairs

b) patching

c) crack sealing

d) surface treatment

e) cold milling

f) drainage rehabilitation

<u>Full-depth repairs</u> of asphalt concrete pavements can be, to some extent, considered as miniature reconstructions of short pavement sections (1,4). This type of repair is usually costly and some agencies tend not to perform it, waiting instead until an overlay of the entire facility is required. However, such an approach can cause even greater expenditure in the future. The full-depth repair is not liked either by contractors and by highway agencies. For a contractor, the work can be much more tedious, requiring special equipment, more expensive manpower and greater effort to organize the work properly. Consequently, it is rather obvious that this work is more suited for smaller, specialized contractors than for larger ones. For a highway agency, the full-depth repair means greater effort in checking the quality of the job, subsequent monitoring of the performance and, so on. However, it is probable that this kind of rehabilitation measure should be performed

more often because of lack of appropriate funds to perform overlays on long sections of roads and difficulties in obtaining large quantities of quality highway materials.

Patching can be done using hot or cold asphalt mixtures (1,4). It can also be considered both as a temporary and a permanent means of rehabilitation.

Patching as a temporary measure is usually performed during inadequate weather conditions. According to Eaton (6) however, all patches should be done permanently at first. The mentioned paper cites a study accomplished by the Pennsylvania Department of Transportation which showed that repeated pothole patching cost five times as much annually as a one-time permanent patch. So that areas needing patches should be repaired at once and using the very well known state-of-the-art techniques.

Crack sealing should also be considered as a rehabilitation measure (1,4). Cracks act as a means of water ingress into a pavement structure and a break in the continuity of the structure have a detrimental influence on the pavement structural capacity and riding quality.

Cracks, if not properly treated, can be especially harmful for a structure in frost areas when they are associated with frost susceptible soils. From this perspective, sealing of all types of cracks can have only a beneficial influence on a pavement's overall performance.

Surface treatment is a kind of rehabilitation that can not be considered as a pavement structural restoration

(1,4). However, surface treatment can extend the life of a facility and indirectly benefit it's structural adequacy.

The major functions of surface treatment are:

- providing a wearing course
- sealing cracks
- waterproofing effect
- skid resistance improvement
- reduction of oxidizing of asphalt in asphalt concrete
- improvement of the surface appearance
- improvement of a pavement's visual delineation

Surface treatments should not be applied when the bearing capacity of a pavement is not adequate to carry traffic over the next few years; otherwise, the use of this treatment will be non-cost effective.

Cold-milling of asphalt concrete surfaces is a technique usually applied to restore pavement riding quality (1,4). Roughness is the reason for a decrease in the riding quality of a pavement. The roughness might be caused by many factors, for example: cracks, deformations of the pavement surface and ravelling. Permanent deformations or rutting can be caused by :

- inadequate stability of the asphalt mixture (caused by improper design or improper construction - compaction)
- inadequate bearing capacities of the base, subbase, or subgrade (note as above)

Other than rutting, forms of permanent deformation are:

- swelling which is caused by volumetric expansion of some soils due to moisture content increase. The increase in water content can be induced by the ingress of water through cracks or the movement of water upward in the subgrade due to capillary or frost action.

- frost heaving is actually a periodic phenomenon which is due to the formation of ice lenses in a frost susceptible soil. The ice lenses cause an upward movement of the pavement. After frost ceases the pavement surface returns to its previous position.

Ravelling is another means of negative contribution to the pavement ride quality. It is generated by a progressive separation of aggregate particles from the pavement surface.

Cold-milling coupled with a subsequent overlay has also gained recognition as a means for the restoration of pavement bearing capabilities. Overlay following the cold-milling procedure may be composed either of new materials or, what is highly desirable from an economical point of view, of the recycled original material. Such an approach utilizes the existing material, provides for energy conservation and also provides the strengthened pavement with the same or almost the same grade elevation.

<u>Drainage rehabilitation</u> (1). Inadequate drainage, although considered as a very detrimental factor to the overall highway performance, very often is not taken into account when a rehabilitation procedure is designed. Evaluation of both surface and subsurface drainage should be one

of the very first steps taken when an overall evaluation of a pavement is performed. Inadequate drainage will usually be responsible for a premature pavement failure; whereas properly designed and constructed drainage system can extend substantially the service life of the entire facility.

2.2.4 Reconstruction

Reconstruction means the restoration of pavement quality by the removal of the entire pavement structure and the placement of a new pavement which can be of the same type as before the reconstruction or utilize a different design concept (1). Reconstruction generally is performed when the pavement is very heavily distressed or its service life is considered to be fully utilized. In a certain sense, reconstruction of a pavement should not be regarded as a rehabilitation measure but more as a design of an entirely new facility.

2.3 Pavement Evaluation

The evaluation of a pavement involves the description or measurement of that pavement's condition and performance with the use of certain evaluation techniques. The objective of pavement evaluation is to check whether the functions the pavement was designed to serve are met and its performance achieved. An evaluation should provide the pavement manager with adequate information to plan a suitable rehabilitation measure to bring the pavement back to its required level of serviceability. The evaluation of pavements takes place on two different levels (3):

a) the network level is where assessments of all pavements confined to a certain network of highways is considered and the priority of rehabilitation treatments are dealt with. Highway sections are periodically evaluated in terms of riding comfort, structural capacity, surface distress, and skid resistance, and, having funding constraints in mind, are ranked in order to prioritize the needs for rehabilitation.

b) the project level is where one particular pavement is taken into consideration and the most cost-effective rehabilitation measure is found. This kind of evaluation is usually much more extensive.

These two levels include some similar aspects, although they serve two quite different purposes. Both procedures involve the assessment of:

- structural adequacy, capacity or bearing capacity of a pavement
- extent and type of distress or deterioration
- quality of ride to the users which the pavement provides
- safety of travel

Evaluation on a project level involves the following assessments beyond these stated above (5,7):

- variation of conditions along the project
- climatic effects
- assessment of pavement materials utilized
- estimation of subgrade condition
- quantity and types of loads imposed by the traffic
- shoulder condition
- previously performed maintenance
- geometric factors
- length and width of a section
- construction data
- estimated residual life
- cost of side effects such as: redoing road marking and signs and raising curbs, gutter and guardrails taken as a function of the predicted overlay thickness
- estimated costs of initial construction, maintenance, and rehabilitation
- users' costs
- salvage value

2.3.1 Evaluating Pavement Structural Capacity

The evaluation of load carrying capacity or bearing capacity of a pavement can be conducted in the laboratory or in the field. The measuring methods can be either of a destructive type, as all laboratory tests, or non-destructive as some of the in-situ measurements (1-5,7-9).

The laboratory tests can be performed on samples retrieved from a pavement. The tests most commonly practiced are:

- triaxial test
- flexural stiffness
- indirect tension
- resilient modulus

The last test in particular, has gained wide acceptance as a procedure which simulates reasonably well in the laboratory the loading and environmental conditions found in the field. Saw-cut samples of asphalt concrete can be tested in different temperatures and the number of load repetitions to failure can be determined. Some problems have been reported when samples of unbound material or subgrade soil have been tested (9).

The in-situ tests can be either of a destructive type:

- California Bearing Ratio test CBR
- plate load
- cone penetrometer test

or non-destructive types such as:

- Benkelman beam test
- Dynaflect
- Lacroix deflectograph
- road raters
- vibratory equipment
- falling weight deflectometer (FWD)

There are many serious problems connected with the use of laboratory tests and the correlation of their results with the in-situ pavement response (9). An adequate simulation of in-situ conditions such as traffic loads, environmental conditions, induction of developing stresses and strains in the laboratory can present other problems. On the other hand, destructive field methods are very slow and when performed during normal traffic operation, cause traffic delays. These types of tests also require a considerable number of personnel, either to perform the test or to provide an adequate safety measure. Lastly, the tests flaw the pavement surface. Because of the above problems, the in-situ, non-destructive methods are gaining a lot of attention and there is a tendency to utilize them more frequently.

The most commonly used of these tests is the Benkelman beam test developed in the mid-fifties (1-4,8). The reason this test has been utilized so widely and for so long is its low cost and the ease and relative speed of measurement. However, the method has certain disadvantages in that it does not simulate the real traffic loading adequately. The pavement response under a fast moving vehicle and the deflection measurement using the Benkelman beam may not correlate well. The Benkelman beam deflection is obtained only in central point near the load, whereas knowing the shape of a deflection bowl would be more beneficial. The pavement bearing capability is correlated with so obtained maximum pavement deflection. However, deterioration of a pavement depends on critical stresses or strains induced in the pavement layers and not on the deflection itself which is incompatible with the stress and strain criteria (7). The fact that the Benkelman beam test might be performed at a creep speed could be a detrimental factor when pavements with a thick asphalt concrete layers are evaluated. In these pavements the asphalt concrete layer can be regarded as a visco-elastic material. This is mean that under a steady load, the strain will increase either linearly or in a non-linear manner and, after the load is released the strain will decrease either linearly or non-linearly, not necessarily following the loading path (7). Having this in mind, one can say that visco-elastic material behaviour will influence the magnitude of the Benkelman beam deflections. The calculated stiffnesses of the pavement materials will be of a "static" type instead of "dynamic". To avoid the above explained discrepancies related to the Benkelman beam method another procedure, which much closer relates the pavement response under testing a real loading should be found. At present, there is a worldwide tendency to design pavements and evaluate their behaviour by using an analytical-empirical or sometimes named mechanistic-empirical approach. This approach generally involves the design of pavement structures by choosing an appropriate combination of materials of known properties (elastic moduli, Poisson's ratio) and layer thicknesses in order to mitigate the various forms of distress which might be caused by traffic or environmental loadings or the two combined. In order to obtain the appropriate data, a piece of equipment is required which would be able to measure shape of the deflection bowl. The layer properties could then be backcalculated using the elastic theory. Taking all the above into consideration it is clear that a different test method is required.

The Falling Weight Deflectometer is, at present, probably one of the most suitable pieces of equipment to fulfill the above requirements (1,9,10,11,12). The load under which the pavement deflects is applied by means of a weight dropped from a specified height. Both the weight and the height can be adjusted in order to produce different impacts imposed on the pavement. The impacts range from 7 to 120 kN and the load duration of sinusoidal shape lasts 25 to 30 milliseconds. The equipment is designed to simultaneously measure up to seven deflections with seven geophones located on a horizontal bar.

The advantages of the FWD equipment are that it simulates very well the fast moving heavy vehicle load, the measurement is performed very quickly, and the shape of the deflection bowl is determined (7). From the output and by utilizing the theory of elasticity, determination of elastic moduli of the pavement layers is possible (1,7,12). Having these data and using the mechanistic-empirical method, it is possible then to calculate the critical stresses and strains induced in each layer of the pavement by different combinations of loads caused either by the traffic or environmental factors. At this time Miner's law (13) can be applied, assuming that a particular material can absorb only a certain amount of load repetitions at a certain level of strain or stress before it fails. For each material, the relationship between the number of repetitions to failure and the level of stress or strain can be established. Summation of the damage is done according to the equation;

$$\sum n/N = 1 \qquad (2.1)$$

where: n - actual number of repetitions at a stress level

N - allowed number of repetitions to produce a failure at the stress level

A question now arises as to how loads caused by different vehicles should be treated. One method of treatment is to convert all vehicle loadings to one equivalent loading. This procedure was first developed during the AASHO Road Test and it is has been widely used. Using this concept any, axle load can be converted to the Equivalent Single Axle Load (ESAL) by using a load equivalency factor determined, for example by applying the "fourth power law" (14).

$$LEF=(\text{single axle load/standard single axle load})^4 \quad (2.2)$$

where LEF stands for Load Equivalency Factor

Knowing the relationship between ESAL and stresses produced by it in a pavement, one applying Miner's law can predict the number of ESAL's required to produce failure to the particular pavement structure. On the other hand, knowing the predicted number of load repetitions to failure or, in the other words the pavement life and the number of repetitions the pavement has carried, an estimation of the pavement's remaining life can be determined. Based on the above, some rehabilitation measures can be applied at the end of the pavement predicted life in order to restore the pavement's load carrying capacity.

2.3.2 Evaluating Pavement Riding Comfort

Each pavement is produced to provide a smooth, comfortable and safe ride to its potential users. In this sense, roughness of the pavement surface is a factor which is considered to be the one most responsible for the quality of the ride. The rougher the pavement, the worse comfort to the rider it provides. Based on this concept, a rating procedure for categorizing pavements has been developed. The rating of a particular pavement is performed by a panel of individuals who ride along the rated highway section and express their opinions about the comfort of the ride based on an especially developed scale (2,3). Another method is to use a mechanical device capable of measuring all unevennesses of the pavement.

Roughness of the pavement which can be defined as deviations of a pavement surface from a true planar surface can be caused by (3):

- transverse variation of a pavement surface
- longitudinal variations
- horizontal variations

Many studies have shown that the longitudinal variations have the biggest influence on the rating. As a result, the equipment commonnly used has been developed to measure only these types of pavement surface variations (3).

Roughness measured mechanically can be evaluated by two methods: by measuring the response of the equipment [illegible] all pavement unevenness or by measuring the pavement real profile.

The first procedure can be evaluated by:

- PCA-type roadmeter
- BPR-type roughometer
- May's ridemeter

The May's ridemeter is by far the most common equipment used (15). It measures rear axle to body excursions through a photocell sensing system with a 2.5 mm resolution (8). This system is equipped with an automatic pen and a moving paper tape. The pen moves at a rate proportional to the movements of the vehicle body and its differential.

The pavement real profile method can be performed by:

- Surface Dynamic Profilometer
- straight edges
- the CHLOE profilometer

The first of these is one of the most accurate methods of pavement longitudinal profile measurement. Accelerations of a vehicle frame's vertical motion are recorded and by double integration the frame displacements are determined. Additionally, the movement of a wheel following the pavement is recorded (2,3,15). Both these movements are added, and by this means, the actual profile of a pavement is determined.

2.3.3 Evaluating Surface Distress

All pavements deteriorate over time due to weathering, aging, structural distress and so on. Although the influence of pavement distress on the structural condition of the pavement is not well defined, there is a justified tendency to correlate these two and supplement the structural evaluation by the distress data. Condition surveys are the only ways of determining the maintenance procedures required to prevent an acceleration of the pavement surface distress which, in turn, may have a disastrous influence on the pavement bearing capacity. Condition surveys are expensive and labor intensive because their performance involves a fair amount of detail.

Evaluation of pavement distress generally involves four main groups of damage (1-4):

- surface defects
- permanent deformation
- cracking
- patching.

Each of the above can be broken down into more specific forms of distress.

A properly performed condition survey will include a description of the distress severity, and its density, or areal extent. Taking the above into consideration, and assuming that the survey should be performed continuously, along each pavement of a highway network, one can realize how time consuming, tedious and expensive it is to adequately perform the task. One solution is to evaluate only randomly chosen samples of highway links (7). This procedure however, always involves a high degree of risk. Another solution yet, is to have the task performed visually by a trained operator travelling along a road. The operator can input the distress onto a computer storing device, coding the distress type and extent in a systematic way. This technique can also be supported by utilizing a video-logging system.

All of the described techniques are similar in the sense that they all rely on a trained operator's eye and are of a subjective nature.

An example of how the surface distress can be evaluated is given by Road and Transportation Association of Canada (16). Each of 15 surface defects is described; possible causes are given, and possible remedial measures suggested. To prepare a pavement section condition rating, each of these defects has to be defined in terms of two factors i.e. density and severity. Density, based on the percentage of surface area of the pavement section being affected by the defect, is described as follows:

1. few	<10 %
2. intermittent	10 - 20 %
3. frequent	20 - 50 %
4. extensive	50 - 80 %
5. throughout	80 -100 %

Severity is described by means of sets of photographs. In the case of pavement permanent deformation, severity is assigned by taking the rut depth into account, and in the case of cracks, their widths are considered. The limits are established as follows:

	rut depth (cm)	crack width (cm)
1. very slight	< 0.5	< 0.7
2. slight	0.5 - 1	0.7 - 1
3. moderate	1 - 2	1 -2
4. severe	2 - 5	2 - 3
5. very severe	> 5	> 3

The individual values of each defect density and severity may be subsequently combined in one overall index by assigning various appropriate weighting values for each distress mode. Two such procedures, implemented for the Province of Prince Edward Island and the Province of New Brunswick, are given in the described work.

2.3.4 Evaluating Pavement Safety

Safety is one of the most important factors when evaluation of a pavement is considered. The public is always very concerned about pavement safety so some forms of rehabilitation have to be considered when the pavement safety is not adequate, even if its bearing capacity and distress are of no concern at all. When one is thinking about pavement safety he or she thinks about a pavement that (1,2,4):

- provides adequate skid resistance
- has negligible rutting as related to aquaplanning
- has such a color that the surface is visible during night time
- does not reflect the light and make the task of driving difficult or dangerous
- is clearly marked

All these requirements besides the skid resistance, although very important, are still very difficult to measure and quantify (2). More research is needed in order to thoroughly investigate their influence on a pavement safety.

Skid resistance, on the other hand, is usually taken as a main factor of a pavement safety because it is comparatively easy to measure. Friction generally, and pavement friction in particular, is a force that prevents an object from moving along another object (15). In the case of pavement, it is a force that prevents a locked vehicle wheel from sliding along the pavement. However, in order to determine a pavement friction, certain factors on which the friction depends must first be defined. The most important of these are: tire type, design, inflation, and wear, and vehicle speed.

At present, the two most common methods used are (15):

- locked wheel trailer method
- yaw mode method

In the first method, a wheel mounted in a car-towed trailer travelling at a specified speed is braked to lock while a film of water is applied to the pavement under the wheel. The friction force generated is a measure of the pavement skid resistance. The second method utilizes tires which are mounted on a towed trailer in a yaw mode to the direction of travel. The side forces developed are measured so the side friction factor can be determined.

Usually, the measurements of pavement friction are performed on a biannual basis on each highway section of a network. The measurement frequency has ranged from 0.6 to 6.3 tests per kilometer (1 to 10 measurements per mile) with the average spacing (in the USA) of 4 points per kilometer (2.5 points per mile) (15).

Generally, gathered information are supplemented with wet-accidents localizations and used to decide if any corrective measure for a pavement surface is necessary.

2.3.5 New Technologies in the Evaluation of Pavements

As stated previously, a comprehensive pavement condition survey is necessary to properly evaluate a pavement's visual quality. It was also shown that a condition survey is an expensive procedure due to the labor intensity required. Consequently, there is a tendency to develop new technologies thus allowing the task to be performed less expensively and freeing the task from its subjective character. Two innovations chosen to be described are the Automatic Road Analyzer (ARAN) (15,17,18) and a Swedish laser road surface tester (18).

2.3.5.1 Automatic Road Analyzer

This equipment, developed by a company in Ontario, Canada, was first described in 1979. Since then it has undergone considerable modification. Presently, the analyzer is designed to measure up to thirty pavement performance parameters. The equipment is mounted in a normal sized van and all the measurements can be performed at a normal highway speed. The pavement roughness is measured by recording the axle vertical accelerations which are then translated by the on-board computer to either the Riding Comfort Index or the Present Serviceability Index. The unit is equipped with thirteen ultrasonic displacement transducers, mounted on a horizontal bar attached to the front bumper of the van, and on-board gyroscopes. By using gyroscopes and ultrasonic transducers some other pavement parameters can also be determined such as :

- crossfall angle
- radius of pavement horizontal curves
- grade.

The pavement condition rating is recorded by the operator making visual observations using two microprocessor keyboards on which coding of particular distress forms and their extent can be accomplished. As an option, the system can be equipped with a video-camera which produces a continuous picture of a pavement. This picture can be used later as a supplement for the Pavement Condition Rating keyboard system.

2.3.5.2 The Swedish Laser Road Surface Tester

The purpose of this device is to measure the pavement profile and cracking using laser technology. Eleven lasers, mounted to the front bumper of a vehicle, continuously measure pavement rut depths. The on-board computer averages the input every 5 meters and the input is further averaged for the entire highway link measured. Pavement transverse cracking is measured with the aid of four of the eleven lasers. The lasers are positioned so as to be able to measure cracking in the center lane, inner and outer wheelpaths, and at the pavement edge. As output, the number of cracks per 100 meters of travel is shown.

The equipment can also measure the macrotexture of the pavement, defined as surface texture, with wavelengths respectively between 0 and 10 mm and 10 and 80 mm. The macrotexture is measured in a continuous mode in each wheelpath.

2.4 Economic Considerations for Pavement Rehabilitation

All pavements have a finite, and usually widely varying, service life. After a pavement reaches a certain age or extent of distress, some rehabilitation measure must be applied in order to rejuvenate the surface and extend the pavement service life. At this point, engineering economics has to be considered in order to chose from among the available rehabilitation alternatives. The economic process must be accomplished in some organized way. One way is to use a systems approach (19). The system approach format can be formulated as follows:

1) identify the problem
   - a) define the basic needs
   - b) define related needs
   - c) define the scope of the problem

d) define time frames

f) obtain relevant data

2) determine the objectives of a solution

3) articulate the measure of effectiveness

4) generate alternatives

5) evaluate the alternatives

6) perform a sensitivity analysis

7) select the best alternative

1.a. In the case of a rehabilitation measure, the basic need can be formulated in many different ways depending on the present status of the pavement, economic constraints, user expectancies, and so on. As an example, the basic need can be to improve one of the following:

- bearing capacity
- skid resistance
- pavement appearance (surface distress)
- reduce user's costs
- riding quality
- or all of these factors combined in one performance index as for example the Pavement Quality Index (PQI).

b. The related needs of a pavement rehabilitation measure can be safety and economy of the considered project.

c. The scope can be limited to only one project, a certain type of project, the entire highway network of a jurisdiction.

d. The analysis period must be determined. The period of time usually considered is taken as 15 to 40 years (19). The period must be the same for all considered alternatives. For example, the Minnesota Department of Transportation estimates service life and analysis period for flexible pavements with bituminous bases to be 35 years (19). At this point, predicted lives of different rehabilitation measures should also be determined. They will be different depending on: pavement type, climate, geotechnical conditions, rehabilitation techniques, quality of workmanship, quality of materials used and so on. The predicted lives of the rehabilitation measures should be estimated based on local experience.

e. All relevant data must be gathered before proceeding with an analysis. These can be: all types of costs, location of highway materials, travel times, possible detours, present quality of pavements expressed in terms of riding quality, structural adequacy, surface distress, and so on.

2. The objectives can be divided into two groups:

- fixed objectives which must be fulfilled and
- variable objectives which can be optimized (minimized or maximized)

3. Next, measures of effectiveness must be established as means of comparing the different alternatives for reha-

bilitation. At this point a method of economic analysis should be chosen. Several different methods are possible with the present worth method and the annualized costs method being the most frequently used. These methods are used by 82 percent of 47 highway agencies in North America questioned as reported in Reference (19).

4. The generation of alternatives can be done using a checklist or a brainstorming technique. Some Pavement Management Systems are able to consider as many as 50 different rehabilitation alternatives (7).

5. The generated alternatives must be evaluated on the basis of established measures of effectiveness. Usually, alternatives will be compared taking costs into account. In such a case, all costs (and benefits) occurring during the life time of a facility should be considered although it is still common for only some costs, for example initial construction costs, to be considered. In other words, the life-cycle costs must be included in the analysis (1,2,3,7,19). The life-cycle costs of a pavement involve such costs as:

- design
- construction
- maintenance
- rehabilitation
- salvage value

- user costs
- interest
- inflation

The first two of these are the most obvious ones and are relatively easy to define them for each alternative being evaluated.

Maintenance, rehabilitation and salvage costs depend on the predicted lives of the maintenance and rehabilitation measures being implemented as well as that of the whole facility.

User costs are often not considered at all. According to Reference 19 only three agencies of 47 questioned claimed to take user costs into consideration while performing an economic analysis. However, none of those agencies revealed the procedure followed to obtain these costs. According to the World Bank (7) and the AASHTO Guide (1), the following costs should be included when the user costs are considered:

- fuel consumption
- oil consumption
- tire wear
- maintenance parts
- maintenance labour
- depreciation
- interest charges
- overhead
- passenger delays

- cargo holding

These include both the vehicle operating associated costs and the user travel time costs.

Accident costs associated with fatal and non-fatal car accidents and property damage during the accidents should be accounted for as well (1). It is relatively easy to estimate the costs if property damage or loss are of concern but in the case of injuries or fatalities the task is much more difficult, if not impossible.

Immediately after the pavement is properly constructed the user costs are kept to a minimum. However, as the pavement gets older, it becomes rougher and more slippery shortening the lives of travelling vehicles; parts wear sooner and maintenance costs rise. Also, because of roughness and slipperiness, the average running speed decrease causing an increase in the costs related to the user travel time. All of these factors can be translated to real dollar values. The example of above was shown in Reference 20. It was stated there that the operating speed of commercial vehicles depends on:

- present serviceability index PSI
- volume/capacity ratio v/c
- speed limit SL (mph)

$$S=2.404*(PSI)^{0.0928}*(v/c)^{-0.0275}*(SL)^{0.704} \qquad (2.3)$$

where S = the average operating speed in mph.

In the described work (20), the differential speeds and operational costs were obtained only from trucking companies.

Other user costs, which should be included in the analysis, and are comparatively easy to obtain, are delay related costs. Delays can be caused by slower movement caused by a deteriorated pavement surface or by pavement rehabilitation activity performed or during pavement reconstruction involving detours. In the case of detours, not only the user's delays should be considered but also the depreciation of the detour route.

Inflation and interest rate. It is generally well known that an amount of money at present does not necessarily have the same value as that same amount of money will h[illegible] the future. The reasons for this are inflation and [illegible]st (19). Inflation is a general increase in prices [illegible] the whole economy. Interest, on the other hand, is the [illegible]t of borrowing money. For example, if some rehabilitation activity is going to be performed in the future, in order to be able to implement it, one should have more money in the future than would be required for the rehabilitation to be performed now. It is clear from this that in order to properly evaluate the cost of a rehabilitation measure, two steps must be carry out:

- the future costs of the rehabilitation activity should be calculated by taking the present costs of materials, labour, design, and so on, then transforming these costs into the amounts that would be required in the future considering the expected rate of inflation

- using the expected interest rate, discount the future money back, as in the present worth method, or spread it over the analysis period, or any chosen period, as in the annualized cost method. In the first situation, one would then know how much money is needed now to perform the rehabilitation in the future; in the second case one would know how much money would have to be put aside monthly or yearly over a given period of time.

Although the above seems to be very straightforward, it is not a real life situation. The reason for this is uncertainty. The future inflation and interest rates can only be estimated. Yet another aspect of price changes, which is even more complex and difficult to deal with, is differential price changes.

As a summary to the above, one can say that taking inflation and interest into account while comparing rehabilitation alternatives can be a complex and difficult task to perform.

A questionnaire distributed to 49 highway agencies in North America indicates how confusing the task is (19). The discount rates (i.e. the real cost of money) used for analyses varied between 1 and 10 percent. Inflation rates used ranged between 0 and 12 percent. Twenty three agencies reported not using any discount rates whatsoever in their analyses, 32 do not take inflation into consideration and

19 do not consider either of these factors.

6. Sensitivity analysis is a very effective tool in performing an evaluation of rehabilitation alternatives when some factors vary (19). This analysis tests the effects of variations on an alternative selection, identifies the variables which influence the overall costs, and reveals the degree of that influence.

In the case of an economic evaluation where the service life length, interest rate, inflation, costs and user benefits are not fixed and the differences between the alternatives are not substantial, a sensitivity analysis can be particularly beneficial.

7. After performing a sensitivity analysis and recognizing the most influential variables and the extent of their influence on the output, certain assumptions regarding the variables can be made. Based on these assumptions the best alternative can be selected and implemented.

2.5 Summary

As stated earlier, the purpose of any highway system is to provide a safe, comfortable and economical means of transporting people and goods. However, only a limited amount of funds is available for the construction of new pavements or the old pavements reconstruction. It is generally accepted that the need for rehabilitation is continuously growing. Taking these into account and the amount of public funds currently being spent on pavement rehabilitation, it is clear that the money ought to be spent effectively.

At this stage, the concept of a Pavement Management System has evolved. This system would be responsible for producing a multi-year program for pavement rehabilitation aimed at utilizing the available funds in the most cost-effective manner (21). The Pavement Management System (PMS) should help the pavement engineer to answer two questions:

- what will be the future pavement standard of the road network, depending on the available budget?
- which maintenance and rehabilitation strategy will result in the highest rate of return to society for the investments made in preserving the road pavements (6)?

In order to answer these questions, the considered pavement must be properly evaluated. As pointed out in this Chapter, an adequate evaluation of pavement structural capacity, functional ability, safety and surface condition is of inestimable importance. New methods for acquiring information regarding a pavement structural condition have been mentioned. As also described, the measurement of both the pavement roughness and skid resistance have been automated. Some problems have been encountered in the automation of pavement surface condition evaluation, but advances have been made. The ARAN and the Swedish laser road tester are good examples of this.

After the overall pavement evaluation is implemented, the rehabilitation alternatives must be chosen. They should be based on the material availability, local experience, and estimated lives of particular rehabilitation measures.

This investigation reveals that, at present, two cost analysis methods are used most often in North American practice: present worth method and annualized cost method. It was also highlighted that factors like the interest rate, inflation, user costs are frequently not taken into consideration. It should be emphasized that the above fact may lead to an incorrect choice of alternative for implementation so that the chosen may not be the most cost-effective solution resulting in unnecessary spending.

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CHAPTER 3

FULL-DEPTH ASPHALT CONCRETE PAVEMENTS IN SEASONAL FROST AREAS

3.1 Introduction

Full-depth asphalt concrete pavement as defined by the Asphalt Institute is a pavement in which only asphalt concrete layers are used for all courses above the subgrade (1).

It is not clear when the asphalt concrete layers began to be employed for all pavement courses but it appears that the AASHO Road Test was the first modern case of using full-depth pavement in road construction (2, 3, 4). Before that there were some trials like those mentioned by Foster (5) or Ellison (6) in Arkansas and Virginia, respectively in the 1940's but those roads can be considered as conventional-type pavements with asphalt concrete bases.

The first modern experiment where full-depth asphalt pavement was employed was the AASHO Road Test constructed in Ottawa, Illinois, USA and carried out from 1957 to 1960. The findings of that experiment were so encouraging that further development and construction of full-depth pavement structures has been observed since that time.

In 1973 a questionnaire as a part of NCHRP - Synthesis of Highway Practice No.26 (7) was mailed to North American highway agencies regarding pavement design criteria for seasonal frost areas. Out of 62 responding agencies, 40 reported that frost action was considered in their pavement design procedures and out of those 40, 20 constructed full-depth asphalt concrete pavements. Eleven agencies representing a variety of conditions were selected for further consideration. Table 3.1 summarizes comments from the eleven agencies regarding the use of full-depth asphalt concrete pavements.

This chapter reviews some detailed experiences with full-depth asphalt concrete pavements in seasonal frost areas other than the Province of Alberta. Experience in Alberta will be discussed in more detail in Chapter 4.

3.2 AASHO Road Test Experiences with Full-Depth Pavements

Among many other pavement sections constructed, two full-depth asphalt sections were constructed in loop no.3. The surface layer 75 mm (3 in.) thick was laid over a wedged shaped asphalt concrete base which thickness varied from 50 to 280 mm (2 to 11 in.) (4). There was no subbase under the sections. The bituminous base material was a high quality plant mix with high stability.

The results of the test showed that the full-depth sections performed well compared to other sections constructed i.e. with crushed stone bases, cement-treated bases and bituminous bases with a subbase under them. The thinnest full-depth A.C. pavement sections still held a Present Serviceability Index (PSI) of 4 at the end of the test, that is the 127 mm (5 in.) structure for the 54 kN (12 kip) single axle load design and the 178 mm (7 in.) pavement for the 108 kN (24 kip) tandem axle load. In this respect only bituminous sections with subbase had better performance. A chart relating thickness of base with depth of ruts shows that the sections with bituminous bases performed poorer than the sections with cement-treated bases and better than the sections with crushed stone bases. Deflections under the 80 kN (18 kip) single axle load found by means of the Benkelman Beam were generally the lowest compared to those obtained for crushed stone bases and cement-treated bases.

It was also noted during the Test that the Benkelman Beam deflections for the full-depth pavements were equal or higher in the summer than in the spring even though most structural damage was associated with the spring period. It was suspected that those high summer deflections were the result of the greater flexibility of the bituminous materials at higher temperatures.

The Canadian Good Road Association Observer Committee on the AASHO Road Test (4) reports that the bituminous treated base material used was superior in its load-supporting capacity to the cement-treated base material. That particular investigation indicated that one inch (25 mm) of bituminous treated base had the same load supporting capacity as 1.7 to 2.4 inches (43 to 61 mm) of crushed stone base course, while one inch (25 mm) of cement treated material had the same load supporting capacity as 1.7 inches (43 mm) of crushed stone base course.

Data regarding construction of full-depth asphalt concrete sections at the AASHO Road Test are given in Tables 3.2 and 3.3.

The AASHO experiment with full-depth pavement structures was so encouraging that many agencies began to use the concept and further investigations were carried out on different test sections.

Those endeavors were especially supported and promoted by The Asphalt Institute in its 1964 "Thickness Design Manual - Seventh Edition"(1). The authors of the cited work outlined the reputed advantages of Full-Depth asphalt pavements (as they designated this type of pavement structure) as:

- because they have no permeable granular layers there is no means to entrap water in the structure
- subsurface drainage is not required
- time required for construction is reduced
- construction season can be extended provided the pavement is constructed in over 100 mm thick lifts

- they are thinner - less material of high quality is required, less interference with city utilities
- they provide uniformity of the pavement structure
- they show lack of a spring-thaw effect
- they minimize construction delays due to bad weather conditions
- they have special advantages for stage construction

Although the Manual recommended the use of Full-Depth structures it had provisions for different structures e.g. structures with untreated granular bases. According to the Manual there is no constant factor for converting a given thickness of asphalt layer into a thickness of untreated granular base that will provide equivalent load-supporting capacity. The Asphalt Institute recommended the use of a Substitution Ratio (S_r) for making a thickness conversion from asphalt layer to untreated granular material layer:

$S_r = 2.0$ in case of high-quality untreated granular base material

$S_r = 2.7$ in case of low-quality untreated granular base material

The concept of layer coefficients developed in 1948 by Hveem was further utilized in the AASHO Road Test (8). The layer coefficient is the empirical relationship between structural number SN of a pavement structure and layer thickness, which expresses the relative ability of a material to function as a structural component of the pavement (9).

The SN in turn is defined as an index number derived from an analysis of traffic, road-bed soil conditions and regional factor that may be converted to thickness of various flexible pavement layers (9).

The layer coefficients for three basic road materials were determined based on the Test findings:

- high stability asphalt concrete mix $a_1=0.44$
- base course crushed stone $a_2=0.14$
- subbase course sandy gravel $a_3=0.11$

Several highway agencies developed their own layer coefficients based on the AASHO Road Test and local experiences, compatible with the local highway materials.

Another approach of the concept was used by the National Asphalt Pavement Association (8). In Reference 8 the term thickness equivalency is used to relate the relative value of layers in a flexible pavement to the value of dense graded aggregate base course. These thickness equivalencies are:

- hot-mix asphalt surface and binder 3.14
- dense graded crushed aggregate base 1.00
- sandy gravel subbase 0.79

In the mid-1960's and early 1970's several test roads were constructed and contained sections with full-depth structures. Six of these test roads located in seasonal frost areas will be discussed in detail since their environmental conditions are important to the performance of full-depth pavements in Alberta.

These test sections are:

1. Brampton Test Road, (Brampton, Ontario)
2. Ste.Anne Test Road, (Ste.Anne, Manitoba)
3. CRREL Test Roads, (Hanover, New Hampshire)
4. Minnesota test sections
5. Saskatchewan Highway 2-9
6. Ordway Colorado experimental base project

3.3 Brampton Road Test

The road link which contained 36 test sections was constructed in 1965 on Highway 10 north of Brampton, Ontario (10,11). There were four full-depth sections with varied thicknesses of asphalt concrete base (Figure 3.1). The surfacing layer of those sections was constructed in two lifts. The top lift was 38 mm (1.5 in.) thick, the bottom lift was 51 mm (2 in.). The surfacing layer was the same for all test sections. Four thicknesses of asphalt concrete base were used: 50,100,150,200 mm (2,4,6 and 8 in.) in the sections 1,2,5,and 3 respectively. Data regarding the asphalt concrete layers are shown in Table 3.4.

The subgrade was uniform for the all test sections and consisted of clay borrow placed over the existing ground to provide uniformity of the subgrade materials. In order to achieve even better uniformity the top 150 mm (6 in.) layer of the subgrade was reworked prior to placing the first asphalt concrete layer. The subgrade was compacted using sheepfoot and vibrating steel-wheeled rollers at the optimum moisture content. Data regarding the subgrade conditions are shown in Table 3.5.

Performance of Full-Depth Pavements at the Brampton Road Test

Mean deflections and temperatures of measurements during four year period are recorded in Table 3.6. The deflection versus equivalent thickness chart developed after four years of the test showed that full-depth pavements perform significantly different than other types of construction, but there was a tendency for the two groups of structures to behave similarly as the years progressed (10).

The rutting observed during the test was least on the cement-treated bases and greater on the asphalt concrete bases. In case of the pavements with the unbound granular bases rutting was the most pronounced. The full-depth asphalt concrete pavement behaved significantly better in this respect than the structures where besides the asphalt concrete layers also subbases were employed.

With respect to the surface distress, the full-depth sections performed very well. None of four full-depth sections had exhibited cracking after four years in service.

Frost-heave movements were the least for the full-depth structures compared to the others used in the Test. This is significant that deep-strength sections i.e. sections which employed a granular subbase under asphalt concrete layers, heaved more than the full-depth sections what means that the subbase material contributed to frost-heaving. The riding quality expressed in terms of Riding Comfort Index remained consistently higher for the full-depth sections then for the other types of structures used in the Test.

The cited data prove that after four years of experience full-depth asphalt concrete sections at Brampton Road Test performed superior to the other sections constructed.

Layer equivalencies based on the Brampton Road Test findings are given in Table 3.7.

Very valuable data on the performance of full-depth sections which have not been published were obtained by private communication with personnel of Ontario Ministry of Communications (11). This information has been summarized and is shown in Figure 3.2. This figure, which is an extension of Figure 20 and 43 of Reference 12 as well as the mentioned private communication, shows the Riding Comfort Index performance of the full-depth sections. The following is illustrated in Figure 3.2:

- section no.3, 290 mm thick (11½ in.) performed 20 years before it reached the terminal RCI and was overlaid,
- section no.5, 240 mm thick (9½ in.) was overlaid for the first time after 13 years of service
- section no.2, 190 mm thick (7½ in.) was divided into

two subsections. The first subsection was first overlaid 6 years after construction and again 14 years later. The second subsection was first overlaid in 1978 i.e. 13 years after construction and was still carrying loads in 1985 i.e. 20 years after construction.

- section no.1, only 140 mm thick (5½ in.) was overlaid twice, for the first time in 1971, 6 years after construction and secondly in 1985.

The accumulated equivalent 80 kN (18 kip) single axle loads carried are estimated to be in the range of 2 million repetitions. The information is based on Figure 43 of Reference 12 as well as the work of Morris et al. (13).

3.4 Ste. Anne Road Test

This test road is situated 40 km east of Winnipeg near Ste. Anne, Manitoba on the Trans-Canada Highway. The road was constructed in 1967 and was composed of 29 test sections, two of which were full-depth asphalt concrete structures (14,15).

The full-depth sections had 250 mm (10 in.) of asphalt concrete pavement laid directly on a clay subgrade. Each of these were constructed using different asphalt cement grades. One employed a high viscosity 150-200 penetration (HV) asphalt whereas the second used a low viscosity 150-200 (LV) asphalt. The rest of the data regarding the asphalt concrete mix used is shown in Table 3.8.

The subgrade soil was a fairly uniform, very heavy (highly plastic) clay described as A-7, with the group index of 20 under the AASHO Soil Classification System. The liquid limit values ranged from 90 to 105 and the plasticity indices ranged from 55 to 65. The soaked laboratory CBR value was determined to be 3. Strict control of the uniformity of soil type and compaction was maintained during construction. The other data regarding the subgrade are shown in Table 3.9.

In order to evaluate the structural response of the test sections, Benkelman Beam deflections were taken during 1968. The results are shown in Table 3.10. It should be emphasized that the Ste. Anne Road Test was designed to study the transverse cracking of asphalt pavements and results are mainly concerned with this problem.

After two years in service, section no.64 with a 150-200 pen LV asphalt cement exhibited 40 percent of the transverse cracking found in a comparable conventional section located on the same subgrade. Section no. 65 with a 150-200 pen HV asphalt cement showed no sign of transverse cracking for that period of time. Further experience with the full-depth pavements proved that they still performed better in terms of transverse cracking than the conventional sections. For the LV 150-200 asphalt grade there were 82 cracks per kilometer and in the case of the HV 150-200 asphalt 50 cracks per kilometer were counted.

The Ste. Anne test road was designed structurally for early failure and had to be overlaid in 1975, eight years after construction. The overlay was 125 mm (5 in.) thick with the SC-5 asphalt type used.

In a report published after an inspection of the pavement which was done 20 years after construction and 12 years after the overlay it was revealed that 50 percent of cracks in the case of LV 150-200 asphalt and 66 percent in the case of HV 150-200 asphalt reflected through the overlay (16). It should be noticed that after the overlay the LV 150-200 sections showed 56 percent and the HV 150-200 sections 46 percent of the number of cracks found in the conventional type sections.

3.5 U.S. Army Cold Regions Research and Engineering Laboratory Full-Depth Test Sections

3.5.1 First Full-Depth Test Sections, Hanover, New Hampshire

The test sections were constructed in 1971 in Hanover, New Hampshire, USA. The scope of the test was to compare full-depth asphalt concrete pavement sections with conventional type sections designed according to the US Corps of Engineers procedure. The sections were designed for "light" and "heavy" traffic for a design period of 20 years (17,18). The full-depth sections had 125 and 230 mm (5 and 9 in.) of asphalt concrete layer (section no. 4 and no. 3

respectively) laid directly on a silty subgrade, classified as ML under the Unified Soil Classification and as F-4 under the Corps of Engineers frost group classification. The "normal period" CBR of the subgrade soil was 8. The in-situ moisture content ranged from 25 percent to 35 percent prior construction whereas the optimum moisture content of the soil was 13 percent. The subgrade was reworked before construction to obtain a uniform layer 300 mm (12 in.) thick. The conventional type sections were constructed using the Corps of Engineers criteria. Section no. 1 consisted of 100 mm (4 in.) of asphalt concrete, 530 mm (21 in.) of granular base and 125 mm (5 in.) of sand which served as a filter layer. Section no. 2 had 100, 330 and 125 mm (4, 13 and 5 in.) of asphalt, gravel and sand respectively.

Sections 1 and 3 were designed for an average daily traffic load (ADTL) of 4000 vehicles for two directions whereas sections 2 and 4 were designed for an ADTL of 100 vehicles (total for both direction).

The mean freezing index for the Hanover location was determined to be 1011 C°-days (1820 F°-days).

During the Test the following data were gathered:

1. frost penetration
2. variation in subsurface moisture
3. frost heave
4. deflection

1) Observation indicated that the conventional type pavements had deeper frost penetration than the full-depth asphalt concrete structures. This is attributed to the fact that the moisture content of the granular base was lower than the moisture content of the silty subgrade. This phenomena is due to the fact that an additional amount of heat is required to change the contained water into ice, so that the wetter a soil the slower the frost front penetrates into the soil. Therefore frost penetration under conventional pavements is generally deeper than under full-depth sections laid directly on, usually wetter, subgrade.

2) Observations of the moisture content in the subgrades under the full-depth pavements showed that the amount of moisture ranged from 26 to 55 percent. During the spring thaw-weakening period the moisture content increased dramatically by 10 to 15 percent. The moisture which was trapped between the pavement and the frozen subgrade below, resulted in very high Benkelman Beam deflections of the 125 mm (5 in.) section. The 230 mm (9 in.) full-depth section was strong enough to distribute the load evenly.

3) Deflection measurements showed, as it was expected, that the 125 mm (5 in.) full-depth section was the weakest of the all and deflected the most, an average 3.2 times greater than the other sections.

Seven year average maximum deflection were (17):

section 1	0.88mm	0.0346in.
section 2	0.82mm	0.0322in.
section 3	0.94mm	0.0371in.
section 4	2.97mm	0.1171in.

The investigators concluded that the 125 mm (5 in.) full-depth section met its design life of 21,900 equivalent single axle loads (ESAL) and failed soon after the 1975 spring thaw i.e. it was 4 years in service whereas the equivalent conventional section met its design life of 21,900 ESAL in 1975 and continued to carry more than 44,700 ESAL with no failure up to 1980. Both the 230 mm (9 in.) full-depth section and the 760 mm (30 in.) conventional section did not reach the end of their design life and had not failed at the time of the report.

3.5.2 Second CRREL Test Area

Based on observations of full-depth sections during the first CRREL Road Test the loss of strength of pavement during the thaw period was caused by the trapped excess moisture under the pavement. In order to promote removal of moisture from under the full-depth pavement a new concept of construction of full-depth asphalt concrete pavement was developed. A filter fabric placed on a prepared subgrade was overlaid by a 50 mm (2 in.) thick open graded layer of asphalt concrete followed by 75 mm (3 in.) of a dense, standard asphalt concrete layer (18). A road section which employed that concept was constructed in 1974 in Hanover, New Hampshire but due to the weather conditions prior and during construction the subgrade was not adequately prepared i.e. it was not reworked, blended and dried. The subgrade was highly frost-susceptible and contained varved silt and silty clay. Because of these facts severe differential frost heaving was observed after the first winter in service. In 1975 the pavement was removed, the subgrade reworked to a depth of 600 mm (24 in.) in one case and 300 mm (12 in.) in the other and the pavement was laid in the same manner as before. After the reconstruction, despite higher freezing indices, the section exhibited very uniform frost heaving.

Benkelman Beam deflections of the reconstructed section were approximately 1.5 times greater than those of the first full-depth pavement 125 mm (5 in.) thick.

3.5.3 South Balch Street, Hanover, New Hampshire

In September 1974 a 150 mm (6 in.) thick full-depth pavement was constructed in the town of Hanover on a residential street (19). The asphalt concrete layer was placed on a highly frost-susceptible silty and clayey subgrade. The subgrade soil was classified as ML and ML-CL under the Unified Soil Classification and F-4 under the Corps of Engineers frost group classification. The design of the street was based on a poor subgrade CBR value of 3.5. During construction of the street no special attention was paid to preparation of the subgrade. As the result the pavement on the street had heaved substantially from 50 to 208 mm (2 to 8.2 in.) and severe differential heaving was observed at manholes, catch basins and so on.

3.5.4 Frost Effects Research Facility (FERF)

It is important to mention a new U.S.A. CRREL facility recently opened in Hanover, New Hampshire. The new facility consists of a 2700 m^2 (29000 sq.ft.) building equipped with surface panels to freeze pavement layers and subgrade soils of a tested pavements (20). The building test area is 55 m (182 ft.) long and 14 m (45 ft.) wide and incorporates twelve test basins. The equipment is designed to achieve test temperatures ranging from -37 ^oC to +32 ^oC (-35 ^oF to +90 ^oF).

To date pavement test sections which utilized various different bases, subgrades, and pavement types have been tested in the facility, but as yet unreported. It is expected that the pavement tests will continue for the next two to three years.

The facility allows the simulation of deeply frozen pavement sections, permafrost, summer conditions in permafrost and near-surface thawing. It is expected that the use of the facility can help to formulate and solve many pavement problems including in particular a better understanding of the behavior of full-depth asphalt concrete pavements located in seasonal frost areas.

3.6 Evaluation of Full-Depth Asphalt Concrete Pavements in Minnesota

The purpose of this investigation was to determine the temperature and seasonal effects on deflections of full-depth asphalt pavements and to determine the granular equivalency for the full-depth sections (21).

In order to evaluate the performance of the pavements the project, which contained 26 test sections located throughout the state of Minnesota, was implemented. The test sections included a wide variety of the subgrade soils and pavement thicknesses.

During the test, core samples of the pavements and soils were taken. To evaluate the performance of the test sections, Benkelman beam deflections were obtained systematically together with temperature measurements. Besides depth of rutting on an annual basis, roughness and surface visual condition were evaluated. The laboratory work consisted of the determination of bituminous layer thicknesses, densities, extractions, gradations, penetrations and air voids contents. The subgrade samples were examined using the Hveem stabilometer R - value, moisture - density relations, gradations and Atterberg limits.

Generally three cores were examined from each pavement section and the Benkelman beam deflections were taken at 15.24 m (50 ft.) intervals.

The investigation reported the following conclusions with regard to the behavior of full-depth asphalt concrete pavement sections:

1) The maximum deflections uncorrected for temperature influence occurs from mid-June to mid-July. This fact was connected with the high pavement temperatures at this time of the year. The temperature corrected maximum deflections occurred in mid-May compared to mid-April for the conventional type pavements. It was found that the temperature correction factors increase quite rapidly at pavement temperatures below 26.7°C. Above this temperature the full-depth pavement strength decreases significantly. In the case of conventional pavements with a granular base and thinner asphalt layers the pavement temperature has not such a great influence on strength.

2) Because of the moderating influence of the asphalt mat temperature the full-depth pavement deflections are high throughout the year and the highest and the lowest deflections differ at most by 30 percent.

3) It was found that the thinner the pavement, the larger the spring recovery factor that should be applied.

4) Because the spring recovery factor is small it is difficult to determine a weak period for full-depth pavements, so that the protection of full-depth pavements by using an

axle-load restrictions during a spring time is doubtful.

Based on the Benkelman beam deflection measurements a Granular Equivalent (GE) was determined. The analysis shows that there is no single GE value for full-depth pavements. The equivalent depends on the thickness of the bituminous layer. It was reported that greater depths of asphalt layers produce higher equivalency factors. A design chart for full-depth pavements has been presented.

In a special study carried out during the full-depth pavement investigation it was found that the edge effect on full-depth pavements located on plastic soils was significant. It was found that the pavement deflects 30 percent more at the edge of the pavement than 60 cm (2 ft.) from the edge.

The overall conclusion drawn from the investigation from 1971 to 1979, was that the performance of full-depth asphalt concrete pavements was comparable with conventional sections when both had the same peak season temperature corrected deflections.

3.7 Performance of Thin Full-Depth Asphalt Concrete Pavements in Saskatchewan

In 1975/76 a test section was constructed on Saskatchewan highway 2-9 (22). The purpose of the section was to evaluate the performance of various thin or staged pavement structures, of varied width and shoulder surface types. In this work only sections built as full-depth structures will be considered. These particular sections were constructed in 1976. There were nine full-depth sections built, all of them 800 m (2625 ft.) long. Two widths of the roadway were used: 7.3 m (24 ft.) and 8.5 m (28 ft.). The depth of the pavements varied from section to section and was: 160 mm, 130 mm, 100 mm, 80 mm, and 50 mm (6.3, 5.1, 4, 3.1, and 2 in.). The binder type also varied. Three asphalts were applied: AC-6, AC-1.5 and SC-4.

The shoulders of the tested sections were treated in five ways. The following shoulder types were utilized:

- paved
- primed base
- unprimed base
- unprimed base uncompacted
- compacted soil

The Average Daily Traffic ADT for the highway was 375 vehicles in 1976 with 15 percent of trucks. The estimated number of equivalent 80 kN (18 kip) axle passes were 85,700 by the end of 1982 for all the full-depth sections. Performance of the test sections were determined by examination of crack development, peak deflection measurements using the Benkelman beam and Dynaflect, measuring deflection slopes, rut depth and dynamic modulus of the materials used in the construction of the pavement layers.

A. Crack development.

Two main types of cracking were observed:

- fatigue cracking
- thermally induced transverse cracking

Generally, the structures less than 100 mm (4 in.) thick failed because of fatigue cracking by 1979, that is 3 years after construction. The transverse cracking did not develop on these sections to a significant extent by that time.

In 1978, two years after construction, the transverse cracking began to develop in the sections thicker than 100 mm (4 in.).

B. Benkelman beam peak deflections.

It was found that deflections increased with time for all full-depth thicknesses. The relationship between deflections and thicknesses was found and it was determined that the relationship changed with time as follows:

$$d = -0.0045 \times T + 1.45 \qquad \text{in 1977} \qquad (3.1)$$

$$d = -0.0106 \times T + 2.80 \qquad \text{in 1981} \qquad (3.2)$$

where d - maximum deflection in mm

T - asphalt layer thickness in cm

The peak deflections were also determined using the Dynaflect equipment. The beam deflections were 14 to 28 times larger than the Dynaflect deflections, with deflections measured by the two procedures showing the same increasing trend with time.

C. Deflection slopes.

The deflection slopes were obtained for both the Dynaflect and the Benkelman beam procedures. The deflection slope was defined as the difference between the peak deflection and the one 300 mm (1 ft.) apart. There is a correlation between the deflection slope and the bending stress induced in a pavement layer: the greater the slope, the higher the bending stress under an imposed load. It was determined during the experiment that the deflection slopes were increasing with time for all the sections tested.

D. Dynamic moduli.

Dynamic moduli were determined using wave propagation techniques. The moduli were then corrected to a standard temperature of 10°C. In order to determine elastic moduli using wave propagation techniques a continuum medium between the vibrator and the receiver is necessary. Because this condition has not been fulfilled for very badly cracked sections the results for these sections are in question. For asphalt concrete layers the moduli ranged between 9,400 and 17,500 MPa (1,363 and 2,540 ksi) during 1978-1982 and 10,000 to 13,200 MPa (1,450 to 1,914 ksi) during 1980-1982. The above finding indicates that the uniformity of the moduli developed for all the full-depth sections with time. The summer dynamic moduli for the subgrade remained in the range of 140-250 MPa (20.3-36.3 ksi) throughout the 6-year period and were only slightly higher under the full-depth sections than under the sections with granular bases.

F. Rutting.

The collected data show that an increase in rut depth is correlated with time. It shows that for the same thickness the increase of rut depth is lower for the full-depth asphalt concrete pavements then for a conventional type of pavement. The data also indicate decrease of rut depth with increase of asphalt concrete layer thickness. No effect of asphalt grade or type on rutting was observed during the investigation. It seems also that the lane width had no effect on rutting.

Summary. It can be concluded after [illegible]th year of observation that full-depth asphalt concrete pavements 100 mm (4 in.) and thicker performed well. No recent information regarding the performance of the sections is available in the literature.

3.8 Ordway Colorado Experimental Base Project

This project was implemented as the extension of the AASHO Road Test with the purpose to apply the AASHO Test Road findings to the Colorado environmental conditions (23). The sections were open to traffic in 1965 and the final set of measurements were done in 1978. There were 26 test sections constructed, each 137.2 m (450 ft.) long. The subgrade soils were determined as A-7-6 under the AASHO Soil Classification System with the CBR value of 2.6 and

A-6 with the CBR value of 3.4.

Pavement thicknesses were selected to give a maximum design life of approximately 20 years. It has been estimated that the cumulative number of 80 kN (18 kip) ESALs was 140,000 during the 13 year period from 1965 to 1978.

The asphalt concrete surface layer of the test sections contained 5.8 percent of asphalt cement. Well-graded aggregate was employed in the mix. The coarse aggregate of the mix had at least 60 percent crushed material.

Analysis of the routine field-performance data shows that base thickness had almost no effect on the Present Serviceability Index (PSI). This fact indicates that most of the loss in PSI observed after 12 years was not load associated. It was concluded that surface erosion of the mat had a large effect on PSI. Most of the PSI loss can be attributed to environmental influences, however, the presence of alligator cracking in the conventional sections with thin asphalt layers and untreated bases and the presence of small amount of rutting in all the sections indicate that axle loads had some influence on the PSI loss.

All constructed test sections exhibited a considerable amount of transverse cracking. It was assumed that these cracks were of the low-temperature induced type because they were observed after the first particularly severe winter.

The most prevalent form of cracking in the asphalt base sections were transverse and longitudinal linear cracking whereas alligator cracking or the load related type cracking was a major factor of distress in the case of the untreated base test sections.

Rutting was observed on all sections; however, for the conventional untreated base sections was of lesser extent than on the asphalt treated base sections.

During the study a special investigation of the subgrade moisture content was conducted. The investigation shows that the subgrade under the full-depth asphalt concrete pavements had a tendency to dry out whereas the subgrades under the conventional pavements were getting wetter with time.

The investigation also showed that large seasonal variations in the deflections were present. The above fact is related more to differences in pavement temperatures than to moisture or frost effects on the subgrade.

It was concluded that the sections with asphalt concrete bases provided good resistance to rutting and the best resistance to all forms of cracking. The conventional type of pavements provided the best performance in terms of rutting but the worst in terms of alligator or load-associated cracking. All test sections after 12 years of traffic without major maintenance exhibited substantial erosion of the surface and severe transverse cracking. Those conditions were described as somewhat more severe than those observed on adjacent highways.

3.9 Summary

From the above cited literature certain advantages and disadvantages of full-depth asphalt concrete pavements can be inferred.

According to the AASHO, Brampton, and Saskatchewan tests full-depth asphalt concrete pavements performed very well when rutting is of concern. Only pavements with cement-treated bases showed better performance in this respect. On the other hand permanent deformations in sections with granular bases were greater than in the case of full-depth pavement structures.

The Colorado test does not support this experience, however. The conventional type of pavement employed during the test was superior to the full-depth pavements in this respect. The above fact may be attributed to substantially higher pavement temperatures that can be expected during the summer months.

Surface deflections and deflection slopes increased with time for all tested sections during the cited observations and the investigated full-depth sections were no exception. It means that bending stresses at the bottom of the asphalt bound layers increase with time. All the tests prove that full-depth sections have a different pattern of maximum pavement deflections than their conventional counterparts. Maximum deflections for full-depth pavement appear much later in the season than in the case of conventional structures and the peak deflection period is not so pronounced. This fact is due to the decrease of asphalt viscosity with increase of pavement temperature during the spring time.

The Ste. Anne Road Test showed that amount of transverse thermal-induced cracking was much less in the case of full-depth structures than the conventional sections with granular bases. This however, was not the case in Colorado where it was found that the low temperature transverse cracking and longitudinal cracking were the most prevalent ones in the case of full-depth pavement whereas fatigue cracking was the most prevalent in the case of conventional structures.

The CRREL and Colorado experiences regarding subgrade moisture content are contrary to each other. The CRREL tests reveal rapid increase of subgrade moisture content under full-depth pavements whereas the findings of the Colorado investigation say that there was a tendency for full-depth subgrades to dry up with time rather than become wetter.

In the case of frost penetration full-depth pavements show superiority to the other pavement structures. Because of their insulating properties frost penetration is not as great as in the case with the pavements with granular bases. Frost heave, which is related to the depth of frost penetration, is less for full-depth pavements; however, it was found that thin full-depth sections can be very susceptible to differential frost heaving. This fact highlights the importance of proper subgrade preparation under full-depth pavements. The subgrades should be reworked, blended, and properly compacted to achieve the best possible uniformity.

In summary, according to the cited experiences, full-depth asphalt concrete pavements show that when properly designed and constructed they perform as well as their conventional type counterparts.

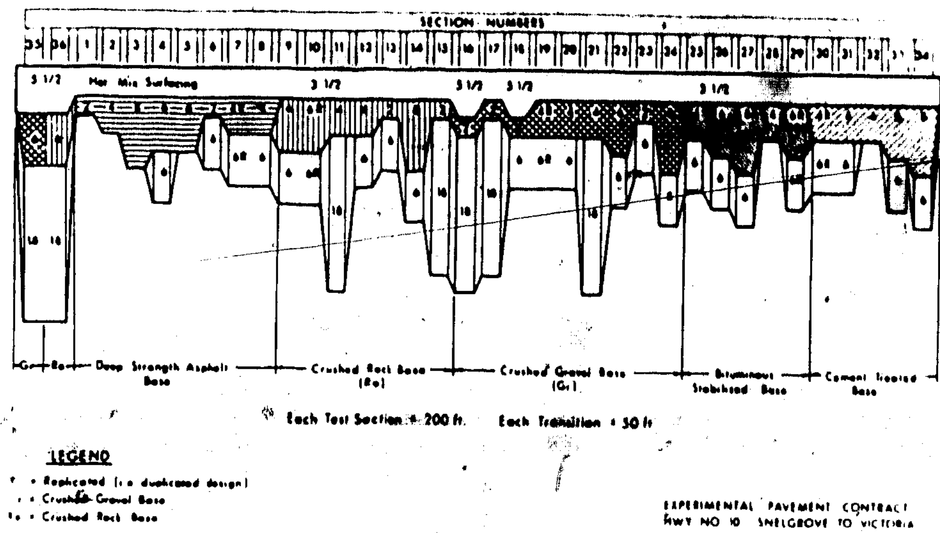

Figure 3.1 Layout of the Brampton experimental pavement sections

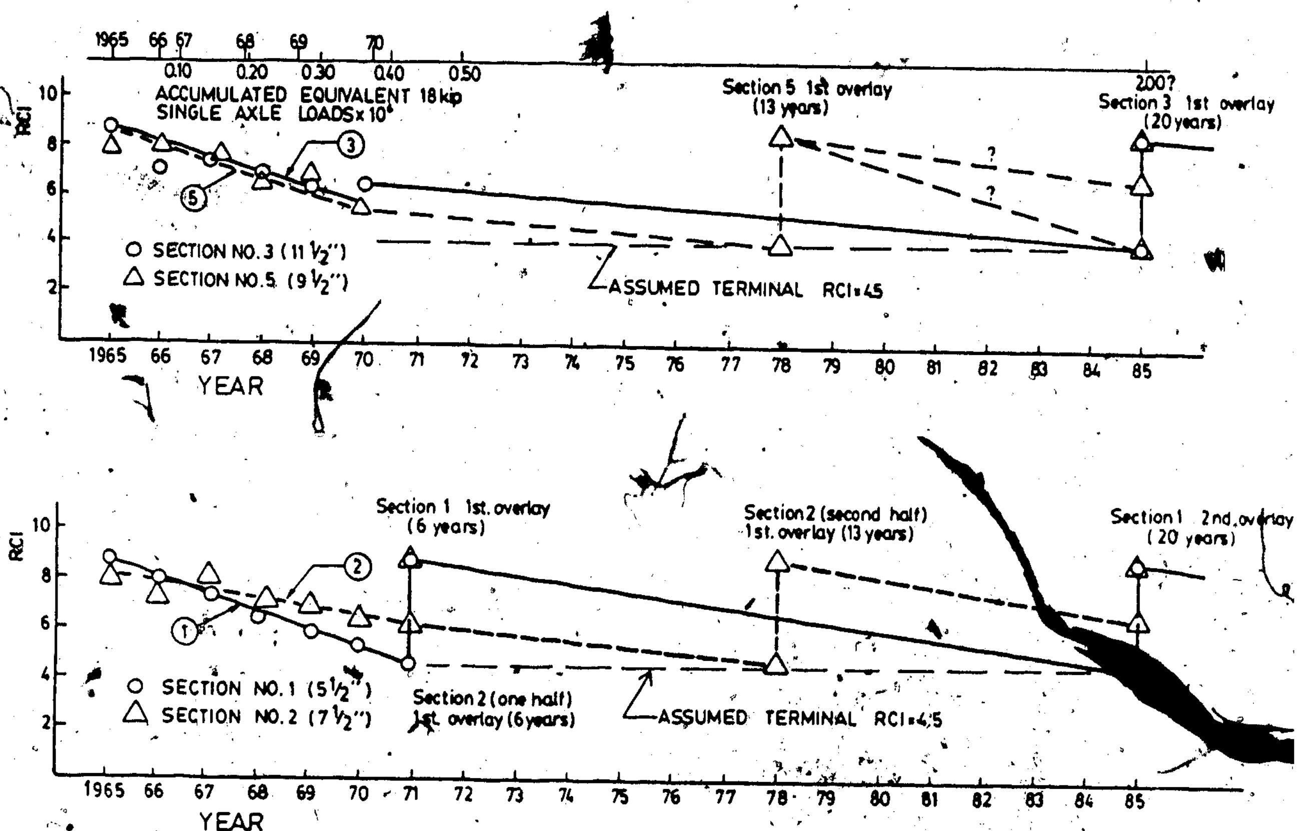

Figure 3.2 RCI versus Time for Full-depth AC Sections at the Brampton Road Test

Table 3.1 Use of Full-Depth Asphaltic Pavements by Eleven Agencies

State or Province	Have you used full-depth asphaltic concrete pavements?
Alberta	Yes, have used 5 years. Have about 300 - 400 miles on primary roads and 300 - 400 miles on secondaries.
Colorado	Yes, about 50 miles, on primary and secondary roads. Choice based strictly on cost, except not used on Interstates, where always require unbound subbase.
Idaho	No, but expect to use over good subgrades.
Maryland	Use thick AC pavements extensively, but always use 4 in. crushed stone unless subgrade is gravel.
Maine	No, but [illegible] using greater thickness of asphaltic material.
Nebraska	Yes. Present design for intermediate and secondary roads is full-depth.
New Hampshire	No. Concerned about possible greater tendency for low-temperature contraction cracks, and reluctant to place AC directly on FS subgrade.
New York	No. This concept is objectionable because it violates the principle that strength needs decrease with depth.
Saskatchewan	Yes. About 200 miles, from 7.5 to 9.5 in.
Wisconsin	Yes, two projects of 6 miles each. One is 9.25 in., the other 12 in.
Corps of Engr.	No, except experimental pavements.

Table 3.2 Subgrade properties at the AASHO Road Test

CBR percent	2 - 4
Liquid Limit LL	29
Plasticity Index PI	13
Optimum moisture content %	15
Max. dry density (pcf) (Mg/cu.m)	116 1.86
Percent compaction	97.7
Constr. moisture cont. %	16

Table 3.3 Asphalt concrete mix properties at the AASHO Road Test

	surface layer	binder layer
Agg. passing No.200 sieve percent	5	4
Max. Marshal density (pcf) (Mg/cu.m)	151 2.42	154 2.47
Percent compaction	97	97
Percent asphalt content	5.4	4.5
Asphalt penetration	85 - 100	
Percent air voids content	3.6	4.8

Table 3.4 Properties of asphalt concrete used at the Brampton Road Test

	surface course	binder course	base course
amount of asphalt %	5.7	6.2	5.8
air voids %	5.5	4.9	7.2
% aggregate passing No.200 sieve	3.7	4.8	3.0
asphalt cem. pen.	85 - 100	85 - 100	85 - 100
% compaction	97.4	96.8	96.3

Table 3.5 Subgrade properties at the Brampton Road Test

Test section No.	1	2	3	4
CBR %		2.5		
Liquid Limit %		29		
Plasticity Index		13.5		
Field dry density (lbs/cu.ft)	131.5	121.3	120.7	129.7
(mg/cu.m)	2.11	1.94	1.93	2.08
Proctor max. density (lbs/cu.ft)	122.2	123.6	122.7	125.7
(mg/cu.m)	1.96	1.98	1.97	2.01
% compaction	107.6	98.1	98.4	103.2
% moisture content	8.9	9.4	9.9	8.8

Table 3.5 Deflections of full-depth sections at the Brampton Road Test

Section No.			1	2	5	3
thick in. (mm)		surface	3.5 (90)	3.5 (90)	3.5 (90)	3.5 (90)
		base	2 (50)	4 (100)	6 (150)	8 (200)
		total	5.5 (140)	7.5 (190)	9.5 (240)	11.5 (290)
year	1966	defl. in. (mm)	0.043 (1.09)	0.057 (1.45)	0.089 (2.26)	0.072 (1.83)
		temp. F (C)	84 (29)	83 (28)	94 (34)	62 (17)
	1967	defl. in. (mm)	0.029 (0.74)	0.041 (1.04)	0.065 (1.65)	0.048 (1.22)
		temp. F (C)	95 (35)	85 (29)	94 (34)	62 (17)
	1968	defl. in. (mm)	0.026 (0.66)	0.031 (0.79)	0.047 (1.19)	0.033 (0.84)
		temp. F (C)	92 (33)	88 (31)	94 (34)	85 (29)
	1969	defl. in. (mm)	0.023 (0.58)	0.023 (0.58)	0.032 (0.81)	0.023 (0.58)
		temp. F (C)	79 (26)	87 (31)	94 (34)	70 (21)

Table 3.7 Layer equivalencies based on equal terminal RCI of 4.5 developed during the Brampton Road Test.

Pavement structure	Equivalent in in. of granular base
Asphalt concrete in conventional or deep strength construction	2.0
Asphalt concrete in full-depth	3.4
Bitumen stabilized base	1.1
Sand subbase	0.6
Granular base	1.0

Table 3.8 Properties of asphalt concrete at the Ste. Anne Road Test

Section number	64	65
Percent asphalt content	4.8	5.1
Percent air voids content	5.0	4.3
Percent passing No.200 screen	2.8	2.6
Asphalt penetration at 25 deg.	150-200LV	150-200HV
Pavement density (lbs/cu.ft.)	147.1	147.8
(Mg/cu.m)	2.36	2.37
Percent compaction	99	100

Table 3.9 Subgrade properties at the Ste. Anne Road Test*

CBR percent	3
Liquid Limit LL	90-105
Plasticity Index PI	55-65
Dry density (lbs/cu.ft.) (Mg/cu.m)	79.6 1.28
Percent opt. moisture cont.	34.6
Percent constr. moisture cont.	37.6
Field dry density (lbs/cu.ft.) (Mg/cu.m)	81.4 1.3
Percent compaction	102

Table 3.10 Deflections at the Ste.Anne Road Test

Section number	Rebounds 0.001 in. (mm)	Temp. F (C)
64	47 (1.19)	83 (28)
65	47 (1.19)	83 (28)

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CHAPTER 4

FULL-DEPTH ASPHALT CONCRETE PAVEMENTS IN ALBERTA

4.1 Introduction

According to the Alberta Pavement Management System records, there were 2089.28 km of full-depth asphalt concrete pavements in the Province in 1986. From 1968 when, the first full-depth structure in the Province was constructed, until 1982, when Alberta Transportation (now Alberta Transportation and Utilities) discontinued construction of this type of pavement, 2210.29 km (837.70 km of primary and 1372.59 km of secondary highways) were built utilizing the full-depth pavement technology. During this time, 84.77 km of full-depth highways was reconstructed (51.85 km on primary and 32.92 km on secondary highways) utilizing a different pavement structure and 36.24 km changed jurisdiction and ceased to belong to the Alberta Transportation highway network.

By 1982, it was thought that the full-depth pavements displayed, on average, only about half of the expected pavement life (1). At this time, a decision was made to discontinue the construction of full-depth pavements and to investigate possible causes of their failure.

Figure 4.1 shows the cumulative kilometers of full-depth pavements in the Province constructed and rehabilitated up to 1986.

4.2 Alberta's Pavement Performance Prediction Models.

The first step in developing a Pavement Management System in Alberta was to establish a Pavement Information and Needs System (PINS) (2). The work on this system was initiated in 1981 and completed in 1982. The PINS is a set of models by means of which the pavement performance can be predicted. These models include three variables: Riding Comfort Index (RCI), Structural Adequacy Index (SAI) based on the Benkelman beam deflections and Visual Condition Rating (VCR). The Alberta Transportation and Utilities Visual Condition Rating procedure is given in Table 4.1. These three variables were first developed for three different pavement types (granular base, soil-cement base and full-depth asphalt concrete) and for three climatic zones in Alberta (southern, central and northern).

Based on the RCI, SAI, and VCR values, the Pavement Quality Index PQI was developed. This is a single variable which assesses a pavement's overall quality.

The development of the models was based on a data bank developed by the Alberta Research Council. This data base included the results of periodic evaluation of the RCI, Benkelman beam rebounds, VCR, structure type and pavement layer thicknesses, traffic in terms of ESAL's and AADT's, subgrade soils, and rehabilitation measures implemented.

Calculation of the predictive models are explained in detail in Reference (3).

Pavement lives for some full-depth pavements as predicted from Alberta's performance models are shown in Table 4.2.

4.3 Early Experiences with Full-Depth Asphalt Concrete Pavements

4.3.1 Highway 2:48

By the early 1980's, it was observed that some full-depth pavements in Alberta were not performing as well as expected. One such case is described in Reference (4). The considered road is the primary highway 2:48 located in Northern Alberta between Slave Lake and Kinuso. The highway was constructed in stages. The subgrade was constructed between 1962 and 1964, and after gravelling, was used up to 1971 as an improved surface. In 1971, the road was paved with a 200 mm (8 in.) asphalt concrete layer laid directly on the subgrade which, prior to the pavement placement had been reworked to a depth of 150 mm (6 in.). The highway pavement showed structural distress by 1974 and it was overlaid with an additional 125 mm (5 in.) of asphalt concrete. However, it was found after another 7 years, that the pavement was again very seriously damaged. At least 15 percent of the pavement surface was patched or required patching or some other form of rehabilitation. Some alligator cracking was as deep as 325 mm (13 in.). In some cases however, the lower pavement layer was intact while the upper was fatigued.

Up to 1981, when an inspection took place, the traffic on the road was estimated to be in the order of 300,000 ESAL. This investigation revealed that in 1981, thawing at the bottom of the pavement started on April 14 and by May 8 it reached a level 950 mm (38 in.) below the underside of the pavement. Another approximately 800 mm (32.5 in.) of the subgrade was still frozen on that day. It is inferred from the above that this subgrade layer was in a thaw-weakening state for a period of 3 weeks.

A theory was proposed by Millions (1,4) which tried to explain the behavior of full-depth pavements in seasonal frost areas.

> As explained by Millons the theory is based on soil consolidation phenomena. When a footing is constructed on a very low permeable, fine grained soil, the load imposed on the footing is taken first by incompressible water thus the porewater pressure rises. With an increasing load, the water slowly drains from the soil causing the porewater pressure to drop gradually. The more permeable the soil, the faster the rate of porewater pressure dissipation. As the porewater pressure is dissipated the load is gradually transferred to the soil particles. However, this is not the case where full-depth highways are concerned. The duration of a highway loading is negligible compared to that of a footing loading thus the drainage of water, under the short duration vehicle load, does not occur or occurs at a very slow rate. This theory assumes

further that under full-depth pavements, water can be entrapped in a "bath tub" for long period of time, as was shown in the case of highway 2:48. This water might not have any opportunity to drain out anywhere causing a weakening of the subgrade and the entire pavement structure. At this point, there is a great risk of approaching and exceeding the shearing strength of the subgrade soil causing structural distress in the pavement.
The saturation of the subgrade, as described by the theory, can be caused by:
1) formation of ice lenses during the freezing period
2) the effect of hot days and cool nights.
The first factor is generally well known although not fully understood. It is obvious that frost lensing occurs and, upon thawing, the ice lenses are a source of the excess water in the pavement subgrade. The second phenomena was described in a study of temperature and moisture conditions under a pavement (5). It was noticed that, at certain times, moisture tends to percolate up through a pavement structure. This occurs during the warmer months of the year and is most marked during the afternoon of clear, hot summer days.

In view of this phenomenon, an investigation of porewater pressure in a pavement saturated subgrade was considered necessary. To simulate field conditions, measurements of the porewater pressures due to highway loading were planned.

Highway 22:32 was selected for such in-situ investigation.

4.3.2 Highway 22:32

This section contains a summary of the investigation undertaken in 1983/84 on highway 22:32 as reported by Plewes and Millions (1)

Primary highway 22:32 south of Mayerthorpe with full-depth pavement and Secondary road 770:06 (the Genesee Power Plant access road) with a pavement structure consisting of a thick asphalt concrete layer and granular base course were compared during that study.

These two selected highway sections were located in the same general area thus they had very similar climatic and soil environment. The both highways were built in the same year, materials used were similar and the thickness granular equivalencies were the same. Traffic volumes and loads were low. For a period from 1979 to 1983 the estimated ESAL's were 45,000 for the full-depth section and 61,000 for the deep-strength pavement.

In order to prove the theory described in the section 4.3.1 the porewater pressures under the Benkelman beam loads and its dissipation after the load release was measured. However, the installed piezometer readings were inconclusive, thus the major objective of this study could not be fulfilled.

4.4 Performance of Full-Depth Pavements. Some Case Studies in Alberta

In 1978/79 an investigation was conducted at the University of Alberta and the Alberta Research Council dealing with a permanent deformation prediction model for full-depth asphalt concrete pavements (6). The following case highways were examined during this study:

1. Secondary road 794:02
2. Primary highway 15:04
3. Primary highway 16:18

One of the purposes of the Uzan et al. investigation (6) was to assess the possible effects of the asphalt concrete (E_p) and the subgrade (E_s) moduli variations on the rate of rutting. Assuming a linear elastic response to the traffic loads a new model for rutting prediction was developed. An older model for prediction of permanent deformation for full-depth pavements, based on AASHO Road Test data, was earlier reported by Finn et al. (7-9)

Three pavement sections which are subsequently described were investigated using the modified model.

Subgrade moduli were calculated from the Benkelman deflection bowl measurements following the Wiseman et al. procedure (10). It was stated that loading times associated with the Benkelman beam deflection measurements resulted in higher deflection values and lower subgrade moduli than those which could be obtained under moving loads. It was suggested that in order to avoid these discrepancies a factor of 1.5 should be used to adjust the bowl derived moduli to these reflecting real traffic load conditions. This suggested factor was based on experience with moduli derived from Benkelman beam deflection bowl analyses under static and moving wheel loads (11,12).

The described work revealed capability of the modified model to predict rutting in full-depth pavements within reasonable limits. It was found, however, that the model was not responsive to rapid increase of rutting experienced as a result of deformation within the asphalt concrete layer.

The investigated case studies are described as follows.

4.4.1 Highway 794 : 02

The first full-depth project examined was a secondary highway 794 constructed in 1973 which is located northwest of Edmonton and leads to the town of Villenueve. Near the town two large gravel pits are located which serve as a major aggregate supplier to the Edmonton metropolitan area. It was obvious that the road would be loaded differently depending on the direction of travel. The outbound lane which carried heavily loaded trucks was designed as 275 mm (11 in.) thick full-depth asphalt concrete structure and the inbound lane which would accommodate mostly empty trucks was 175 mm (7 in.) thick (12). The subgrade of the road section was a uniform, highly plastic clay, designated as a CH type of soil under the Unified Soil Classification.

The average subgrade moduli were 30 MPa (4400 psi.) from 1973 to 1977 and 24 MPa (3400 psi.) in 1978 (6).

The traffic loading was 600,000 ESAL from 1973 to 1980 when the road was overlaid. After the 100 mm (4 in.) overlay the highway carried another 800,000 ESAL to 1986, but is now scheduled for rehabilitation due to excessive rutting.

4.4.2 Highway 15 : 04

Another full-depth pavement investigated was highway 15:04. This highway section is presently located within the limits of the City of Edmonton. The highway was constructed in 1971-1972 with the subgrade prepared in the late summer of 1971. Two test sections with necessary instrumentation were constructed at km 15.1 (9.44 mi.) and at km 17.6 (11.00 mi.) as measured from the centre of the city.

The subgrade soil of the project was as an inorganic clay of medium plasticity of a CL type. Subgrade moduli were calculated by Dasmohapatra (13) from Benkelman beam deflection basins, with the help of the CHEV5L computer program.

Uzan et al. reported the subgrade moduli to be decreasing with time (6). These moduli were as follows:

1971 - 72	39 MPa (5700 psi)
1973 - 74	26 MPa (3700 psi)
1975 - 77	17 MPa (2400 psi)

The asphalt concrete pavement of this section was designed to be 200mm (8 in.) thick. However, due to construction variations it was 180 mm (7.2 in.) thick at km 15.1 (mile 9.44) and 235 mm (9.3 in.), at km 17.6 (mile 11.00).

The considered highway carried substantial traffic, which in the period from 1972 to 1982 was estimated as 1,300,000 ESALs. In 1981 the PQI of the pavement was 4.77 just above the minimum acceptable level.

This highway section became a

part of the city of Edmonton Street Network when the surrounding area was incorporated into the City. In 1982 this highway section was overlaid with 100 mm (4 in.) of asphalt concrete and has served up to the present (August 1988). A brief inspection of the highway section accomplished in late July, 1988 revealed low temperature transverse cracking is present with an average frequency of 30 cracks per kilometer. There is a sign of fatigue cracking in the wheel paths. These cracks are only slightly visible but they suggest some rehabilitation measures should be undertaken:

4.4.3 Highway 16 :18

A section of highway 16 :18 was the third full-depth pavement investigated. The section is located just east of Edmonton and was constructed in 1969. The pavement structure consists of 300 mm (11.8 in.) of asphalt concrete. The road is part of a four lane highway and is composed of two median separated roadways each 7.4 m (24.5 ft.) wide plus 3.0 m (10 ft.) wide paved shoulders.

The subgrade under the pavement is a low to medium plasticity clay, classified as a CL-CI type. The subgrade moduli derived from the Benkelman beam bowl measurements were relatively constant over the years and equaled 40MPa (5800 psi.) (6).

The number of 80 kN ESAL repetitions from 1969 to the first overlay in 1985 was estimated as 3.4 million. After the overlay with 75 mm (3 in.) of asphalt concrete, the pavement has carried a further amount of approximately 600,000 ESAL.

Before the overlay in 1985 the PQI of the pavement was assessed as 3.00.

A site inspection of the highway was performed in late July of 1988. The inspection revealed that, three years after overlay (150 mm), the pavement was in a very good condition. The quality of ride is still very good, there are no visible signs of distress except transverse cracking of low spacing frequency.

4.5 Reconstruction of Full-Depth Pavements (Secondary Road 507:02)

In 1984 a section of full-depth pavement structure of the Secondary Road 507:02 was reconstructed. The existing pavement was replaced by the conventional structure with granular base.

The original pavement section, 18 km long (km 15.46 to km 32.12) was laid on a fine-grained soil. The soil was classified as a clay soil (CI-CH) with occasional pockets of organic material (14). The liquid limits of the soil were generally in the range of 35 to 60 percent. The field moisture contents were above the optimum content by two to three percent and higher in some sections.

The pavement was 9 m wide and consisted of 125 mm

(7 in.) of asphalt layer (km 15.46 to km 24.78 and km 26.02 to km 32.12) and 150 mm (8 in.) from km 24.78 to km 26.02.

The surface of the original pavement was assessed to be in a fair condition. There was very little cracking and rutting. Patched areas were present but they did not reach more than one percent of the pavement total surface area. It was found that some patches were up to 220 mm deep (14).

Several alternatives of the section reconstruction were considered. The asphalt concrete overlay thicknesses required, according to the RTAC design procedure, was 115 mm (4.5 in.), and 145 mm (5.7 in.) in weaker sections. However, because of an inadequate width and the weakened subgrade condition in some places, reconstruction of the pavement was chosen as the desirable alternative.

4.6 Experience with Low-Temperature Transverse Cracking in Full-Depth Asphalt Concrete Pavements

Palsat investigated reasons of low-temperature transverse cracking which have been developed in asphalt concrete pavements located in Alberta (16). The main objective of this research was to develop a mathematical model for prediction of the low-temperature cracking frequency in asphalt concrete pavements. In this investigation two models of cracking frequency in full-depth pavements were developed. In both these models only full-depth pavements which had not been overlaid, had not exhibited highly varible cracking frequency from kilometer to kilometer, and were constructed between 1970 and 1979, were considered.

It was inferred from the investigation that in order to eliminate or minimize low temperature transverse cracking the thickness of the asphalt concrete pavement should be maximized and the original stiffness at critical ambient air temperature should be minimized ("softest" grade and type of asphalt cement should be utilized).

4.7 Performance of Full-Depth Pavements in Region 3

In 1987 an evaluation of all full-depth asphalt concrete pavements in Region 3 of Alberta Transportation and Utilities Highway Network was performed. The investigation was undertaken by the Alberta Transportation staff using their Pavement Management System. The objective of this evaluation was to examine the condition of full-depth pavements in the Region (17).

There are 456 km highways with full-depth pavements in this Region. Of these 110 km are Primary highways. The present status of these pavements was assessed in terms of the PQI , RCI, SAI AND VCI values. It was found that PQI for all full-depth pavements in the Region were 4.0 and 4.1 for Primary and Secondary highways respectively. These low values were mainly due to a deficiency in the bearing capacity of the pavements. The average SAI values for the considered pavement sections were 2.1 and 2.0 for Primary and Secondary highways respectively. The minimum acceptable level of SAI is 3.0. The average RCI values for Primary highways were 5.37 and 5.67 for Secondary roads. The minimum level of RCI is 5.5. The VCI values were, on the average, 5.76 for the main highways and 6.22 for the local roads. The minimum value of VCI is established as 3.5.

The described evaluation revealed that the actual service life of the newly constructed full-depth pavements in the Region is 6.7 years for the main roads and 6.5 years for the local highways. The expected service life is 7.2 and 6.6 years respectively.

Some of the investigated highways have already received one overlay. The expected life of these overlays is 7.2 and 7.0 years for main and local roads respectively.

The investigation concluded that when a full-depth pavement has no or little structural life remaining, the addition of an overlay is not always able to restore the pavement structural strength.

4.8 Summary

From the above cited material one can draw two quite different conclusions about behaviour and performance of full-depth asphalt concrete pavements in Alberta. Some highway sections like sections 16:18, 15:04, 794:02 have been performing very satisfactorily, despite quite substantial traffic carried by them. Soil conditions of these pavement sections are representative for the overall Alberta soil conditions i.e. the subgrade soils are usually of a clayey type, fine grained, with low permeability and low to medium frost-susceptibility.

There are also some bad examples of full-depth pavement performance as in the case of highway 22:32 south of Mayerthorpe or highway 2:48 west of Slave Lake. These highway pavements exhibited failures very early after construction. The structural distress was substantial and rehabilitation in a form of an overlay was not a solution of the problem.

There are also highways which performance is doubtful, e.g. Secondary Road 507:02. Before reconstruction of this highway in 1984 the full-depth pavement was generally in a fair condition, except for some very short weak sections. However, based on the Benkelman beam deflection measurements the pavement was assessed as very weak and required a very substantial thickness of overlay. The decision was taken to reconstruct the pavement using different than full-depth structure technology.

A very similar situation appears to be in the case of full-depth pavements in Region 3 of the Alberta Transportation Highway Network. Deficiency in the Structural Adequacy Index SAI of the full-depth pavements is the primary cause of low Pavement Quality Index PQI.

The full-depth pavements constitute approximately 15 percent of the Alberta's highway network and because they are approaching their design life, a suitable method of bearing capacity assessment is required and cost-efficient methods of rehabilitation must be found as soon as possible.

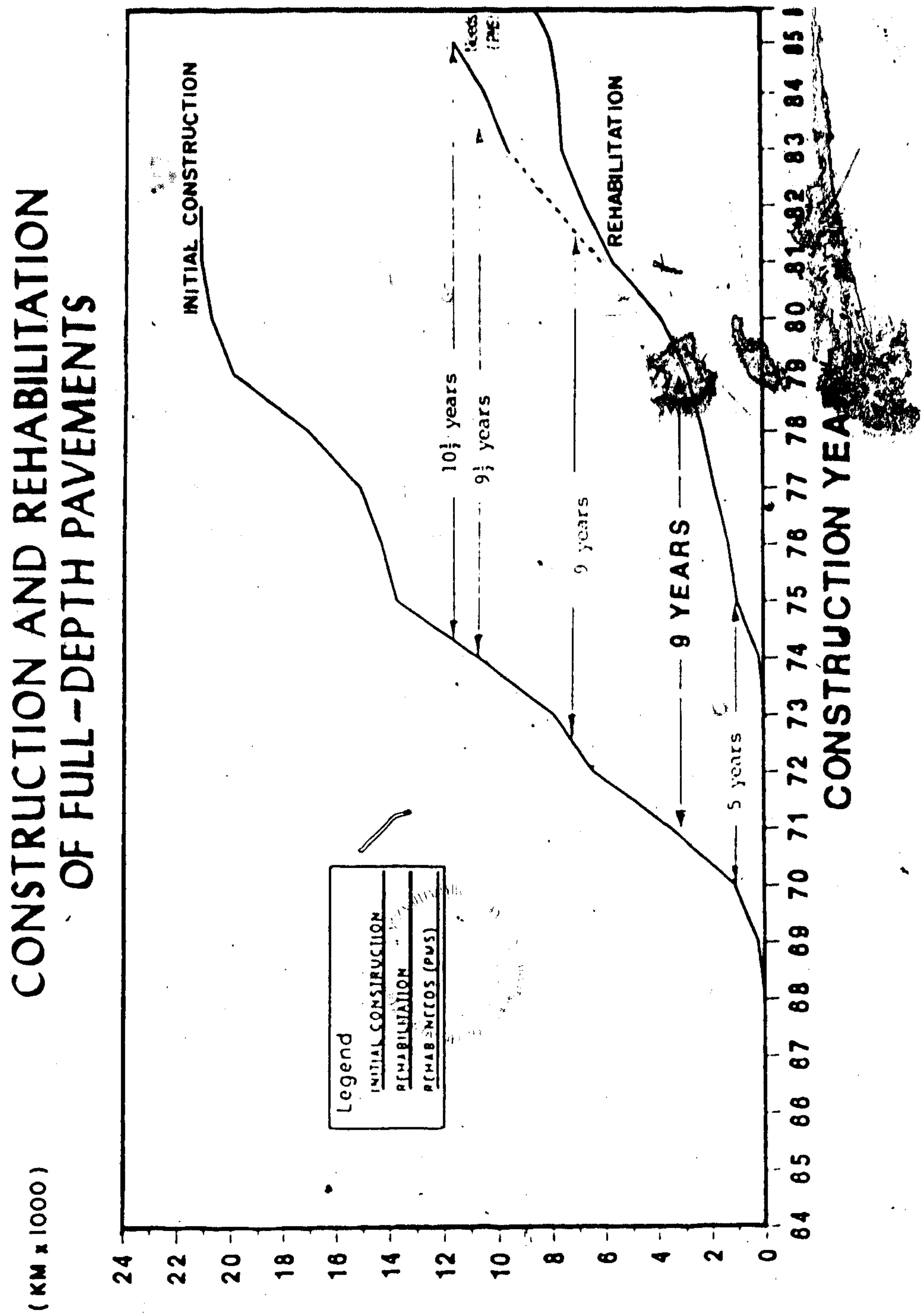

Figure 4.1 Construction and Rehabilitation of Full-Depth Asphalt Concrete Pavements in Alberta.

VISUAL CONDITION RATING

HIGHWAY I.D.				SECTION			MILEAGES		DATE			RATER	RATING
SR AR PR	HWY NO.	ALTR.	C.S.	DIST	[illegible]	DIV	FROM	TO	DAY	MON	YR.		

	VISUAL CONDITION ITEMS	ESTIMATING UNIT	WEIGHTING FACTOR	x RATED NUMBER	= RATING	REMARKS
1.	Stripping and/or Ravelling Seal Coat Deterioration	length	3	40		
2.	Bleeding	length	2			
3.	Shoving	length	3			
4.	Rutting	length	2			depth
5.	Surface Distortion and/or Subgrade Settlement/Heave	area	5			
6.	Chicken Wire Cracks (3")	area	2	45		
7.	Alligator Cracks (6")	area	1			
8.	Map Cracks (12"-24"+)	area	3			
9.	Centerline longitudinal Cracks	length	5			
10.	Random longitudinal Cracks	length	4			
11.	Transverse Cracks	length	4	50		
12.	Pot Holes	area	2			
13.	Skin Patching	area	4			
14.	Deep Patching	area	2			
15.	Edge Breaking Shoulder Cracks	length	4			
16.	Deformation, Bumps & Deterioration at Transverse Cracks	length	3	55		
17.	Roadway Drainage Deficiency Surface and Subsurface	length	3			
18.	Culvert Dips & Rough Bridge Ends	length	4			

TOTAL =

RATING = $\frac{}{2.24}$ = ______

Estimating Unit:
None = 4
Minor = 3
Moderate = 2
Major = 1
Severe = 0

GENERAL REMARKS:

Table 4.1 Alberta Transportation Method of Visual Condition Rating.

TABLE 4.2 Pavement lives as predicted from Alberta's performance models

No.	HIGHWAY	km-km	year of constr.	age of surface	RCI predicted min.: year of min.	RCI predicted min.: age at min.	SAI predicted min.: year of min.	SAI predicted min.: age at min.	VCI predicted min.: year of min.	VCI predicted min.: age at min.	PCI predicted min.: year of min.	PCI predicted min.: age at min.
1.	1:10	6.44 - 13.60	1975	12	1983	8	1989	14	1997	22	1985	10
2.	1A:08	5.10 - 13.35	1973	14	1985	12	1987	14	1999	26	1986	13
3.	2:32	1.32 - 2.51	1979	8	1985	6	1989	10	1996	17	1985	[illegible]
4.	3:08	5.78 - 6.21	1973	14	1991	18	1998	25	2000	27	1994	21
5.	9:06	21.72 - 22.20	1974	13	1984	10	1989	15	1997	23	1984	10
6.	12:08	39.02 - 42.09	1977	10	1984	7	1988	11	1998	21	1986	[illegible]
7.	14X:02	0.00 - 3.22	1975	12	1985	10	1992	17	1995	20	1987	12
8.	16A:20	12.18 - 13.03	1973	14	1987	14	1988	15	1994	21	1987	14
9.	20:02	0.00 - 4.97	1971	16	1984	13	1984	13	1994	23	1984	13
10.	27:10	36.82 - 37.16	1980	7	1990	10	1992	12	1999	19	1990	10
11.	28:02	29.64 - 31.29	1974	13	1984	10	1990	16	1993	19	1985	11
12.	36:02	43.44 - 56.64	1979	8	1994	15	1990	11	2009	30	1991	12
13.	49:04	7.48 - 26.79	1969	18	1984	15	1984	15	1988	19	1984	15
14.	55:10	0.32 - 5.15	1974	13	1984	10	1986	12	1992	18	1984	[illegible]
15.	55:16	24.62 - 36.77	1979	8	1987	8	1987	8	1999	20	1987	[illegible]
16.	60:04	0.00 - 3.22	1975	12	1984	9	1989	14	1994	19	1984	[illegible]
17.	881:10	0.93 - 11.10	1975	12	1990	15	1985	10	2000	25	1986	[illegible]
			Average	12.0		11.2		13.6		21.7		11.4
			St.dev.	3.0		3.3		3.8		3.5		3.3

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CHAPTER 5

MECHANISTIC - EMPIRICAL MODELS FOR DESIGN AND REHABILITATION OF ASPHALT CONCRETE PAVEMENTS

5.1 General

For the two last decades the mechanistic-empirical (analytical-empirical, quasi-analytical) approach of pavement design, evaluation and rehabilitation has been gaining worldwide acceptance.

Pavement structures in the analytical methods are usually modelled as:

- linear elastic
- non-linear elastic
- linear viscoelastic
- non-linear viscoelastic

Other more elaborate models are also available, but the linear elastic model with elements which taking the subgrade and base non-linearity into consideration is probably, because of its relative simplicity, the one most frequently used. In this model a pavement is assumed as a multilayer elastic medium loaded vertically over one or more areas, usually of the circular shape. To solve the problem analytically a number of simplifying assumptions are necessary and are as follows (1):

- material properties in each layer of the multilayer system are identical disregarding the location within the layer

- each layer has a finite thickness except for the subgrade
- horizontal dimensions are infinite
- each layer material is isotropic
- there is full friction between layers at each interface (this assumption is valid only for some models. Other models have been developed which can assume either full friction or frictionless interfaces.)
- surface shearing forces are not present
- the stress solutions are characterized by two material properties for each layer i.e. elastic modulus and Poisson's ratio.

Certain precautions must be exercised when an asphalt concrete pavement structure is analyzed with the use of an elastic model. Firstly, it must be recognized that the modulus of asphalt concrete is a function of temperature and load duration, to mention only the most influential factors. Several different temperature - modulus relationships have been developed as for example the Asphalt Institute equation (2), or the Shell relationship (3). Two such relationships have been developed for materials used in Alberta (4,5).

Secondly, the non-linearity of the subgrade should be recognized. Conventionally, the subgrade can be characterized by a stress softening model in which its resilient modulus is a non-linear function of deviator stress (6). The above mentioned resilient modulus is defined as:

$$M_R = \sigma_d/\epsilon_r \tag{5.1}$$

where σ_d - deviator stress (the difference between a total vertical stress σ_1 and a confining stress σ_3).

ϵ_r - recoverable strain

Uddin et al.(6) has utilized a concept of strain sensitivity. According to this theory the strain softening behaviour is exhibited by granular as well as cohesive materials and can be applied to pavement analysis. The cited work states that the peak shear strain amplitudes generated by the FWD are approximately the same as those under the design load. It is concluded that in-situ moduli derived from the FWD basins are the effective moduli and need no further corrections.

At the beginning of the use of the mechanistic approach for pavement design the main input values i.e. moduli of materials and Poisson's ratios were based on the laboratory testing. Such a procedure has been applied in the Asphalt Institute method (2), FHWA 1975 method (7), and the early Danish method (8). However, a question arises as to how the pavement material elastic moduli obtained in the laboratory correlates with the moduli of real pavement materials. The problem of adequate simulation of real pavement conditions and loadings in the laboratory is always of a great concern.

To avoid the above described difficulty a method for obtaining the moduli directly from in-situ non-destructive testing has been developed. The method employs deflection basin measurements and the theory of elasticity to find the required moduli. The deflection basins can be determined by means of any multisensor deflection device, but as it has already been pointed out in section 2.3.1 the Falling Weight Deflectometer is considered by many as one of best pieces of deflection equipment available at the present time.

Having the measured deflections and utilizing one of many elastic layer computer programs (BISAR, CHEVRON, ELSYM5 are commonly used ones) the required moduli of particular pavement layers can be computed using the procedure of successive approximation. As a first step of the procedure, a set of initial or seed moduli needs to be assumed (6,9,10). The closer the assumed values are to the correct moduli , the faster the convergence is made.

Based on the seed moduli, the pavement deflection can be calculated using the elastic layer computer program. Then the measured and computed deflection basins can be compared. If the two deflection basins do not match then the assumed moduli were erroneously chosen. At this moment the iteration procedure should be applied, subsequently changing the pavement layer moduli, calculating the deflections and comparing them with the measured ones. To avoid very drastic changes the moduli should be modified according to some rule, for example as suggested by Kilareski and Anani (9):

$$E_{i,NEW} = E_{i,OLD}*(D_j/2 + \beta_j/2)/D_j \qquad (5.2)$$

where $E_{i,NEW}$ - new elastic modulus of layer "i",

$E_{i,OLD}$ - seed or old elastic modulus of layer "i"

D_j - deflection measured by sensor "j"

β_j - deflection calculated for distance of sensor "j"

The procedure should start from the subgrade modulus, calculate deflections and compare the measured and calculated deflections. If they do not match a new subgrade modulus should be chosen according to the above equation, whereas the moduli of the other layers should stay unchanged. After a new set of deflections is calculated and compared with the measured ones the modulus of layer overlaying the subgrade has to be modified. The procedure repeats for all the pavement layers and if the convergence is not yet accomplished should be started again with the subgrade modulus. The iterations stop when the chosen convergence criteria are fulfilled. For example the University of Nottingham method (10) employs two convergence criteria: for the moduli and for the deflections.

When the mechanistic procedure of pavement design is used a question arises how the deflection data should be treated. One approach is to backcalculate the elastic moduli for each point where deflections were tested. Such an approach can be very time consuming unless a very fast computer or a program utilizing a very fast algorithm is used. The ELMOD computer program is one example of such an approach (11,12,13). Uddin et al.(6) follow the same concept stating that an "average" deflection basin obtained from all measured basins of a considered pavement section is not a rational method of analysis.

Richter and Irwin (14) agree with the above suggestion, but, ask how the all calculated overlay thicknesses (based on earlier found moduli) for each tested point should be treated. According to them the strict use of any design procedure does not make use of the acquired information on the variability in the pavement and also an overlay designed this way can be unduly influenced by a few excessively weak test points. Taking the above into account they propose the following procedure of overlay thickness selection

1) calculate the required overlay thickness for each test point,
2) determine proportion of the test results which require zero or a very small overlay thickness,

3) - if a large number of test points require a very thin overlay then spot improvements for locations which need overlay should be considered,

- if, however, a small number of test points require a thin overlay still some spot improvements can be considered but the overall length of the considered pavement section should be also overlaid. The 85th percentile overlay thickness can then be determined taking the all non-zero overlay thickness test points excluding points selected for the spot improvements.

The described study which used 250-foot and 50-foot test section spacing reported the need for short intervals between test points. The tested pavement showed a very high variability in the needed overlay thicknesses. It was believed that variations in the moisture content of the upper subgrade soil was the reason for this high variability.

Some agencies, as for example Transport and Road Research Laboratories (TRRL), advise spacing to be in a range of 12 to 25 m (15). Taking into account the use of the FWD, which productivity is very high and as reported approaches 250 points per day (16), short spacings between tested points should not be a problem.

Marchionna et al. (17) propose a different answer as to variability in layer moduli. They assume fatigue cracking is responsible for the asphalt concrete moduli variations and therefore variations in a pavement strength. The moduli of the asphalt layer will degrade from an initial value

(E_s - sound material) to a final value E_c of completely cracked asphalt concrete layer. The value of E_c is not affected by the temperature and it is assumed to be in a range of 1000 MPa (145,000 psi). If the calculation based on the FWD measurement shows that the asphalt modulus E_m is smaller than E_s ($E_s > E_m > E_c$) Marchionna et. al. conclude that some thickness of the asphalt concrete layer is completely cracked. Based on this assumption, and using simple relationships, it is possible to find the thickness of the cracked part of the considered asphalt concrete layer (see Fig.5.1).

At this point it is worthwhile to point out that ARE Inc. in their work (7) as well as Monismith et al. (18) assumed 483 MPa (70,000 psi) for a completely cracked asphalt concrete.

In addition to the two approaches of non-destructive testing evaluation i.e. treating just one representative ("average") set of moduli of a homogenous section or treating the all moduli obtained in each test point, there are also some other possibilities of data treatment.

One such approach based on Kiewit et al. work (19) considers subdivision of a highway link on homogenous sections based on some values derived from the deflection basin measurements. The considered values are:

- SCI - surface curvature index $SCI = d_0 - d_1$ (5.3)

This index indicates the pavement surface properties.

- BCI - base curvature index $BCI = d_5 - d_6$ (5.4)

The BCI indicates the base properties

- SPR - spreadability index

$$SPR = (d_0+d_1+d_2+d_3+d_4+d_5+d_6)/7*d_0 \quad (5.5)$$

where d_0 to d_6 are the FWD deflections

The spreadability index gives information about the E-modulus ratio of the surface and subgrade layers. A large value means that the bounded layers do have a good potential to spread the induced stresses over the subgrade. Figures 5.2 and 5.3 shows the indices for a highway section.

Molenaar et al. (20,21) reported correlation between SCI and the equivalent layer thickness (h_e) and also related the subgrade modulus to the farthest FWD reading. This sensor was located 2 m off the loading centre. The value of SCI was defined there as the difference between the central deflection and the one 0.5 m off the center. The h_e value for a three layer structure is defined as follows:

$$h_e = a*h_1*\sqrt[3]{E_1}/\sqrt[3]{E_2}+a*h_2\sqrt[3]{E_2}/\sqrt[3]{E_3} \quad (5.6)$$

where E_1, E_2, E_3 - moduli for surface, base and subgrade respectively

h_1, h_2 - thickness of surface and base layers

$a = 0.85$ for flexible layers

A very comprehensive analysis on the most beneficial the FWD sensor locations to obtain the best possible data was introduced by Brown et al.(10). In this investigation a number of four layer pavement structures were analyzed over a wide range of varying thicknesses and moduli. The study has revealed that to get quality data, the measuring sensors have to be located at certain distances off the loading point.

5.2 Residual Life and Overlay Design

Having the pavement layer elastic moduli computed (in the case of full-depth pavements the asphalt and the subgrade moduli) and using the elastic-layer theory (by means of the available computer programs: for example DAMA or ELMOD) the design or the expected life of considered pavements can be calculated. The life is usually expressed in terms of the equivalent standard axle loads (ESAL) repetitions to reach the terminal level determined for this type of pavement structure.

As the asphalt concrete modulus decreases with time the number of load repetitions which cause failure also decrease. However, it should be emphasized that the relationship is by no means linear. As pointed out by Claessen et al.(22) and also by Brown et al.(10) the decrease of asphalt concrete moduli can be divided into three different periods. After an initial, quite sudden, but relatively small, decrease of the AC modulus a phase of very slow decrease of the modulus with continued load repetitions occurs. The last phase can be described as a failure state because the AC modulus decreases very rapidly as the terminal life approaches. The above phases are schematically shown in Figure 5.4.

The residual or remaining life of a pavement can be defined as the difference between the expected design life and the life already used in terms of load repetitions. The design life depends on the design criteria used.

At present the most commonly used design criteria are those limiting the tensile strain on the underside of the asphalt-bound layer, and those limiting the vertical compressive strain at the subgrade surface. The latter criterion limits the permanent deformation assuming that the rutting in the asphalt concrete itself is limited by proper design of the asphaltic mixture. Other criterion which makes possible to calculate the permanent deformation in the asphalt concrete has been introduced in the Shell procedure (3). The criterion has the following form:

$$Dh_1 = C_M * h_1 * \sigma_{av}/S_{mix} \qquad (5.7)$$

where
- Dh_1 - reduction in layer thickness
- h_1 - thickness of the considered AC layer
- C_M - correction factor (takes into account the difference between static and dynamic asphalt behaviour)
- σ_{av} - average stress in the pavement under the moving load
- S_{mix} - the AC stiffness

The S_{mix} is a function of the bitumen stiffness. The bitumen stiffness, in turn, depends on the number of load repetitions, time of loading and the bitumen viscosity. Having the maximum allowable rut depth established and knowing the material properties, one may, establish an allowable number of load repetitions.

An overlay, when considered as a remedy for a structural weakness, is applied to alleviate the critical stresses and strains in the existing AC layer. The overlay thickness can be estimated taking into account the following equation (10,22,23):

$$N_d/N_{E2} = 1 - (N_p/N_{E1}) \qquad (5.8)$$

where $1-(N_p/N_{E1}) = RL$ is a pavement remaining life

N_p - traffic loading up to date

N_{E1} - number of load repetitions to failure at the strain level imposed by traffic "p"

N_d - expected traffic after overlay will be placed

N_{E2} - allowable number of load repetitions to failure at the strain level after the overlay will be applied

The above equation is commonly used to estimate remaining life of a pavement and consequently for overlay design.

According to Anderson et al.(23) determination of the overlay thickness using the above relationship should be applied very cautiously. After rearranging of the above equation to the form as follows:

$$N_{E2} = N_d/(1 - (N_p/N_{E1})) \tag{5.9}$$

it is easily recognized that when N_p approaches N_{E1} i.e. the considered pavement approaches its design life, the value of N_{E2} and so the overlay thickness should approach infinity. In his work Anderson (23) neglects the original asphalt layer and designs only to prevent fatigue cracking in the overlay as an alternative concept of overlay design.

5.3 Review of Some Mechanistic-Empirical Models and Backcalculation Techniques

5.3.1 MAPCON

The MAPCON computer system is a composition of many computer programs which analyze pavement safety, serviceability, structural capacity, surface condition and a combination of the latter three. The system has been developed as an interactive set of programs which can be run on a microcomputer. In this section only programs analyzing the structural capacity will be considered.

The purpose of structural capacity analysis is to determine the ability of a pavement to withstand loads (24). Generally two types of data can be used to determine this ability. The data, as it was introduced in Section 5.1, can be obtained by evaluating material properties in the laboratory or by means of measuring pavement deflection bowls using any multisensor device.

MAPCON has six programs for structural analysis of flexible pavements (25,26). Of these, two programs i.e. DYNAFIT and FWDUT1S are for iterative backcalculation of pavement layers moduli based on deflection basins obtained with Dynaflect or the FWD respectively. Two other programs i.e. HCF and FATLIF are designed for calculation of pavement fatigue life. MAPCON is also equipped with ELSYM5 and GENDEF. The latter program is to calculate deflection basin statistics, such as surface curvature index (SCI), base curvature index (BCI), separability (SPR) and also the mean and standard deviation of the sensor readings.

DYNAFIT and FWDUT1S are interactive computer programs which use subroutine LAYER of ELSYM5 to calculate vertical deflections corresponding to the load and sensor locations available for a number of deflection measuring devices, in this case Dynaflect and the FWD. Material properties and/or thicknesses of the pavement layers may be changed by the user until the predicted theoretical basin matches the measured one.

As reported (25) many studies have demonstrated a unique relationship between the fifth Dynaflect sensor reading and the subgrade modulus. This relationship can be used as a method of estimating the subgrade modulus for the first iteration. The first estimate of asphalt concrete layer modulus can be based on a relationship between the modulus and temperature. Other properties of the asphalt concrete used may also be included in the relationship as for example percent of air voids, asphalt content, viscosity of asphalt, load duration, and so on. The Asphalt Institute equation (2) may serve as a good example of such a relationship.

Having the seed moduli of the layers established and knowing the layer thicknesses, the process of iteration may begin. The moduli should be changed using a certain rule, e.g. the one introduced in Section 5.1, to avoid sudden changes of the calculated deflections. The described programs may plot the theoretical basins produced for each calculation trial.

In the case of cracked or fatigued asphalt concrete pavements the modulus can be estimated using the concept developed by ARE Inc. (7). Cracking can be divided into two classes:

Class 2 - cracks form a grid-type pattern

Class 3 - the asphalt concrete segments become loose

The pavement surface that has Class 2 cracking may be assumed as failed in fatigue but because of some aggregate interlock the surface layer is considered to have some load-supporting ability. The Reference (7) proposes a modulus of 483 MPa (70,000 psi) for such a pavement. Class 3 cracks indicate that a considered pavement has failed in fatigue and is approaching a condition of very low serviceability. It has been suggested that a modulus value of 138

MPa (20,000 psi) be assigned in such a case. In both these cases the variation of the asphalt concrete modulus with temperature is not considered.

Monismith et al. (18) have suggested that determination of the asphalt concrete modulus of a pavement section based on the above described concept can be accomplished as follows:

1. A surface with less then 5 percent Class 2 cracking may be treated as intact.
2. If the amount of Class 2 cracking is greater than 5 percent but the amount of Class 3 cracking is less than 5 percent the layer moduli may be estimated to be 483 MPa (70,000 psi).
3. If the amount of Class 3 cracking is greater than 5 percent the modulus of the asphalt pavement should be assigned value of 138 MPa (20,000 psi).

The MAPCON system utilizes two programs for determination of pavement fatigue life i.e. HCF and FATLIF. During development of the programs it was believed that direct use of the elastic layer theory would be prohibitively time consuming and so prohibitively expensive. Because of such a belief instead of using the elastic layer theory a set of regression equations has been developed. Fifteen regression equations have been required for analyzing flexible pavements. Of these, seven equations are required to determine the unknowns in flexible pavements with granular bases and eight for the pavements with stabilized bases. The unknown values are: moduli of surface, base, subbase, and subgrade all under the Dynaflect load, subgrade deviator stress both under the Dynaflect load and the design load (80 kN or FWD load), and tensile strain in asphalt concrete for the design load. Additionally, the tensile strain in a stabilized base under the design load is needed.

To obtain the required relationships four factorials were needed:

1. Dynaflect load, granular bases.
2. Dynaflect load, stabilized bases.
3. 80 kN load, granular bases.
4. 80 kN load, stabilized bases.

Each of the factorials consisted of seven factors ($E_1, E_2, E_3, E_4, D_1, D_2, D_3$) at three levels (low, medium, high) (where E_i are the pavement layer moduli and D_i are the layer thicknesses). The combinations were run using the elastic layer program and data obtained by this means served as a base for a stepwise linear regression analysis to develop predictive models of the variables.

The HCF computer program input can be either as Dynaflect deflections or moduli of pavement layers established in the laboratory. If the Dynaflect deflections are the input, the HCF uses the elastic layer theory regression equation to estimate the moduli. The equations are shown elsewhere (27). Fatigue life of asphalt concrete layer is calculated using the equation of Finn et al.(28):

$$\log N_f = 15.947 - 3.291 * \log(\epsilon/10^{-6}) - 0.854 * \log(E^*/10^3) \quad (5.10)$$

For the asphalt stabilized bases a similar relationship takes the following form:

$$\log N_f = 13.31 - 3.7058 * \log(\epsilon/10^{-6}) - 1.6384 * \log(E^*/10^3) \quad (5.11)$$

where ϵ is the tensile strain at the bottom of the asphalt stabilized layer

E^* is the asphalt concrete modulus

If the fatigue life of the asphalt treated base is shorter than the fatigue life of the surface layer, the procedure assumes that the base behaves as a granular layer after it has reached its fatigue life. Based on the above assumption the total fatigue life of the pavement is then calculated.

FATLIF takes as an input the results of deflection basin matching performed using the DYNAFIT or FWDUT1S programs. The results contain the thicknesses of the pavement layers, the layers Poisson's ratios and the iteratively obtained layers' moduli. The program then uses ELSYM5 to calculate the critical stresses or strains in the considered pavement. The fatigue life equations, the same as described in the case of the HCF program, are employed to find the estimated fatigue life of the pavement.

5.3.2 The University of Nottingham Method

The University of Nottingham procedure reported by Brown et al. (10) emphasizes the use of the FWD as a very effective tool for in-situ determination of pavement elastic parameters. The deflection basin shape induced under a heavy, fast moving vehicle load and also under the FWD dropping weight are directly dependent upon the elastic properties of each pavement layer. The procedure validates that different pavement layers influence different parts of the deflection bowl. The total central deflection at the surface is an integration of the area under a curve describing the vertical strain as a function of depth. The contribution of the subgrade to the area is very substantial. Typical curves for the FWD deflection for a "weak" and a "strong" full-depth asphalt concrete pavements of 215 mm (8.5 in.) thickness on a medium strength subgrade are shown in Figure 5.5 and Figure 5.6 respectively. It is clearly visible that in the case of the full-depth pavements presented in these Figures almost the entire central deflection depends on the subgrade as seen by the relative areas under the vertical strain versus depth plots.

Brown et al.(10) supported the above fact by emphasizing the necessity of careful subgrade modelling. In their model the subgrade stiffness is modelled as follows:

$$E_r = A(p_0/q_r)^B \tag{5.12}$$

where E_r is elastic stiffness of the subgrade

p_0 is a mean normal stress due to weight of the pavement structure above the considered point

q_r is deviator stress induced by wheel load

A,B, are soil constants

In the described model the subgrade appears as a series of layers which stiffnesses increase with depth.

The Nottingham model assumes a similar concept of the asphalt concrete stiffness deterioration with traffic loading to that shown by Claessen et al.(22). This concept may now be validated by means of multisensor deflection devices as for example the FWD or Dynaflect.

The computer program PADAL is used in the Nottingham model for calculation of the pavement layer moduli. The program incorporates a dynamic interactive procedure. Two convergence criteria have been employed in the program; one which limits the calculated stiffness of each layer and the other which limits the error between the measured and calculated deflections.

For a four layer structure, with two layers of asphalt concrete, subbase and subgrade, it was found that the best locations of the geophones to properly determine the layer stiffnesses are:

- for the first layer of asphalt — load center
- for the second layer of asphalt — 200 mm off the center
- for subbase — $d_i/d_0 = 0.8$ which means that the deflection at this position

should amount to 80 percent of the central deflection

- for subgrade 0.6 m and 1.2 m from the geophone specified for subbase respectively

To evaluate pavement structural condition and overlay design two sets of the FWD deflection bowl measurements are required. One set is obtained in the outer wheel paths whereas the second one in between the wheel paths. The latter set relates to the pavement structure in its undamaged state. Then one representative deflection basin for each set of basins for a considered highway link should be selected based on the 85th percentile level. Deflection profiles using deflections d_1, d_1-d_4, and d_7 of the FWD are plotted against the length of the considered highway section to help establishing the representative basins. Examples of such deflection profiles are shown in Figure 5.7. By means of the PADAL computer program the elastic analysis of the selected basins is performed and the obtained moduli are compared with the ones considered to be typical for the pavement materials used. If the asphalt concrete modulus falls below 50 percent of the typical value for this type of material, it is considered that a failure of the asphalt layer has taken place. Further analysis is carried out assuming this layer as a very good granular layer. The modulus assigned to so defined layer should be limited to the modulus of granular layer (200 to 500 MPa or 29,000 to 72,500 psi). If, however, the modulus is still reasonable the remaining life analysis is performed. In order to find the overlay thickness fulfilling future traffic requirements a plot of overlay thickness versus maximum tensile strain at the bottom of the asphalt concrete layer is produced. The calculations to produce the plot and the plot itself are based on three initially chosen overlay thicknesses. After establishing the maximum allowable asphalt strain it is possible to read off the required overlay thickness from the plot by taking the thickness which just satisfies the maximum allowable strain criterion.

5.3.3 Danish Method and ELMOD Computer Program

The Danish method is quite different from the all other methods discussed. This is due to the fact that an "approximate" solution is used rather than an "exact" elastic layer analysis represented by such programs as CHEV5L, ELSYM5, or BISAR. However, the exactness of any method depends whether the assumptions on which the method is based are correct (11). In the case of the above stated elastic layer theory programs the assumptions simplify the real pavement situation exactly in the same way as in the approximate Danish method.

The Danish method is based on a combination of Odemark's solution and Boussinesq's equations (11). The method has been developed because of it simplicity (very fast algorithm) and so the ability to be applied in a very large computer simulations as those required for Pavement Management Systems where the pavement response has to be calculated a large number of times.

The principle of Odemark's method is to transform a system consisting of layers with different moduli into an equivalent system where all the layers have the same modulus, and for which Boussinesq's equations may be used. Figure 5.8 explains the principle of the Odemark's transformations. Generally the Odemark's transformations for a two layer pavement structure are:

- when response is calculated above an interface, the structure is treated as a half-space with modulus value of E_1 i.e. the upper layer
- when response is calculated below the interface, the upper layer moduli will be equal to that of the lower layer but its thickness is changed to obtain the same stiffness of the upper layer.

This new thickness is called the equivalent thickness and for a two layer structure is calculated using the following formula:

$$h_e=f*h_1*(E_1/E_2*(1-u^2)/(1-u_2^2))^{1/3} \qquad (5.13)$$

where

h_e	-	equivalent thickness
E_1, E_2	-	layer 1 and 2 elastic moduli
u_1, u_2	-	layer 1 and 2 Poisson's ratios
h_1	-	thickness of layer 1
f	-	correction factor

The correction factors f are applied to obtain a better agreement with the exact elastic theory. A set of the factors for calculating different pavement responses was developed and is reported elsewhere (11).

According to the Boussinesq's formula, the vertical stress at any depth below the earth's surface due to a point load can be calculated as follows (1):

$$\sigma_z = k*P/z^2 \tag{5.14}$$

and

$$k = 3/(2*3.14*(1+(r/z)^2)^{5/2}) \tag{5.15}$$

where r - a radial distance from the point load

z - depth below earth's surface

Accordingly other stresses, strains and displacements can be calculated from other Boussinesq's equations which can be found in Reference (11). The equivalent thickness is substituted for a "z" value when the pavement responses (stresses, strains and so on) are to be calculated on the layer's interface.

The ELMOD computer program is able to calculate the layers moduli based on the FWD deflection measurements. Practically the program is capable to determine the layer moduli of asphalt concrete, base course, and also the surface modulus and two non-linear subgrade parameters C and n (29). To find the five unknown at least 5 deflections of a deflection basin should be measured, but preferably 7 deflections should be determined.

The surface modulus at the equivalent depth $h_e = r$ where $r>2*a$ r - distance off the load center

a - the radius of the loading plate

can be calculated from (11,29,30):

$$E_0(r)=(\sigma_0*a^2*(1-u^2))/(d(r)*r) \qquad (5.16)$$

where σ_0 is the contact pressure

d(r) is the deflection at distance r off the load

Before calculating the layer moduli the program checks for the subgrade non-linearity by calculating the surface moduli for different distances off the loading center. It is possible to tell from the generated plots whether the subgrade's behaviour is linear or not (11). If the subgrade shows non-linearity, what is usually the case with fine-grained soils, the subgrade modulus can be determined from the approximate relationship (30):

$$E = M_R = C(\sigma_1/\sigma')^n \qquad (5.17)$$

where C and n are constants (n is negative) and σ' is a reference stress equal to 0.1 MPa (145 psi). The subgrade moduli are determined by the program using the outermost deflections in the deflection basin as these deflections are almost completely controlled by the subgrade modulus.

After the subgrade modulus is calculated the moduli of the stiffer layers (surface and base) are determined through an iteration process. The central deflection and the shape of the deflection bowl are considered. The subgrade modulus is then adjusted according to the stress level found, outer deflections are checked and new iteration is carried, if it is still required. Once the moduli of the pavement layer are determined the moduli for each season, using the environmental data, are estimated (11). The program then calculates stresses and strains for each layer interface (31). Miner's law is subsequently applied to determine the overall damage due to the cumulative traffic loading. The required overlay is calculated by ensuring that the critical stresses and strains for the design load are not exceeded in the modelled pavement structure (31).

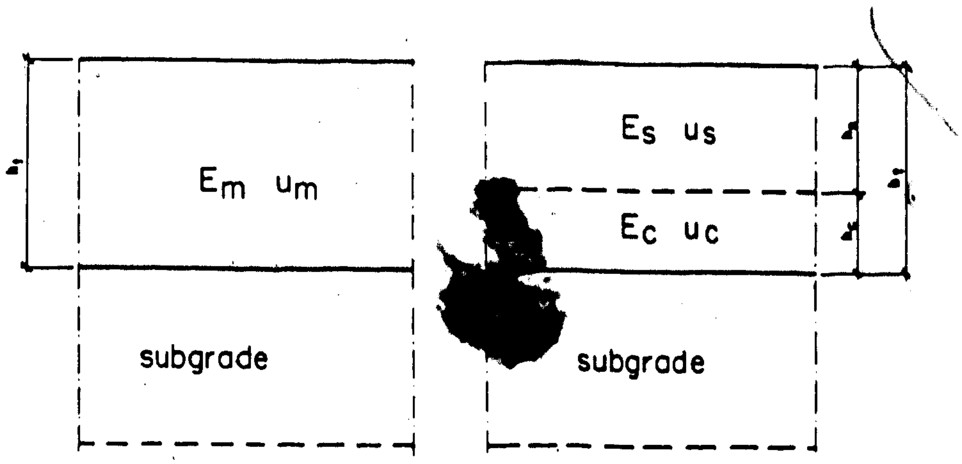
5.4 Summary

This chapter has introduced some mechanistic-empirical models for the design and overlay design of asphalt concrete pavements. Linear elastic theory with elements of non-linearity (assumption of non-linear subgrade and dependance of asphalt concrete moduli on temperature and load duration) is the most commonly employed procedure in analyzing asphalt concrete pavements. The procedure appears particularly applicable to full-depth asphalt concrete pavements.

When pavement bearing capacity data are obtained using a multisensor deflection device, it is the best to treat each deflection basin separately rather than use an "average" deflection bowl or a representative section. The deflection basin shapes should be measured quite frequently along the highway link and the spacings should not exceed 50 meters at most.

The concept of residual pavement life is very commonly applied in various overlay design procedures, however, as pointed out by Anderson et al.(23) this should be used with caution, if not abandoned.

In this chapter three computer program systems dealing with the structural capacity concept have been introduced. Each uses a different approach to pavement structure analysis for determining fatigue life. MAPCON applies regression equations, PADAL, in the University of Nottingham procedure, uses a strict elastic layer theory solution and ELMOD employs an approximate elastic layer theory based on Boussinesq's equation and Odemark's transformations. Overlay thickness estimates using ELMOD can be determined while deflection testing is underway. Ali and Khosla (29) report that the approximate character of the latter program does not harm exactness of the results.



where E_s, E_c, E_m - moduli of sound, cracked and measured material respectively

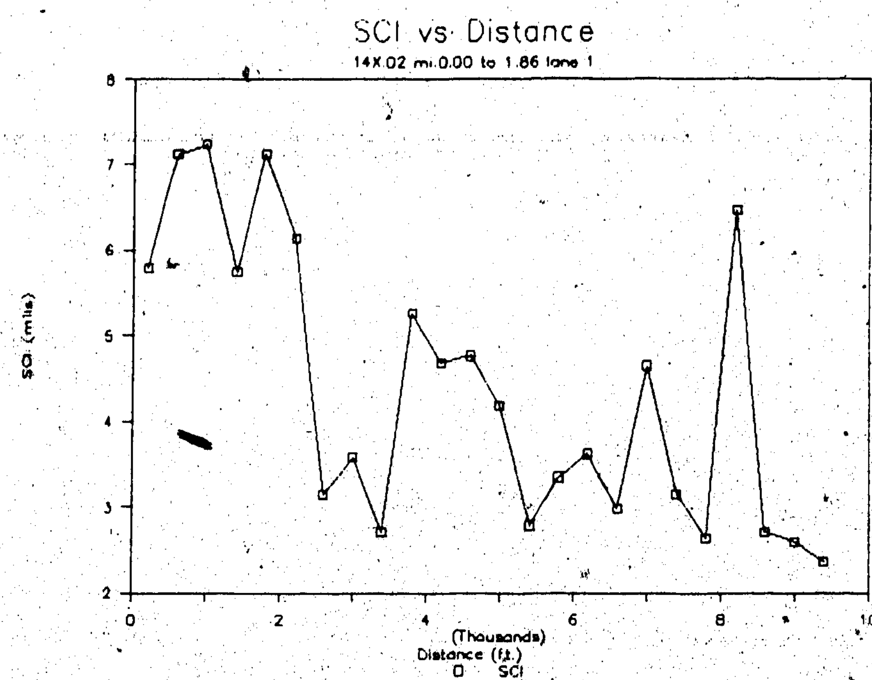
u_s, u_c, u_m - Poisson's ratio for the above materials

If $u_s = u_c = u_m$ - in order to find thickness of the cracked part of the asphalt layer a system of two equations with two unknowns has to be solved as follows:

1) $h_t = h_s*(E_s/E_m)^{1/3} + h_c*(E_c/E_m)^{1/3}$

2) $h_t = h_s + h_c$

Figure 5.1 Equivalence scheme and relationships for a cracked asphalt concrete layer



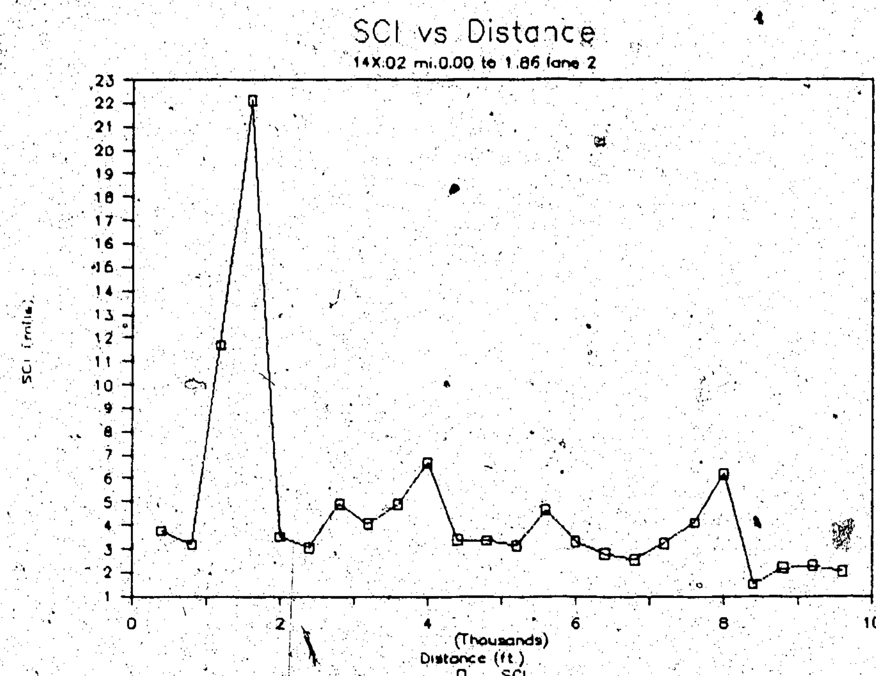
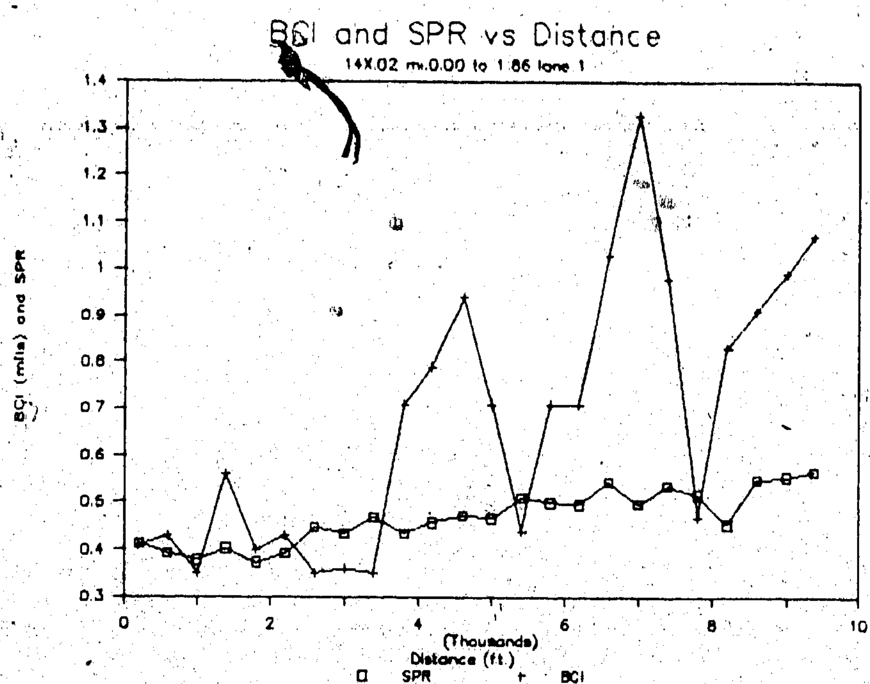


Figure 5.2 Surface curvature index for a section of highway 14X:02



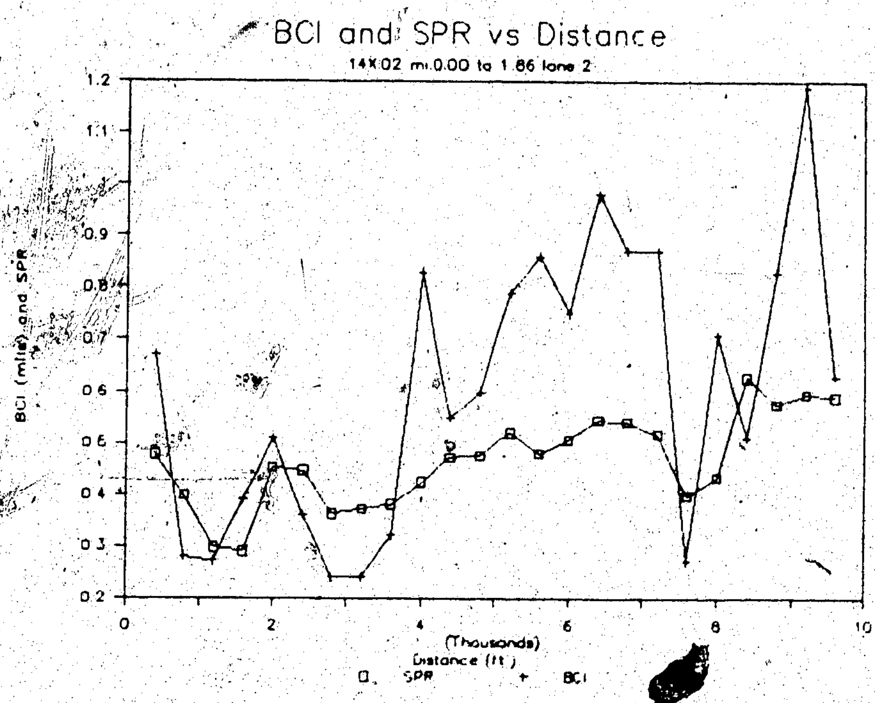


Figure 5.3 Base curvature and separability for a section of highway 14X:02

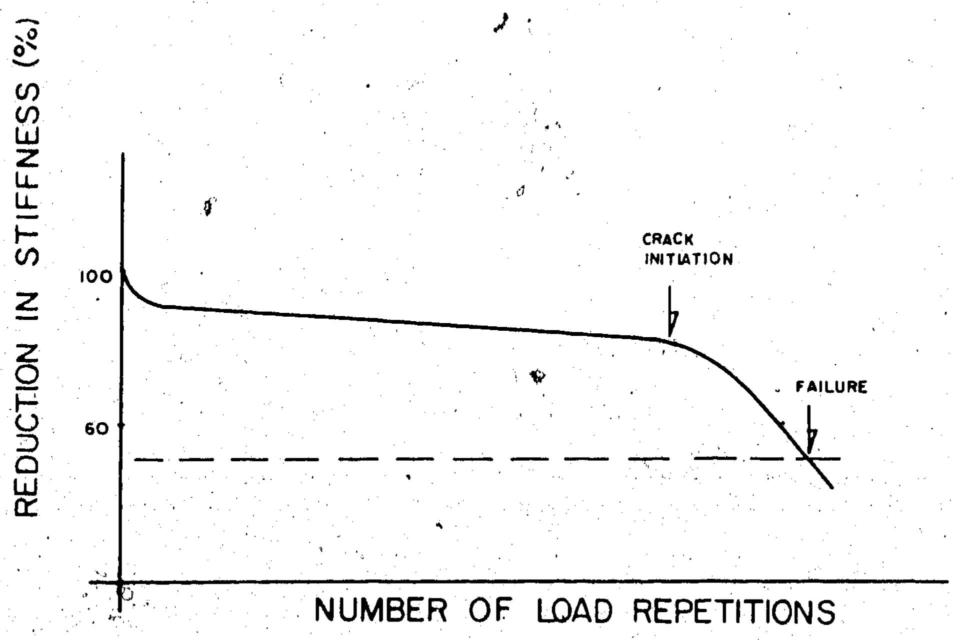


Figure 5.4 Asphalt modulus deterioration with traffic loading

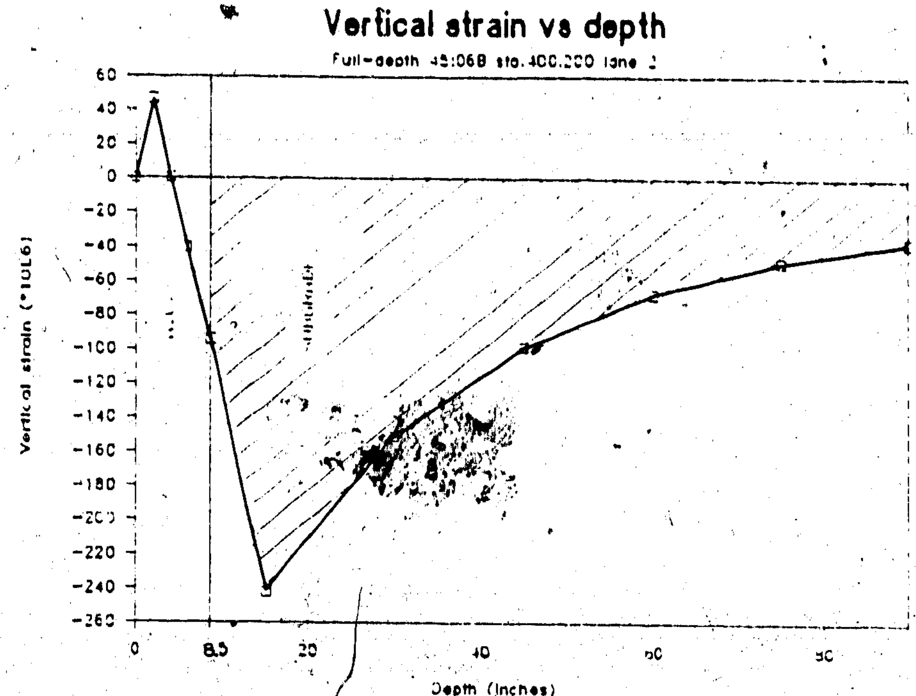


Figure 5.5 Vertical strain in a function of depth for a strong full-depth section (E_A=5188 MPa, E_S=70 MPa, AC layer thickness = 215 mm)

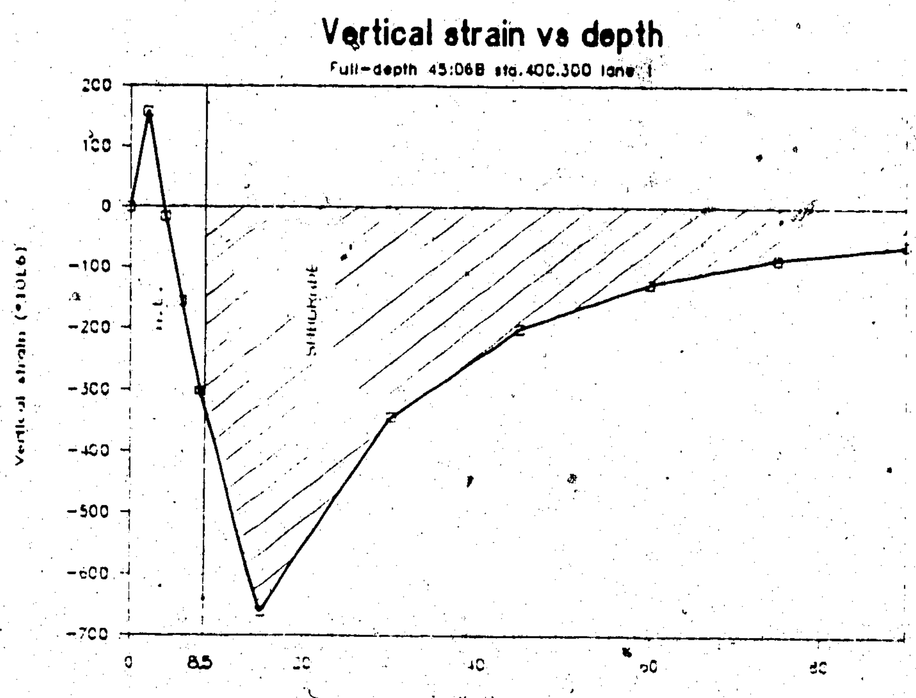
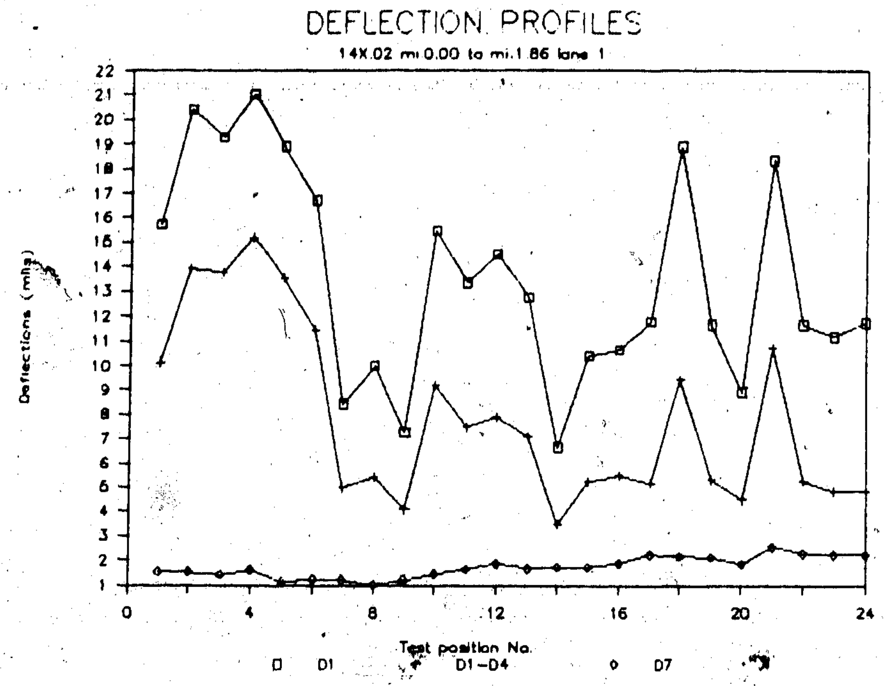


Figure 5.6 Vertical strain in a function of depth for a weak full-depth section (E_A=998 MPa, E_S=50 MPa, AC layer thickness = 215 mm)



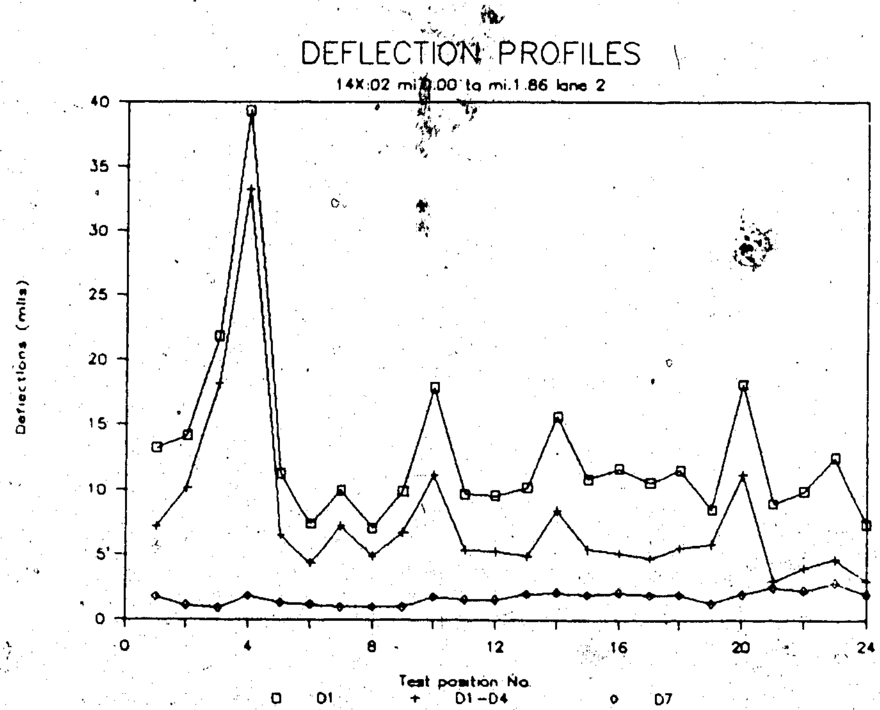
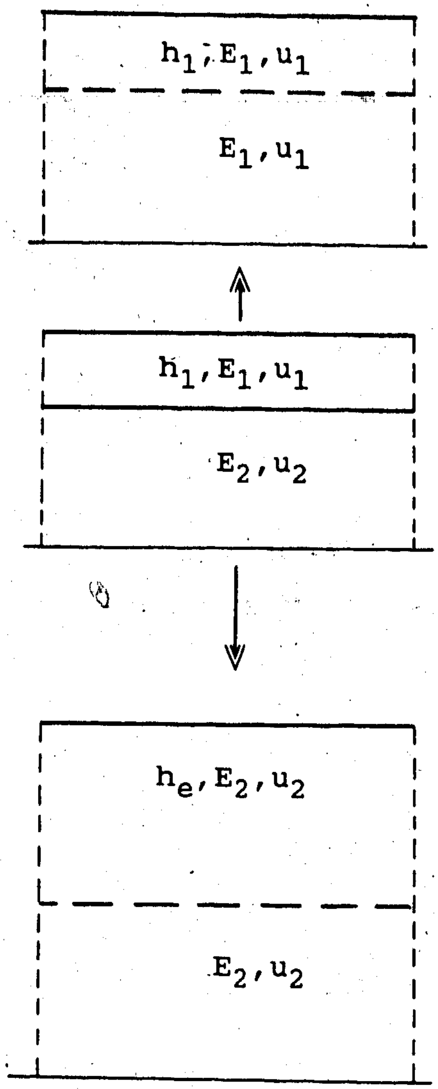


Figure 5.7 FWD deflection profiles for a section of highway 14X:02. The University of Nottingham approach



When calculating stresses and strains above an interface or the compression of the layer over the interface, the system is treated as a half-space with modulus E_1

Two-layer original system

When calculating stresses, strains and deflections at or below the layer above the interface, the system are transformed to an equivalent layer with modulus E_2 and Poisson's ratio u_2, but with the same stiffness as the original layer

Figure 5.8 Odemark's transformations

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CHAPTER 6

APPLICATION OF MECHANISTIC-EMPIRICAL METHODS TO SELECTED ALBERTA FULL-DEPTH PAVEMENTS

6.1 Introduction

This chapter is primarily concerned with the use of mechanistic-empirical methods for structural evaluation of some full-depth pavements selected from the Alberta's highway network. These pavements can be divided into two groups. One group was chosen by the author of the thesis. The selection was made to cover a great variability of the Province's climatic and soil conditions, as well as pavement thicknesses and construction materials. The second group was chosen by the Alberta Transportation and Utilities staff for a testing program run in September of 1987. The program aimed at developing relationships between different non-destructive methods of pavement measurements i.e. the Benkelman beam, Dynaflect and the Falling Weight Deflectometer.

6.2 DAMA Computer Program.

The computer program DAMA was developed at the University of Maryland, and served as a main tool in developing the Asphalt Institute thickness design curves (1). The program uses the elastic layer theory concept for computation of pavement's responses. The layer theory is introduced by a modified version of CHEV5L computer program form. In the case of full-depth pavements the responses considered as critical are: tensile strain at the bottom of the asphalt bound layer and vertical compressive strain at the top of the subgrade. Both asphalt and subgrade layers are characterized by elastic moduli on a monthly basis. In the case of the asphalt layer the moduli can be calculated using the Asphalt Institute regression equation, or can be user selected. The mentioned regression equation relates the asphalt dynamic moduli with:

- percent of aggregate passing No.200 sieve, P_{200}
- frequency, f
- air voids content in compacted mix, V_V
- original viscosity of asphalt cement used, ETA
- asphalt content, V_B
- pavement temperature, t_p

what can be expressed as a function:

$$E^* = f(P_{200}, f, V_V, V_B, ETA, t_p) \tag{6.1}$$

The subgrade moduli can be selected based on the approximate relationships:

$$M_R = 1500*CBR \text{ (psi.) or} \tag{6.2}$$

$$M_R = 10*CBR \text{ (MPa)} \tag{6.3}$$

where CBR is a California Bearing Ratio of the subgrade.

The subgrade is modelled assuming its strength variability throughout the year. The lowest moduli are during the thaw period and the highest in winter time when the subgrade is completely frozen.

Monthly responses of a pavement are calculated using the described moduli and the appropriate thicknesses as an input. Based on the responses and selected distress criteria, monthly damages are computed. The expected design lives, in years or in number of standard axle load repetitions, are obtained.

The program utilizes the following distress criteria:

a) for fatigue

$$N=18.4*(0.004325*\epsilon_t^{(-3.291)}*E^{*(-0.854)}) \qquad (6.4)$$

This equation was developed by Finn et al.(2) and is valid for more than 45 percent fatigue cracking of asphalt concrete in the wheel path area. The above equation can be formulated in a different form. The form is shown below and was utilized in the program:

$$N = f0*10^{M}*f1*\epsilon_t^{(-f2)}*E^{*(-f3)} \qquad (6.5)$$

$$M = f4[V_B/(V_V+V_B)-f5] \qquad (6.6)$$

where N - number of standard axle loads to failure

ϵ_t - tensile strain at the bottom of asphalt layer

E^* - asphalt concrete dynamic modulus (psi.)

V_B - percent of asphalt cement content in mix

V_V - percent of air voids in compacted mix

f0 - shift factor = 18.4

$f1 = 0.4325*10^{-2}$

$f2 = 3.291$

$f3 = 0.854$

$f4 = 4.84$

$f5 = 0.69$

b) for permanent deformation

The assumptions are made in the program that:

- the asphalt concrete does not contribute to permanent rutting if the mix is properly designed and constructed,
- the subgrade is responsible for rutting in full-depth pavements. Rutting can be limited to 13 mm (0.5 in.) by limiting the vertical compressive strain at the subgrade surface.

$$N = 1.365*10^{-9}*\epsilon_c^{(-4.477)} \qquad (6.7)$$

where N - number of ESAL repetitions to reach the above described rut depth.

ϵ_c - vertical compressive strain at the subgrade surface.

At this moment the flexibility of DAMA should be outlined. The program allows for user-defined failure criteria to be used for each distress mode. As mentioned, the monthly asphalt moduli can be defined using the Asphalt

Institute regression equation. They may also be determined based on laboratory testing or non-destructive pavement testing supported by a backcalculation technique.

6.2.1 Preliminary Analysis Using DAMA

In order to determine the performance of full-depth asphalt concrete pavements located in Alberta highway sections have been selected for analysis using the computer program DAMA. To run the program the following information are required as input data:

1. the amount of air voids in the asphalt mix in percent
2. asphalt cement volume in percent
3. percent of aggregate passing No.200 sieve
4. viscosity of the original asphalt at 70 ^{0}F
5. month when pavement was opened to traffic
6. thickness of asphalt layer
7. number of ESAL's repetitions per month

The above information was obtained from Alberta Transportation and Utilities Laboratory using weekly construction reports or pavement inventory data base as sources.

The mean monthly air temperatures, also required as an input, are based on the Environment Canada publication (3). The summary of these data acquired for the selected pavement sections are shown in Tables 6.1 and 6.2 respectively.

The subgrade moduli have been estimated from CBR values according to the formula 6.2. It should be emphasized that the CBR test is not routinely performed in the Alberta Transportation and Utilities laboratory to quantify the subgrade bearing strength. As such, it can only be approximated based on available soil data. Present subgrade quality control procedures are based on achieved density and moisture content. Information regarding density and moisture content, although carefully stored in the weekly construction reports, are of little value when estimates of the subgrade moduli are of concern.

Another way of calculating the subgrade modulus values is by using the Group Index and a correlation chart. To obtain the Group Index, the Atterberg limits and sieve analysis of a considered subgrade soil are required. Usually the soil identification data is acquired at the preliminary stage of highway design, therefore it is not certain what type of soil is actually placed in the subgrade top layers.

All the above considerations support the idea that establishing CBR values from the outlined sources, and then estimation of the subgrade moduli is of a doubtful value. However, because no other method was available, this procedure was utilized for the first set of DAMA analysis.

Khogali (4) has found that the subgrade modulus is the most significant factor in calculation of pavement design life. This fact indicates that the method of using CBR values for subgrade moduli estimation should be applied with great caution.

Highway sections were chosen, that a wide range of full-depth pavements in the Province of Alberta could be analyzed. Consideration included climatic, soil and traffic conditions. The selected highway sections constituted 140 two-lane kilometers, that is about [illegible] percent of the Provincial full-depth pavement network.

The subgrades were approximated by the appropriate estimated CBR values, and seasonal variation of the subgrade strengths were modelled as illustrated in Figure 6.1. It is assumed that this adequately describes the subgrade soil strength and its variations throughout the year. The period of winter freeze is extended to five months. The period of critical spring thaw continues for two months. This latter assumption is based on the findings of Eaton (5). As reported by Eaton the 127 mm (5 in.) full-depth pavement section took 25 days for thawing compared to four days for its equivalent conventional counterpart. A through investigation into the subgrade moduli variations under full-depth pavements in Alberta would be necessary to better describe the subgrade strength variations throughout the year.

In the preliminary DAMA analyses the original Asphalt Institute criteria for fatigue and permanent deformation were utilized. Cumulative damages for fatigue and permanent deformation were computed. They were calculated as ratios of the actual number of ESAL repetitions up to 1987 to an allowable number as determined using the program. For the overlaid or reconstructed pavement sections the cumulative damages were calculated just before rehabilitation. Results indicate very low level of damage and that none of the selected full-depth pavements reached their fatigue life and only some reached or exceeded terminal levels for permanent deformation. Results of these analyses are shown in Table 6.3.

Subsequently, two sets of the DAMA analyses of the selected pavements were executed, but this time with different fatigue criteria (i.e. second Finn criterion for less than 10 percent fatigue cracking and Danish criterion). In both these cases the permanent deformation criterion remained unchanged. Results are different but the general trend i.e. low levels of damages are recognized. Results of these runs are illustrated in Tables 6.4 and 6.5 respectively.

Assuming 40 percent remaining life as a threshold value, one may say, based on the above results, that only very few pavements require some form of structural rehabilitation.

The described preliminary analysis using the DAMA program shows that the subgrade modulus has a very great influence on pavement life. The analyses also show that the Danish fatigue criterion is the most conservative of the three in the case of thin asphalt pavements i.e. for high horizontal strains developed at the bottom of the asphalt layer. When, in turn, thick full-depth pavements were considered (i.e. the strains were relatively low) the criterion was the least conservative.

Taking into account all the above described preliminary analyses it should be recognized that proper structural evaluation of the subgrade as well as proper selection of distress criteria are of a great concern.

It was decided to select two of the three mentioned fatigue criteria i.e. the Danish criterion and the second Finn criterion for further analyses. The second Finn criterion was developed based on the AASHO Road Test (2), and predicts the number of load repetitions to produce less than 10 percent of fatigue cracking in the wheel paths. It is believed that this amount of fatigue cracking would be a threshold value. Beyond this point the structural integrity of any pavement could be drastically reduced. The second Finn fatigue equation has the following mathematical form:

$$\log N = 15.947 - 3.291 * \log(\epsilon_t / 10^{-6}) - 0.854 * \log(E^* / 10^3) \qquad (6.9)$$

where N allowable number of load repetitions

ϵ_t max. horizontal strain at the bottom of AC layer

E^* asphalt concrete dynamic modulus (psi.)

6.3 The Falling Weight Deflectometer testing on Full-Depth Pavements.

Several studies have been carried out over the past few years using the FWD as a main tool for non-destructive pavement testing, however, none of them were concerned with full-depth asphalt pavements. All the studies have indicated the FWD is reliable and advanced measuring device for non-destructive pavement testing. At the present time, however, non-destructive testing is still performed using various different equipment and because of this many highway agencies desire correlation of the FWD with the other measuring devices.

One such attempt was accomplished in Indiana (7). Another similar study was performed by Alberta Transportation and Utilities, which will be described in the following section.

6.3.1 The FWD Testing in Alberta

In the fall of 1987 Alberta Transportation and Utilities investigated correlations between three NDT devices i.e. the Benkelman beam, Dynaflect, and the Falling Weight Deflectometer. The first two are routinely used in the Province, whereas the third piece of equipment was evaluated for possible purchase. The three devices were used in a side-by-side testing on selected highway sections. The structure of the chosen highways reflects the composition of the Alberta's Highway Network. The network is composed of 65 percent of conventional asphalt pavements, 20 percent of asphalt pavements with the soil cement base courses, and 15 percent full-depth asphalt concrete pavements (8). The selection of the test sections was based on previous Benkelman beam and Dynaflect deflection profiles and was aimed at examining pavements which offered a wide range of deflections.

Five sections with full-depth pavements were selected. Table 6.6 provides some important inventory data regarding these sections.

A total of 239 test points were investigated. The points were located every 60 m (200 ft.) along a highway lane. The deflection measurements for the opposite lane were also 60 m (200 ft.) apart, but they were 30 m (100 ft.) offset compared to the first lane. The study showed, as expected, that deflections measured on the full-depth pavements were more temperature affected than for the sections with other structure types. It has been found that the correlation coefficient between the Benkelman beam and Dynaflect was higher that the one between the Benkelman beam and the FWD. Despite the existence of a correlation between the Benkelman beam and the FWD, this relationship should be very cautiously applied, having in mind different measurement concepts utilized by the two devices. Table 6.7 gives pavement deflections of the selected full-depth asphalt concrete pavements as obtained using the three considered deflection measuring devices. In the case of Benkelman beam deflections the mean is based on temperature corrected deflection values.

6.3.2 Moduli Backcalculation Using ELMOD Computer Program.

In March of 1988 analyses of the pavement sections tested in the fall of 1987 were conducted (9). Deflection basin measurements obtained using the FWD were analyzed utilizing the Dynatest ELMOD computer program. The work was performed by John Emery Geotechnical Engineering Limited of Downsview, Ontario retained by Alberta Transportation and Utilities. Based on these measurements, the pavement layer moduli at each test point were calculated and subsequently the required overlay thicknesses computed. The calculated asphalt moduli of the selected full-depth sections ranged between 150 and 10,760 MPa (21,750 to 1,560,200 psi.) with the pavement test temperatures ranging between 8 and 25 ^{0}C. The subgrade moduli were not as dispersed and ranged from 36 to 288 MPa (5,220 to 41,760 psi.). The required overlay thicknesses were between 0 and 239 mm (0 and 9.5 in.). A more detailed description of the analyses and the results may be found in Reference (9).

In the above depicted procedure certain assumptions were necessary in order to estimate the time of expected pavement failure. Failure of the asphalt layers was defined as, the occurrence of the first fatigue crack at the bottom of the asphalt layer under the maximum allowable number of load repetitions. The fatigue cracking formula developed in Denmark has the following form (10):

$$\epsilon_t = 0.00228*N^{-0.178} \tag{6.10}$$

where:

ϵ_t max. strain developed at the bottom of AC layer

N number of standard load repetitions to failure

For unbound materials, including subgrade, vertical compressive strain of the top of the layer is the critical value. The equation predicting the decrease of Present Serviceability Index (PSI) of 2 was used in this analysis and had the following mathematical form:

$$\epsilon_v = 8.34*N^{1.307}*(E/E_0)^{1 \text{ or } 1.16} \tag{6.11}$$

where:

ϵ_v max. vert. strain @ top of the layer

N number of standard load repetitions to failure

E modulus of the considered material

E_0 reference modulus (for subgrade 160MPa or 23,200 psi.)

Note: the power of 1.16 is used when the modulus is lower than the reference modulus, otherwise the power of 1.00 is applied.

In the described investigation each year was divided into 5 climatic seasons. For each season different asphalt moduli were assigned. Seasonal variations of the subgrade moduli were taken into account assuming the spring-thaw type of model (10) with ratio between minimum and maximum moduli of 0.5 (except when the subgrade is frozen).

It is concluded that the mechanistic-empirical overlay design procedure, in the form of the ELMOD computer program utilizing the FWD deflection basin measurements, provides a rapid and very effective tool for assessing pavement structural capacity and formulation of rehabilitation and maintenance decisions.

6.3.3 Moduli Backcalculation Using FWDUT1S Computer Program

This section describes analysis of the previously selected full-depth pavements with the use of FWDUT1S computer program. This program was taken from the MAPCON computer software system, already mentioned in section 5.3.1. FWDUT1S is an interactive program utilizing the ELSYM5's subroutine LAYER to calculate pavement vertical deflections. These deflections correspond with the FWD sensor locations. The following constraints have been adopted in the program (11):

- number of layers 2 to 6
- the semi-infinite bottom layer. The program calculates deflections employing layer theory based on layer properties (moduli, thicknesses) which are supplied by the

user. Subsequently, the computed deflections are compared with the FWD measured deflections. A screen plot is generated which assists the analyst to vary the modulus values until the measured and the calculated deflection basins are in an agreement.

The program assumes the following constants of the FWD device. However, all the values are also user-selected (11):

- load equals the peak force of the FWD
- loading plate radius default values is taken as 300mm (11.8 in.)
- default number of points where deflections are acquired is 6
- default values of sensors locations are 0, 300, 600, 900, 1200, 1500 mm (0, 12, 24, 36, 48, 60 in.)

An example run of the described program is presented in Appendix A.

The FWDUT1S program was used to backcalculate the elastic moduli of the full-depth asphalt pavement sections tested in Alberta in the fall of 1987. The full-depth pavements have been modelled as two layer structures with the asphalt concrete layer on top of the semi-infinite subgrade. The asphalt concrete layer thicknesses were taken as reported by Alberta Transportation and Utilities (9). The moduli of asphalt concrete and subgrade calculated by JEGEL have been utilized as seed moduli and served as an input for the first set of the computer runs. It was found that convergence was, in some cases, easier to obtain than in others, but any general rule regarding this could not been observed. The cases where the convergence could not be obtained during approximately 20 iterations have been excluded from further analysis. The convergence rate for highways 14X:02, 36:18, 45:06A, 45:06B, and 857:04 are 42, 50, 52, 61, and 82 percent respectively. Total percentage of the convergence for all the highway sections tested is 62. The results of the computer analyses as compared with the ELMOD results are summarized in Tables 6.8., 6.9., 6.10., 6.11, and 6.12.

Visual analysis of the results indicates that the FW-DUT1S computer program predicts lower asphalt concrete moduli than the ELMOD computer program. This observation is strongly supported by the statistical comparison performed on the results. The FWDUT1S asphalt concrete moduli are statistically lower than the ELMOD calculated moduli. In the case of highways 14X:02, 45:06A and 45:06B the level of significance is 0.0005, whereas in the case of sections 36:18 and 857:04 the level of significance is 0.025 (12).

The subgrade moduli for sections 14X:02, 45:06A, 45:06B and 857:04 as predicted by FWDUT1S were statistically higher than the ones obtained using ELMOD (0.5 per mill level of significance). For section of highway 36:18 the level of significance was 2.5 percent. When the results of all the selected highways were taken into consideration statistical analysis showed that FWDUT1S predicted statistically lower asphalt concrete moduli than ELMOD at the 0.5 per mill level of significance. However, FWDUT1S predicted higher subgrade moduli than ELMOD, at the same level of significance. All the mentioned statistical calculations are presented in Appendix B.

6.4 Selection of Method for Moduli Backcalculation and Method for Overlay Design. Case Studies Using the Selected Approach.

In this investigation it has been decided to select the ELMOD calculated moduli for further analysis. ELMOD, as reported by Ali and Khosla (13), exhibits a great degree of agreement between the backcalculated and the laboratory obtained moduli. The program takes into account the subgrade non-linearity which, in the case of fine grained soils very commonly encountered in Alberta , may have a very substantial meaning. The ELMOD calculated subgrade moduli were on average 15.3 MPa (2200 psi.) lower than the moduli calculated with the use of FWDUT1S. Overlay thicknesses calculated with the ELMOD provided moduli would then be more conservative than those calculated using the FWDUT1S moduli. Another reason for such a selection was the lack of convergence in many test points when FWDUT1S was employed. The findings of the JEGEL (9) study was also taken into account. This work stated that the majority of the ELMOD calculated moduli were within the expected range for pavement materials, and environmental conditions encountered near the test sites.

The Asphalt Institute DAMA computer program was chosen as a main tool to calculate expected pavement lives, and the required overlay thicknesses. For closer investigation ten test points for each of the five selected full-depth pavements have been chosen. Two investigations using the program have been performed, each with different fatigue criterion incorporated.

The following full-depth pavement sections were selected:

14X:02	km 1.80	to	km 2.90	lane 1
36:18	km 1.04	to	km 1.60	
45:06A	km 13.43	to	km 13.98	
45:06B	km 23.27	to	km 23.82	
857:04	km 12.25	to	km 12.80	

In the first set of the analyses the second Finn fatigue criterion was applied. The seasonal variations in subgrade moduli were modelled according to the procedure explained earlier in Section 6.2.1. The in-situ asphalt concrete moduli were adjusted to a design temperatures, according to the Asphalt Institute relationship for temperature sensitivity (1). Twelve asphalt concrete moduli in the temperature ranges occurring in a particular location are required as DAMA input. However, only one backcalculated asphalt concrete modulus was available at the test temperature. To obtain all the needed asphalt concrete moduli, the temperature correction concept was applied, according to the procedure reported by Uddin (14). A temperature corrected modulus ($E_1{'}$) is a product of in-situ backcalculated modulus (E_1) and the ratio of the moduli at the design and test temperatures calculated from the laboratory derived modulus versus temperature relationship.

For example, when the backcalculated modulus was 2789 MPa (highway 14X:02 sta. 100.060), and the measurement was taken at 16^{0}C, the modulus at this temperature as derived from the laboratory procedure was necessary. In our case the modulus was calculated according to The Asphalt Institute regression equation (15). For the highway 14X:02 mix parameters and 16 ^{0}C, the modulus was found equal to 4064 MPa. The ratio was then 2789/4064 = 0.69. Subsequently, the moduli calculated using the regression equation for the entire required range of input temperatures were multiplied by the above ratio.

This concept, although straightforward, should be cautiously applied. It was found in this investigation, especially in the case of highway 36:18, that the asphalt concrete moduli calculated using the described approach had excessively high values. Large variations in the calculated asphalt concrete moduli may be due in part to variability in the actual thickness of the asphalt concrete layer from that assumed in the applied backcalculation techniques. To avoid the above discrepancies coring is recommended to more adequately define the layer thicknesses (8).

For this investigation the design period was chosen as

15 years and traffic was calculated assuming a 4.5 percent yearly increase for highway 14X:02 and a 4 percent for the other highway sections. The traffic calculations for the considered pavement sections are shown in Appendix C. Four iterations of DAMA were run to find the required overlay thickness for each considered test point. Initially, two-layer pavement structures were considered with the ELMOD calculated moduli incorporated. When the predicted allowable number of standard load repetitions for the design period was greater than the expected number, no overlay was required. If, however, this was not the case the pavement strengthening was necessary. In such a situation, three subsequent computer runs were performed, using three-layer pavement structures. In such structures the top layer consisted of an asphalt concrete overlay with the moduli calculated according to the Asphalt Institute regression equation. Mix properties chosen for the overlay layer were: V_B=13.10 percent, V_V=7.0 percent, ETA=0.15, P_{200}=6.0 percent. The lower asphalt concrete layer and the subgrade moduli had values as backcalculated using ELMOD. The subgrade moduli of the selected sections are reported in Tables 6.13, 6.14, 6.15, 6.16, and 6.17. The corrected asphalt moduli of the existing asphalt concrete layers are shown in Tables 6.18, 6.19, 6.20, 6.21, and 6.22.

The three runs of DAMA were performed using different overlay thicknesses i.e. 25, 75, 125 mm (1, 3, 5 in.).

having the allowable number of 80 kN load applications calculated for pavement structures with different overlay thicknesses and the corresponding overlay thicknesses, plots of the allowable load repetitions versus overlay thickness were developed. These plots are similar to those reported by Finn and Monismith in Reference (16). Based on the plots, appropriate overlay thicknesses were estimated. Results of the above described procedure are shown in Table 6.23. An example run using the described methodology for one tested point is shown in Appendix D.

An identical procedure using the Danish fatigue criterion incorporated into the DAMA program, was also investigated. The Danish fatigue relationship was incorporated into DAMA in such a way that only the basic fatigue equation was changed. The laboratory-to-mix performance adjustment factor as well as the mix adjustment factor remained unchanged. The analyses using DAMA with such a fatigue equation incorporated were performed, but this time four different overlay thicknesses i.e. 25, 75, 125, and 175mm (1,3,5,7 in.) were considered. Results of these analyses are shown in Table 6.24.

6.5 Site Inspection of Selected Full-Depth Pavements

Prior to the DAMA computer analyses a site inspection of the considered pavements was undertaken in order to obtain current observations of the pavements conditions.

Some pictures of the pavements are presented in Appendix E.

6.5.1 Highway 14X:02

This highway is located just east of the City of Edmonton and consists of a 4-lane two-way facility divided by a median of a varied width. The facility was constructed in 1975 with 300 mm (11.8 in.) full-depth structure placed on a CL type subgrade. The average mix parameters obtained from the weekly construction reports were as follows: V_V=6.5 percent, V_B=11.6 percent, P_{200}=4.4 percent and ETA=0.133 (poises * 10^6).

The highway has carried a very heavy traffic. Traffic from 1975 to 1985 constituted of above 1.5 million standard load repetitions.

Some forms of distress are visible on the pavement. Low temperature transverse cracking is present and in some sections spacings are in the order of 5 m. A few patches are also present. Permanent deformation in the wheel paths is slight, but longitudinal cracks in the paths have developed. These cracks are very slight, yet they indicate that the pavement has fatigued to some extent. Some of these cracks are sealed, but others recently developed have not been treated. It would appear in view of the traffic the highway has carried and the described forms of distress the pavement should be rehabilitated. A possible rehabilitation measure would consist of partial cold-milling and a substantial overlay using recycled or virgin mix.

6.5.2 Highway 36:18

This highway section is located about 120 km east of Edmonton. The considered section begins in the town of Viking and goes northwards. The highway carries medium volumes of traffic. In a period between 1974, when the facility was constructed, to 1986, the cumulative number of standard load repetitions was above 180,000. The section consists of 115 mm (4.5 in.) of asphalt layer placed on a loamy subgrade. From weekly construction reports the average mix properties of the original asphalt mix were: V_V=5.4 percent, V_B=12.3 percent, P_{200}=4.66 percent and ETA=0.176 (poises * 10^6). The amount of air voids in the mix indicates that good compaction of the asphalt concrete layer was achieved. The pavement has very narrow paved shoulders. The highway provides relatively rough riding quality, but little rutting is present. There are many patched areas indicating local loss of pavement bearing capacity. In the testing during the fall of 1987 there was little correlation between the non-destructive device measurements (7). This fact excluded this pavement from that analysis. Before any rehabilitation measure is taken, it would be valuable to test the pavement non-destructively again. The retrieved pavement cores would reveal actual pavement thicknesses, mix properties, together with the asphalt concrete and the subgrade structural conditions.

6.5.3 Highway 45:06A

This highway section is located approximately 100 km north-east of Edmonton and 12 km west of the town of Andrew. The section was constructed in 1971 as a 150 mm (6 in.) thick structure laid on a CL type subgrade. It has carried medium traffic that in a period from 1971 to 1985 consisted of 200,000 cumulative standard 80 kN load applications. The properties as obtained from weekly construction reports were: V_V=9.56 percent, V_B=13.59 percent, P_{200}=6.33 percent and ETA=0.1[illegible]4 (poises * 10^6). The section surface has been resurfaced with the use of a chip seal. The ride quality over the pavement is fair but several forms of distress are visible. The most common type of failure encountered are numerous patches indicating local loss of bearing capacity. Also low temperature transverse cracking is present at approximately 10 m spacing. An overall assessment of the pavement visual condition indicates that substantial rehabilitation measures should be undertaken.

6.5.4 Highway 45:06B

This adjacent highway section was originally constructed in 1970 with the asphalt pavement 150 mm (6 in.) thick placed in 1971. The subgrade is characterized as a CL-CI type of soil. The two investigated sections of highway 45 i.e. 45:06A and 45:06B were constructed under the same contract, by the same contractor and at the same time so one might expect the same workmanship and, similar asphalt mix properties. In 1978, 7 years after the construction the pavement section 45:06B was overlaid with 65 mm (2.5 in.) of asphalt concrete with the following mix prope[illegible]: V_V=7.3 percent, V_B=14.06 percent, P_{200}=4.2 pe[illegible] and ETA=0.352 (poises * 10^6). Prior to the ov[illegible] two sections i.e. 45:06A and [illegible] were evaluated as follows:

H[illegible]ay	45:06A	45:06B
RCI	7.2	7.0
VCR	76	68
Avg.Benkelman defl.	0.069 in.	0.079 in.
Max.Benkelman defl.	0.104 in.	0.148 in.

The decision to overlay section 45:06B was most probably due to somewhat poorer conditions as shown in the table above.

At present the visual condition of the section is satisfactory. The low-temperature transverse cracks average 10 m apart. There are some patches and areas with loss of the seal coat aggregate, but there are no signs of rutting and other types of structural distress. Generally the section provides a relatively good ride quality.

6.5.5 Highway #857:04

This secondary road is located north of the town of Vegreville and approximately 120 km east of Edmonton. The pavement was constructed in 1979 as 180 mm (7 in.) thick structure. The subgrade type soil is not indicated in the Alberta Transportation inventory but probably is of a CL type. The estimated cumulative traffic between 1979 and 1986 is in the range of 45,000 standard 80 kN load repetitions. The section visual appearance may be assessed as excellent. The ride over the pavement is generally good to very good. There are no sign of structural or environmental type of distress. There is only one short section (about 100 m long) which has been repaired and is indicative of subgrade failure due to frost action. The rest of the highway will likely need only normal maintenance in the future.

6.6 Overlay Design Based on the RTAC Benkelman Beam Test Procedure

The Benkelman beam design procedure was applied to the case sections to compare the results of the mechanistic-empirical design procedures with the procedure commonly applied in Alberta. The Benkelman beam procedure recommends at least 10 measurements to be taken in the spring for a specific pavement section. If deflections are acquired at another time they have to be converted to the maximum spring values by a suitable factor (15,17). In the case when no data regarding spring to fall subgrade strength variation is available a factor of 2.5 may be applied, having in mind, however, that so obtained spring deflection values are rather crude approximations of the real conditions. For this analysis a factor of 2.0 was applied. The pavement temperatures should be measured at the time of deflection measurement to permit correction of the deflections to the standard temperature of 21 ^{0}C.

Based on the subgrade type, measured deflections, pavement conditions, traffic, and existing structural thickness of the pavement, homogenous pavement section selection is made. The design deflection of the selected homogenous section is calculated as follows:

$$x_D = \bar{x} + 2 * \sigma \qquad (6.12)$$

where:

- x_D design deflection
- $\bar{x}$ mean of ten deflection measurements
- σ standard deviation of the deflection sample

Overlay design is based on two empirically developed charts (16). In the first chart (see Figure 6.2) criteria for maximum Benkelman beam rebound versus cumulative axle load repetitions are shown. For future traffic estimated, a maximum allowable spring rebound value may be obtained. The second chart (Figure 6.3) gives the additional thickness of granular base required to reduce the measured rebound value to the allowable one. The thickness is subsequently converted to asphalt layer thickness by applying locally used equivalency factors, which normally range from 2.0 to 2.5.

The above described method has been applied for calculation of overlay thicknesses of the case pavements, using an equivalent factor of 2.25. The calculations are shown in Appendix F. The calculated design deflections as defined by equation 6.12 are presented in Table 6.25. The calculated overlay thicknesses using the Benkelman beam deflection approach are shown below:

Hwy 14X:02	250 mm	(10.2 in.)
Hwy 36:18	110 mm	(5.[illegible] in.)
Hwy 45:06A	95 mm	(4.[illegible] in.)
Hwy 45:06B	130 mm	(5.3 in.)
Hwy 857:04	100 mm	(4.3 in.)

6.7 Comparison of Results

At this point a comparison of the results of different overlay thickness design procedures is presented. Four procedures have been used and are as follows:

1) Dynatest ELMOD
2) DAMA with the Danish fatigue criterion incorporated
3) DAMA with the second Finn criterion incorporated
4) RTAC Benkelman beam based procedure

In the case of the mechanistic-empirical procedures i.e. points 1, 2 and 3 calculations of the design overlay thicknesses was based on the mean plus one standard deviation value of the calculated overlay thicknesses from each considered test station. Such a procedure is suggested in Reference (8) as a minimum for the Alberta conditions.

The fourth of the presented procedures i.e. the Benkelman beam deflection method, is of empirical character and the overlay thicknesses obtained are based on design deflections calculated according to equation 6.12 and charts illustrated in Figures 6.2 and 6.3.

The overlay thicknesses obtained using the above procedures are presented in Table 6.26 and illustrated graphically in Figure 6.4. In this Figure DAMA1 means that the program DAMA incorporated the Danish fatigue criterion, whereas in DAMA2 the second fatigue criterion developed by Finn et al. was utilized (2). It is seen from this Figure that in the case of highways 36:18, 45:06A and 45:06B the ELMOD based procedure predicts overlays comparable with these obtained using the Benkelman beam method. In the case of highway 14X:02 i.e. the section with a very thick asphalt concrete layer, and highway 857:04 i.e. the section with no signs of structural distress, the ELMOD estimated overlay thicknesses are lower than the ones obtained using the Benkelman beam procedure.

The DAMA procedure with the Danish fatigue criterion incorporated, generally predicts (section of highway 36:18 is an exception) greater overlay thicknesses compared with the ELMOD method. The overlay thicknesses predicted using the mentioned DAMA procedure are 35, 61, 40 and 86 percent greater than these predicted using ELMOD for highway sections 14X:02, 45:06A, 45:06B and 857:04 respectively. In the case of highway 36:18 the procedure predicts thickness 25 percent less than ELMOD.

The second DAMA based procedure (i.e. with the second Finn fatigue criterion incorporated) gives the least overlay thicknesses. This procedure predicts thicknesses 63, 72, 56 and 63 percent lower when compared with ELMOD for highway sections 14X:02, 36:18, 45:06A and 45:06B respectively. In the case of highway 857:04 the described procedure predicts no overlay necessary for the design period compared to 22 mm (~1 in.) of AC overlay calculated using ELMOD.

6.8 Summary

This Chapter contains a discussion regarding methods of structural evaluation of some full-depth pavement sections in Alberta. Two groups of pavements were investigated i.e. those selected by the author for a preliminary analysis, and those selected by the Alberta Transportation and Utilities personnel which were investigated in more detail. The preliminary analyses using the DAMA computer program indicated that the subgrade moduli have a very great influence on predicted lives of the pavement. It was also shown that the asphalt concrete fatigue criteria chosen have a great influence on the overlay thickness requirements.

Two moduli backcalculation procedures were described. One utilizes the Dynaflect ELMOD computer program while the other FWDUT1S computer program included in the MAPCON computer software system. The results of the two procedures were statistically compared. It was found that ELMOD predicts higher asphalt concrete moduli and lower subgrade moduli than moduli predicted using FWDUT1S.

It was also found that the two programs sometimes predict unacceptably high asphalt concrete moduli. This fact is most probably due to variations in pavement thickness, as well as temperature influence.

Finally a method of mechanistic-empirical overlay design was described. The method is based on the DAMA computer program. Two sets of fatigue criteria were incorporated into the program. One criterion incorporated was the Danish fatigue criterion whereas the second one, based on the AASHO Road Test was developed by Finn et. al. Consequently overlays calculated using each approach were compared with the results obtained using the ELMOD based procedure and the RTAC Benkelman beam deflection based procedure. The comparison shows quite a large variation in results obtained. It seems that the ELMOD procedure is comparable with the RTAC empirical procedure in the case of full-depth pavements with a considerable structural distress, however for pavements with very thick asphalt concrete layers the empirical deflection method predicts greater thicknesses.

This was also the case with a pavement showing no structural distress at all.

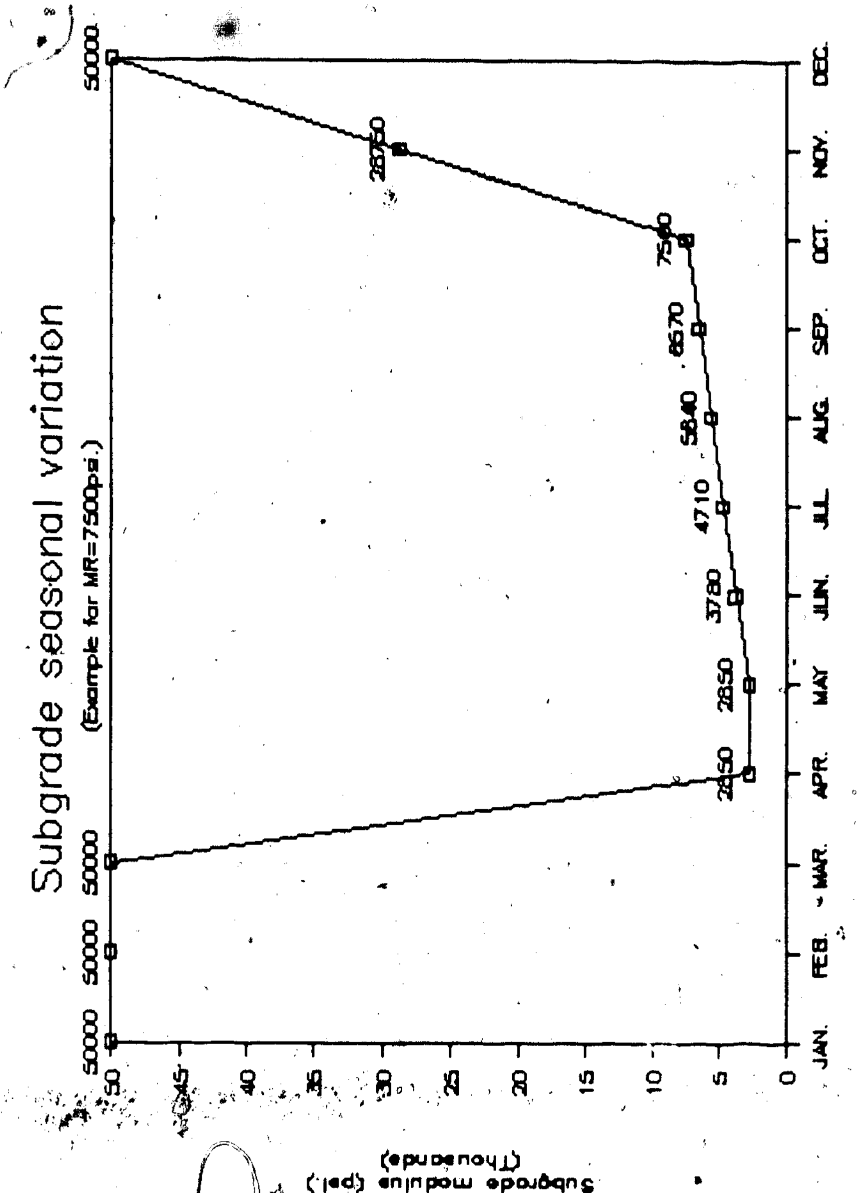

Figure 6.1 Subgrade seasonal variation concept applied in DAMA overlay procedures

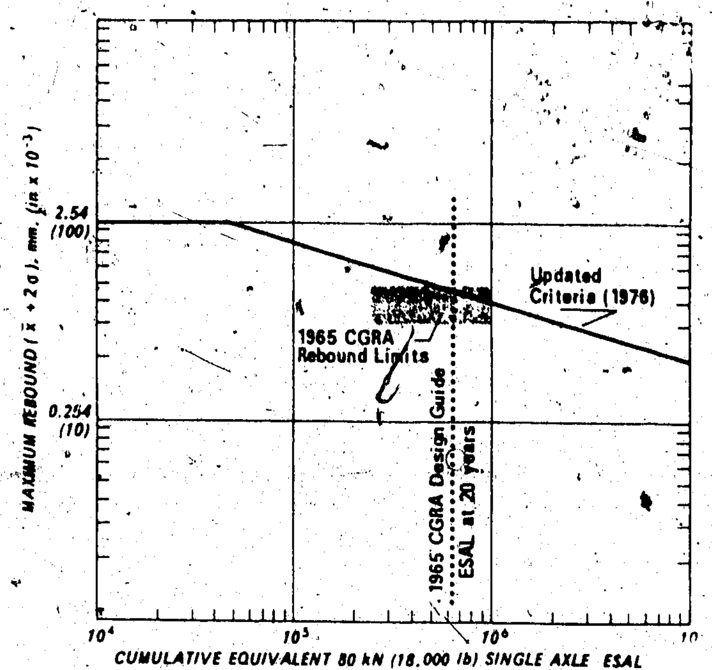

Figure 6.2 Recommended criteria for Benkelman beam rebound versus cumulative axle loads

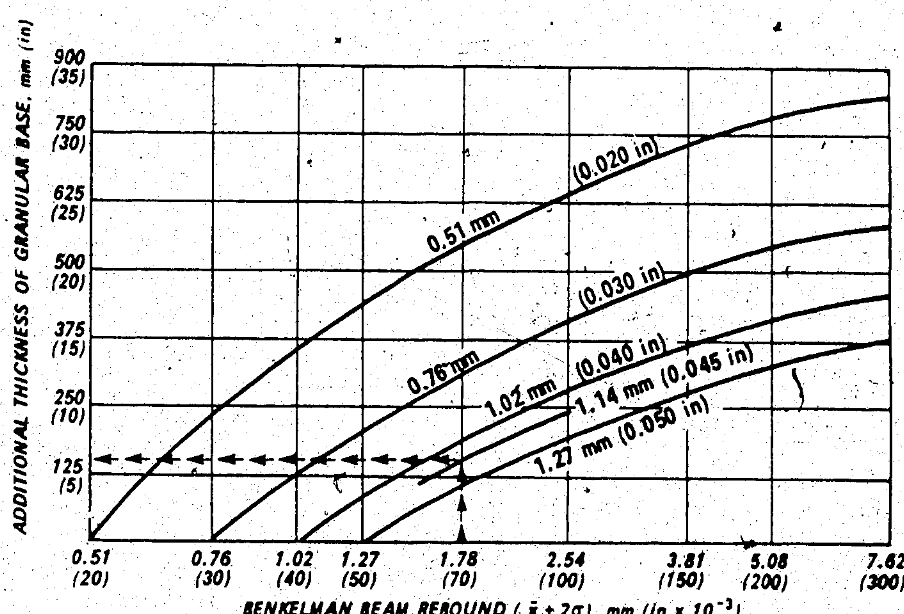

Figure 6.3 Additional thickness of granular base required to reduce a Benkelman beam rebound on an existing surface to a designated or design rebound

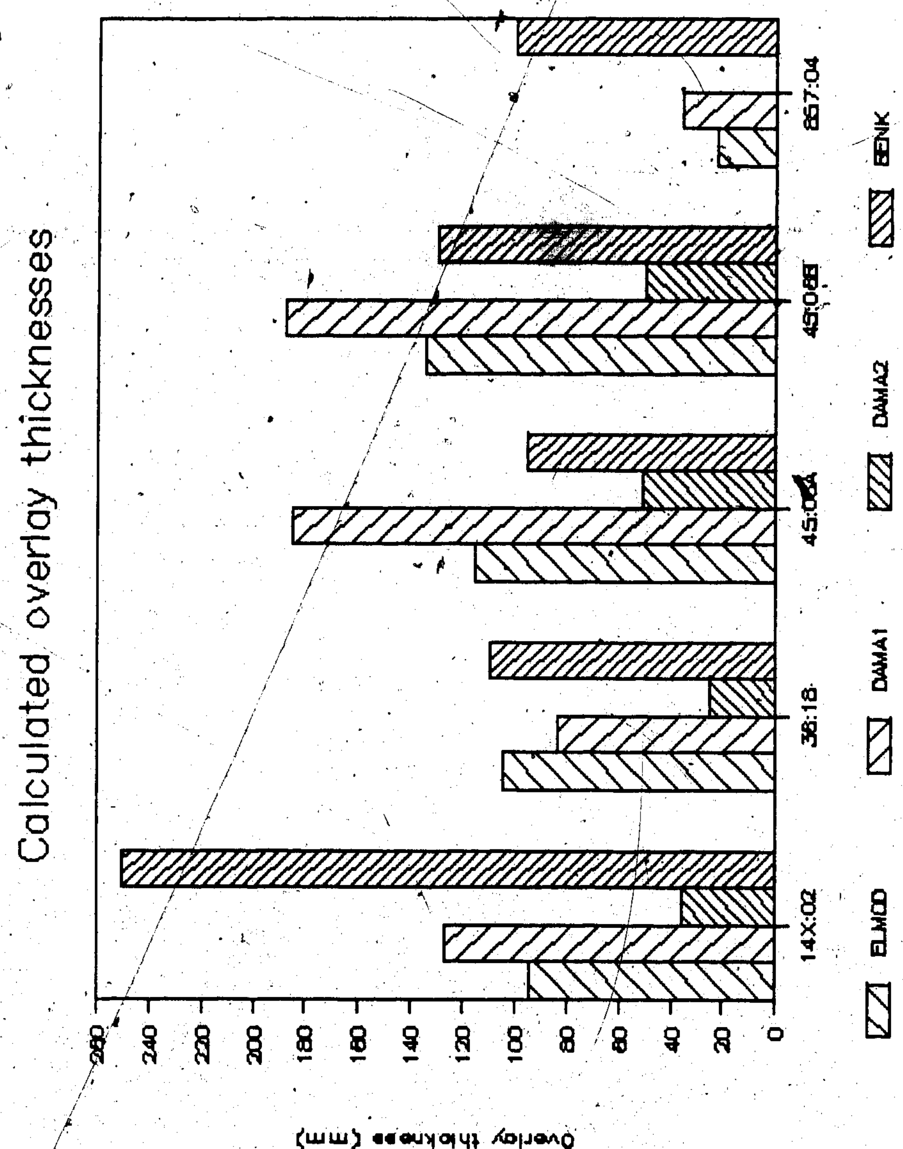

Figure 6.4 Overlay thicknesses calculated using different design procedures

TABLE 6.1. Data for DAMA computer program for selected highway sections

No.	HIGHWAY	Month opened to traffic	Air void (percent)	Asphalt volume (percent)	P200 (percent)	Absolute viscosity @25C (10^6 poise)
1.	1:10	JUNE	8.4	13.8	12.4	0.172
2.	1A:08	AUG.	6.8	11.8	8.0	0.154
3.	2:32	SEP.	9.5	13.2	9.2	3.710
4.	3:08	SEP.	7.2	13.5	6.5	0.160
5.	9:06	AUG.	8.2	14.1	7.1	0.131
6.	12:08	JUL.	8.0	16.0	7.6	0.130
7.	14X:02	JUN.	7.3	11.8	3.6	0.130
8.	16A:20	NOV.	6.6	14.1	8.7	0.150
9.	20:02	SEP.	5.2	14.5	5.6	0.151
10.	27:10	SEP.	10.2	14.5	11.1	0.168
11.	28:02	OCT.	5.9	13.6	4.7	0.130
12.	36:02	AUG.	10.3	13.8	10.1	0.360
13.	49:04	SEP.	5.6	11.1	4.9	0.61[illegible]
14.	55:10	JUL.	9.1	14.3	6.0	0.140
15.	55:16	JUL.	6.0	13.8	4.4	0.131
16.	60:04	NOV.	5.3	13.3	10.1	0.130
17.	881:10	SEP.	8.6	12.8	7.9	0.130
18.	16:18	JUN.	8.2	14.1	9.0	0.363
19.	33:04	SEP.	7.9	15.4	6.9	0.151
20.	60:02	DEC.	5.4	13.5	8.0	0.130
21.	507:02	SEP.	4.2	11.1	6.1	0.130
		Avg.	7.3	13.5	7.5	0.355
		St.dev.	1.7	1.2	2.3	0.759

Table 6.2 Mean monthly air temperatures for selected highway locations (deg. C)

Highway	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.	OCT.	NOV.	DEC.
1:10	-11.8	-7.3	-4.0	3.3	9.4	13.5	16.4	15.2	10.6	5.5	-2.7	-7.8
1A:08	-11.8	-7.3	-4.0	3.3	9.4	13.5	16.4	15.2	10.6	5.5	-2.7	-7.8
2:32	-16.5	-11.4	-6.7	3.2	10.1	14.1	15.8	14.8	9.8	4.7	-5.5	-13.1
3:08	-10.3	-5.4	-2.1	4.9	11.0	15.4	18.6	17.6	12.7	7.5	-0.8	-5.8
9:06	-14.7	-10.4	-4.0	5.3	11.9	16.4	19.2	17.9	12.4	5.8	-4.6	-11.7
12:08	-15.5	-10.5	-6.1	3.1	9.9	13.7	16.1	14.9	10.1	4.7	-4.4	-11.3
14X:02	-15.6	-10.9	-5.6	3.9	10.8	14.7	16.9	15.6	10.4	5.1	-4.9	-11.8
16A:20	-15.6	-10.9	-5.6	3.9	10.8	14.7	16.9	15.6	10.4	5.1	-4.9	-11.8
20:02	-15.5	-10.5	-6.1	3.1	9.9	13.7	16.1	14.9	10.1	4.7	-4.4	-11.3
27:10	-14.7	-10.4	-4.0	5.3	11.9	16.4	19.2	17.9	12.4	5.8	-4.6	-11.7
28:02	-15.6	-10.9	-5.6	3.9	10.8	14.7	16.9	15.6	10.4	5.1	-4.9	-11.8
36:02	-10.8	-5.6	-1.8	4.8	11.0	15.4	18.9	17.9	12.5	7.3	-0.9	-6.1
49:04	-17.0	-10.4	-5.7	3.8	11.2	14.8	16.8	15.6	10.6	5.4	-4.8	-12.0
55:10	-17.9	-11.9	-6.4	3.4	10.1	14.1	16.2	14.8	9.5	4.6	-6.1	-13.8
55:16	-17.8	-13.6	-7.6	2.9	10.4	14.5	16.9	15.5	9.8	4.5	-6.2	-14.2
60:04	-15.1	-10.0	-5.6	3.4	10.4	14.0	16.3	15.1	10.1	5.2	-4.2	-10.7
881:10	-19.5	-13.8	-8.3	2.8	10.2	14.0	16.4	14.9	9.4	3.8	-6.8	-14.7
16:18	-15.6	-10.9	-5.6	3.9	10.8	14.7	16.9	15.6	10.4	5.1	-4.9	-11.8
33:04	-15.1	-10.0	-5.6	3.4	10.4	14.0	16.3	15.1	10.1	5.2	-4.2	-10.7
60:02	-15.1	-10.0	-5.6	3.4	10.4	14.0	16.3	15.1	10.1	5.2	-4.2	-10.7
507:02	-10.2	-5.2	-2.6	3.4	9.2	13.3	16.5	15.4	10.9	6.1	-1.5	-6.1
Avg.	-14.2	-9.4	-4.9	3.6	10.0	13.8	16.2	15.0	10.1	5.1	-4.0	-10.3

TABLE 6.3. Output of DAMA computer program for selected highway sections
(The Asphalt Institute original fatigue criterion)

No.	HIGHWAY	km-km	ESAL/mo.	Subgrade modulus (psi.)	Thick. (in.)	Year of constr.	Allowable no.of rep. (fatigue)	Allowable no.of rep. (deform.)	No.of rep. to 1987*	Cumulative damage (fatigue)	Cumulative damage (deform.)
1.	1:10	6.44 - 13.60	12942	12000	10.00	1975	7821000	29240000	1863648	0.24	0.06
2.	1A:08	5.10 - 13.35	4511	12000	8.00	1973	3312000	6851000	757848	0.23	0.11
3.	2:32	1.32 - 2.51	31600	4500	12.00	1979	12110000	48400000	3033600	0.25	0.06
4.	3:08	5.78 - 6.21	10253	50000	9.00	1973	24720000	118400000	1722504	0.07	0.01
5.	9:06	21.72 - 22.20	3183	3000	9.00	1974	730200	243200	496548	0.68	2.04
6.	12:08	39.02 - 42.09	3702	7500	9.00	1977	3036000	2297000	444240	0.15	0.19
7.	14X:02	0.00 - 3.22	13930	17000	11.80	1975	15880000	77910000	2005920	0.13	0.03
8.	16A:20	12.18 - 13.03	17433	12000	12.00	1973	32920000	71220000	2928744	0.09	0.04
9.	20:02	0.00 - 4.97	938	7500	4.50	1971	619000	77140	180096	0.29	2.33
10.	27:10	36.82 - 37.16	1324	3000	9.00	1974	442000	239200	206544	0.47	0.86
11.	28:02	29.64 - 31.29	4612	12000	8.00	1974	4563000	3142000	719472	0.16	0.23
12.	36:02	43.44 - 56.64	2097	7500	8.00	1979	495000	644600	201312	0.41	0.31
13.	49:04	7.48 - 26.79	2140	4500	8.00	1969	1963000	989200	462240	0.24	0.47
14.	55:10	0.32 - 5.15	1571	15000	6.00	1974	530200	515900	245076	0.46	0.48
15.	55:16	24.62 - 36.77	1054	7500	7.00	1979	2799000	1261000	101184	0.04	0.08
16.	60:04	0.00 - 3.22	9145	12000	9.00	1975	17100000	22070000	1316880	0.08	0.06
17.	881:10	0.93 - 11.10	591	12000	6.00	1975	438900	508700	85104	0.19	0.17
18.	16:18	14.84 - 17.45	17915	7500	12.00	1969	11530000	25020000	3869640 *	0.34	0.15
19.	33:04	16.09 - 30.03	1851	7500	6.00	1974	417000	129400	288756 *	0.69	2.23
20.	60:02	31.46 - 32.78	5037	15000	9.00	1975	18510000	25450000	725328 *	0.04	0.03
21.	507:02	17.61 - 24.56	31	6750	4.13	1974	418500	67200	4836 *	0.01	0.07

NOTE: '*' means the year is different than 1987
16:18 overlaid in 1985
33:04 overlaid in 1985
60:02 overlaid in 1982
507:02 reconstructed in 1983

TABLE 6.4 Output of DAMA computer program for selected highway sections
(Finn et al. second criterion i.e.<10% fatigue cracking)

No.	HIGHWAY	km-km	ESAL/mo.	Subgrade modulus (psi.)	Thick. (in.)	Year of constr.	Allowable no.of rep. (fatigue)	Allowable no.of rep. (deform.)	No.of rep. to 1987*	Cumulative damage (fatigue)	Cumulative damage (deform.)
1.	1:10	6.44 - 13.60	12942	12000	10.00	1975	5694000	29240000	1863648	0.33	0.06
2.	1A:08	5.10 - 13.35	4511	12000	8.00	1973	2411000	6851000	757848	0.31	0.11
3.	2:32	1.32 - 2.51	31600	4500	12.00	1979	8820000	48400000	3033600	0.34	0.06
4.	3:08	5.78 - 6.21	10253	50000	9.00	1973	18000000	118400000	1722504	0.10	0.01
5.	9:06	21.72 - 22.20	3183	3000	9.00	1974	531700	243200	496548	0.93	2.04
6.	12:08	39.02 - 42.09	3702	7500	9.00	1977	2211000	2297000	444240	0.20	0.19
7.	14X:02	0.00 - 3.22	13930	17000	11.80	1975	11560000	77910000	2005920	0.17	0.03
8.	16A:20	12.18 - 13.03	17433	12000	12.00	1973	23970000	71220000	2928744	0.12	0.04
9.	20:02	0.00 - 4.97	938	7500	4.50	1971	450700	77140	180096	0.40	2.33
10.	27:10	36.82 - 37.16	1324	3000	9.00	1974	321800	239200	206544	0.64	0.86
11.	28:02	29.64 - 31.29	4612	12000	8.00	1974	3322000	3142000	719472	0.22	0.23
12.	36:02	43.44 - 56.64	2097	7500	8.00	1979	360400	644600	201312	0.56	0.31
13.	49:04	7.48 - 26.79	2140	4500	8.00	1969	1429000	989200	462240	0.32	0.47
14.	55:10	0.32 - 5.15	1571	15000	6.00	1974	386100	515900	245076	0.63	0.48
15.	55:16	24.62 - 36.77	1054	7500	7.00	1979	2038000	1261000	101184	0.05	0.08
16.	60:04	0.00 - 3.22	9145	12000	9.00	1975	12450000	22070000	1316880	0.11	0.06
17.	881:10	0.93 - 11.10	591	12000	6.00	1975	319600	508700	85104	0.27	0.17
18.	16:18	14.84 - 17.45	17915	7500	12.00	1969	8395000	25020000	3869640 *	0.46	0.15
19.	33:04	16.09 - 30.03	1851	7500	6.00	1974	303600	129400	288756 *	0.95	2.23
20.	60:02	31.46 - 32.78	5037	15000	9.00	1975	13480000	25450000	725328 *	0.05	0.03
21.	507:02	17.61 - 24.56	31	6750	4.13	1974	304700	67200	4836 *	0.02	0.07

NOTE: "*" means the year is different than 1987
16:18 overlaid in 1985
33:04 overlaid in 1985

60:02 overlaid in 1982
507:02 reconstructed in 1983

TABLE 6.5 Output of DAMA computer program for selected highway sections
(Danish fatigue criterion)

No.	HIGHWAY	km-km	ESAL/mo.	Subgrade modulus (psi.)	Thick. (in.)	Year of constr.	Allowable no.of rep. (fatigue)	Allowable no.of rep. (deform.)	No.of rep. to 1987*	Cumulative damage (fatigue)	Cumulative damage (deform.)
1.	1:10	6.44 - 13.60	12942	12000	10.00	1975	11460000	29240000	1863648	0.16	0.06
2.	1A:08	5.10 - 13.35	4511	12000	8.00	1973	2725000	6851000	757848	0.28	0.11
3.	2:32	1.32 - 2.51	31600	4500	12.00	1979	8820000	48400000	3033600	0.34	0.06
4.	3:08	5.78 - 6.21	10253	50000	9.00	1973	54510000	118400000	1722504	0.03	0.01
5.	9:06	21.72 - 22.20	3183	3000	9.00	1974	74320	243200	496548	6.68	2.64
6.	12:08	39.02 - 42.09	3702	7500	9.00	1977	968800	2297000	444240	0.46	0.19
7.	14X:02	0.00 - 3.22	13930	17000	11.80	1975	30580000	77910000	2005920	0.07	0.03
8.	16A:20	12.18 - 13.03	17433	12000	12.00	[illegible]	71200000	71220000	2928744	0.04	0.04
9.	20:02	0.00 - 4.97	938	7500	4.50	1971	51630	[illegible]	180096	3.49	2.33
10.	27:10	36.82 - 37.16	1324	3000	9.00	1974	[illegible]	239200	206544	4.72	0.86
11.	28:02	29.64 - 31.29	4612	12000	8.00	1974	1848000	3142000	719472	0.39	0.23
12.	36:02	43.44 - 56.64	2097	7500	8.00	1979	70140	644600	201312	2.87	0.31
13.	49:04	7.48 - 26.79	2140	4500	8.00	1969	868400	989200	462240	0.53	0.47
14.	55:10	0.32 - 5.15	1571	15000	6.00	1974	75110	515900	245076	3.26	0.48
15.	55:16	24.62 - 36.77	1054	7500	7.00	1979	739600	1261000	101184	0.14	0.08
16.	60:04	0.00 - 3.22	9145	12000	9.00	1975	26980000	22070000	1316880	0.05	0.06
17.	881:10	0.93 - 11.10	591	12000	6.00	1975	79080	508700	85104	1.08	0.11
18.	16:18	14.84 - 17.45	17915	7500	12.00	1969	15150000	25020000	3869640 *	0.26	0.15
19.	33:04	16.09 - 30.03	1851	7500	6.00	1974	34210	129400	288756 *	8.44	2.23
20.	60:02	31.46 - 32.78	5037	15000	9.00	1975	26990000	25450000	725328 *	0.03	0.03
21	507:02	17.61 - 24.56	31	6750	4.13	1974	79710	67200	4836 *	0.06	0.07

NOTE: "*" means the year is different than 1987
16:18 overlaid in 1985
33:04 overlaid in 1985

60:02 overlaid in 1982
507:02 reconstructed in 1982

TABLE 6.6 Inventory data regarding full-depth sections selected for FWD-testing

No.	HIGHWAY	km-km	ESAL/mo.	AC Thick. (in.)	Year of constr.	RCI/year	VCR/year	Benkelman beam testing year	temp.C	DBAR	DMAX
1	14X:02	0.00 - 3.22	13930	11.80	1975	5.2/1985	73/1978	1985	15	0.021	0.037
2	36:18	0.00 - 1.61	1532	4.50	1974	6.0/1986	72/1985	1986	11	0.036	0.055
3	45:06A	12.41 - 13.98	1730	6.00	1971	6.2/1984	63/1985	1984	21	0.041	0.056
4	857:04	8.00 - 13.00	486	7.00	1978	7.7/1986	72/1985	1984	25	0.095	0.120
5	45:06B	23.11 - 23.88	811	8.50	1971	6.2/1984	64/1985	1984	24	0.53	0.089

Table 6.7 Deflections on full-depth pavements

Project	Structure (mm)	FWD Temp.range deg.C	FWD Mean defl. (microns)	FWD St.dev.	DYNAFLECT Temp.range deg.C	DYNAFLECT Mean defl. (microns)	DYNAFLECT St.dev.	BENKELMAN BEAM Temp.range deg.C	BENKELMAN BEAM Mean defl. (microns)	BENKELMAN BEAM St.dev.
14X:02	300	16	340	150	24	33	8	16 - 19	1313	503
36:18	115	24 - 25	654	138	24	43	19	23	1313	274
45:06A	150	15 - 20	553	194	20	41	6	12 - 20	1580	343
45:06B	215	20 - 22	592	223	22	58	17	18 - 19	1730	516
857:04	180	8 - 15	432	112	15	47	11	13 - 18	2492	615

Table 6.8 Highway 14X:02. Comparison of backcalculated moduli using ELMOD and FWDUTIS

sta.	ELMOD E1(MPa)	ELMOD E2(MPa)	FWDUTIS E1(MPa)	FWDUTIS E2(MPa)	DIFFERENCE E1(MPa)	DIFFERENCE E2(MPa)
0.020	1208	93	793	132	415	39
0.220	764	119	552	138	212	19
0.240	2281	221	2281	221	0	0
0.260	2007	197	2007	197	0	0
0.280	1025	158	1025	255	0	97
0.340	2690	197	2621	214	69	17
0.360	1620	170	1172	248	448	78
0.400	1188	71	897	110	291	39
0.440	2393	138	2069	159	324	21
0.480	2547	129	2000	160	547	31
0.520	2781	94	2069	138	712	44
100.040	1613	74	1103	98	510	24
100.080	2518	99	2345	126	173	27
100.100	2536	108	1931	141	605	33
100.200	2753	85	1931	122	822	37
100.220	3063	82	2000	111	1063	29
100.260	2382	152	2382	153	0	1
100.280	940	80	828	97	112	17
100.300	1012	77	828	97	184	20
100.360	3383	96	3103	114	280	18
Mean:	2035	122	1697	152	338	30
St dev :	777	44	729	50	300	24

Table 6.9 Highway 38:18. Comparison of backcalculated moduli using ELMOD and FWDUTIS

	ELMOD		FWDUTIS		DIFFERENCE	
sta.	E1 (MPa)	E2 (MPa)	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)
0.04	4827	84	4620	91	207	7
0.08	5188	74	3889	101	1299	27
0.12	3628	66	3572	76	56	10
0.16	3462	70	2759	76	703	6
0.20	7317	73	7415	75	-98	2
0.22	7777	62	7777	62	0	0
0.24	6012	57	6012	57	0	0
0.26	8852	59	9090	62	-238	3
0.28	9148	86	9121	84	27	-2
0.30	9113	87	9143	91	0	4
0.32	3872	69	3840	72	32	3
0.38	2504	64	1385	88	1119	24
0.50	2875	66	1724	101	1151	35
Mean:	5737	71	5409	80	327	9
St. dev.:	2161	12	2837	15	538	12

Table 6.10. Highway 45:06A. Comparison of backcalculated moduli using ELMOD and FWDUTIS

	ELMOD		FWDUTIS		DIFFERENCE	
sta.	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)
684.460	10760	102	8966	110	1794	8
684.480	2121	59	1103	84	1018	25
684.500	7217	67	5517	100	1700	33
684.520	6204	73	4138	103	2066	30
700.120	1187	95	828	121	359	26
700.160	4899	114	4138	137	761	23
700.200	5912	103	4138	133	1774	30
700.220	8205	113	6414	145	1791	32
700.260	6133	111	6759	123	-626	12
700.280	1514	94	897	128	617	34
700.300	359	140	552	121	-193	-19
700.340	3210	95	2069	131	1141	36
700.380	1297	80	1172	90	125	10
700.460	1193	97	828	121	365	24
700.480	2080	103	1517	124	563	21
700.500	398	92	448	90	-50	-2
800.040	4463	104	3655	135	808	31
800.060	986	77	759	88	227	11
800.080	3791	90	3791	97	0	7
800.120	814	97	814	97	0	0
800.140	3135	100	2759	100	376	0
800.180	2885	93	2897	93	-12	0
800.200	2213	100	2213	100	0	0
800.320	824	100	824	100	0	0
800.340	876	90	876	97	0	7
Mean:	3307	96	2723	111	584	15
St.dev.:	2765	16	2299	18	746	15

Table 6.11 Highway 45:068. Comparison of backcalculated moduli using ELMOD and FWDUTIS

	ELMOD		FWDUTIS		DIFFERENCE	
sta.	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)
367.380	1007	146	1007	146	0	0
367.420	1218	126	1223	129	-5	3
367.440	3292	96	2345	116	947	20
367.460	2865	116	2865	122	0	6
367.480	4077	94	3310	110	767	16
367.500	4249	69	3874	95	375	26
400.020	2962	49	1379	75	1583	26
400.100	4818	64	3310	87	1508	23
400.120	1801	55	552	105	1249	50
400.140	1548	59	1034	77	514	18
400.160	1550	73	1295	99	255	26
400.180	1290	61	1034	73	256	12
400.240	7380	72	5724	77	1656	5
400.280	652	45	483	58	169	13
400.300	998	50	759	66	239	16
400.340	2303	55	1517	74	786	19
400.360	1044	51	897	60	147	9
400.380	1942	51	1379	74	563	23
400.440	3496	94	3496	94	0	0
Mean:	2552	75	1973	91	579	16
St dev.:	1707	29	1415	25	566	12

Table 6.12 Highway 857:04. Comparison of backcalculated moduli using ELMOD and FWDUTIS

	ELMOD		FWDUTIS		DIFFERENCE	
sta.	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)
497.520	3741	75	3103	87	638	12
500.020	3918	95	3918	99	0	4
500.040	3316	68	2621	97	695	29
500.060	7008	101	7008	101	0	0
500.080	2261	75	1586	92	675	17
500.100	5179	102	5179	113	0	11
500.120	2741	81	2414	100	327	19
500.160	4357	103	3793	123	564	20
500.200	4818	109	5103	107	-285	-2
500.220	5846	125	6276	131	-430	6
500.260	7943	112	8966	117	-1023	5
500.280	2466	73	2466	81	0	8
500.300	7380	100	8276	103	-896	3
500.320	1648	86	1648	92	0	6
500.340	4882	101	4882	114	0	13
500.360	3412	90	4000	87	-588	-3
500.380	3979	90	3979	97	0	7
500.400	3241	73	3241	81	0	8
500.420	5370	89	5862	93	-492	4
500.460	4654	88	4654	88	0	0
500.480	1312	62	966	77	346	15
500.500	3780	70	3448	86	332	16
500.520	1923	68	1923	74	0	6
600.020	3167	80	3167	90	0	10
600.040	2083	52	1724	72	359	20
600.060	4556	85	4556	85	0	0
600.120	1750	52	1379	66	371	14
600.140	8746	94	8746	103	0	9
600.160	3054	81	2759	97	295	16
600.180	3135	100	3793	100	-658	0
600.200	5068	117	4931	130	137	13
600.220	3861	85	3448	101	413	16
600.240	5670	128	5793	132	-123	4
600.260	6017	139	5862	155	155	16
600.280	2404	81	2404	92	0	11
600.300	5673	97	5903	102	-230	5
600.320	2386	78	2069	93	317	15
600.340	4697	100	4697	104	0	4
600.360	2709	77	2414	94	295	17
600.380	3829	73	3829	83	0	10
600.400	3841	73	3841	83	0	10
600.420	5914	93	6690	92	-776	-1
600.440	3470	65	3103	76	367	11
600.460	3027	74	3027	83	0	9
600.480	5241	74	5241	79	0	5
600.500	2525	58	2525	76	0	18

Table 6.12 (cont.) Highway 857:04. Comparison of backcalculated moduli using ELMOD and FWDUT1S

	ELMOD		FWDUT1S		DIFFERENCE	
sta.	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)	E1(MPa)	E2(MPa)
600.520	4910	68	4483	77	427	9
700.020	3672	63	2759	83	913	20
700.040	3213	59	2966	81	247	22
700.060	2381	74	1724	92	657	18
700.080	3168	64	2414	81	754	17
700.100	4259	75	4259	90	0	15
700.140	2489	66	1552	93	937	27
700.160	3349	82	2138	110	1211	28
700.180	2911	79	2414	103	497	24
700.260	2446	73	2069	97	377	24
700.280	4505	81	3793	98	712	17
700.300	1329	81	1329	93	0	12
700.320	9152	92	9152	95	0	3
700.340	4293	88	4293	94	0	6
700.380	3180	83	3241	97	-61	14
700.400	3195	78	2759	95	436	17
700.420	3517	86	3586	93	-69	7
700.440	2689	89	2689	97	0	8
700.460	3985	88	4069	95	-84	7
700.480	3999	85	3448	100	551	15
700.500	3929	84	3862	112	67	28
700.520	2353	76	2069	97	284	21
800.040	2602	72	2602	103	0	31
Mean:	3642	79	3525	90	117	11
St.dev.:	1643	17	1860	15	412	8

Table [illegible] Highway [illegible]:02. Subgrade moduli as used for DAMA analyses.

sta.	sub.mod (MPa)	Rt (%)	JAN.-MAR	APR.	MAY	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.
100.060	107	56.70	345	61	61	72	84	95	107	107	226	345
100.100	108	56.97	345	62	62	73	85	96	108	108	226	345
100.140	78	47.24	345	37	37	47	57	68	78	78	211	345
100.180	50	31.00	345	16	16	24	33	41	50	50	197	345
100.220	82	49.56	345	41	41	51	61	72	82	82	213	345
100.260	152	69.12	345	105	105	117	129	140	152	152	248	345
100.300	77	46.66	345	36	36	46	56	67	77	77	211	345
100.340	84	50.34	345	42	42	53	63	74	84	84	214	345
100.380	88	51.45	345	45	45	56	67	77	88	88	216	345
100.420	69	42.02	345	29	29	39	49	59	69	69	207	345

Table 6.14 Highway 36:18. Subgrade moduli as used for DAMA computer analyses

sta.	sub.mod (MPa)	Rt (%)	JAN.-MAR	APR.	MAY	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.
0.340	48	29.84	345	14	14	23	31	40	48	48	196	345
0.360	66	40.28	345	27	27	36	46	56	66	66	205	345
0.380	64	39.12	345	25	25	35	45	54	64	64	204	345
0.400	61	37.38	345	23	23	32	42	51	61	61	203	345
0.420	70	42.60	345	30	30	40	50	60	70	70	207	345
0.440	58	35.64	345	21	21	30	39	49	58	58	201	345
0.460	74	44.92	345	33	33	43	54	64	74	74	209	345
0.480	69	42.02	345	29	29	39	49	59	69	69	207	345
0.500	66	40.28	345	27	27	36	46	56	66	66	205	345
0.520	74	44.92	345	33	33	43	54	64	74	74	209	345

Table 6.15 Highway 45:06A. Subgrade moduli as used for DAMA analyses.

sta.	sub.mod (MPa)	Rt (%)	JAN.-MAR	APR.	MAY	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.
800.180	93	52.83	345	49	49	60	71	82	93	93	219	345
800.200	100	54.76	345	55	55	66	77	89	100	100	222	345
800.220	101	55.04	345	56	56	67	78	90	101	101	223	345
800.240	101	55.04	345	56	56	67	78	90	101	101	223	345
800.260	87	51.17	345	45	45	55	66	76	87	87	216	345
800.280	82	49.56	345	41	41	51	61	72	82	82	213	345
800.300	90	52.00	345	47	47	58	68	79	90	90	217	345
800.320	100	54.76	345	55	55	66	77	89	100	100	222	345
800.340	90	52.00	345	47	47	58	68	79	90	90	217	345
800.360	85	50.62	345	43	43	54	64	75	85	85	215	345

NOTE: Rt - thaw reduction factor

Table 6.16 Highway 45:06B. Subgrade moduli as used for DAMA analyses.

sta.	sub.mod (MPa)	Rt (%)	JAN.-MAR	APR.	MAY	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.
400.240	72	43.76	345	32	32	42	52	62	72	72	208	345
400.260	51	31.58	345	16	16	25	34	42	51	51	198	345
400.280	45	28.10	345	13	13	21	29	37	45	45	195	345
400.300	50	31.00	345	16	16	24	33	41	50	50	197	345
400.320	63	38.54	345	24	24	34	44	53	63	63	204	345
400.340	55	33.90	345	19	19	28	37	46	55	55	200	345
400.360	51	31.58	345	16	16	25	34	42	51	51	198	345
400.380	51	31.58	345	16	16	25	34	42	51	51	198	345
400.400	48	29.84	345	14	14	23	31	40	48	48	196	345
400.420	48	29.84	345	14	14	23	31	40	48	48	196	345

Table 6.17 Highway 857:04. Subgrade moduli as used for DAMA analyses.

sta.	sub.mod (MPa)	Rt (%)	JAN.-MAR	APR.	MAY	JUN.	JUL.	AUG.	SEP.	OCT.	NOV.	DEC.
700.320	92	52.55	345	48	48	59	70	81	92	92	218	345
700.340	88	51.45	345	45	45	56	67	77	88	88	216	345
700.360	72	43.76	345	32	32	42	52	62	72	72	208	345
700.380	83	50.07	345	42	42	52	62	73	83	83	214	345
700.400	78	47.24	345	37	37	47	57	68	78	78	211	345
700.420	86	50.90	345	44	44	54	65	75	86	86	215	345
700.440	89	51.72	345	46	46	57	68	78	89	89	217	345
700.460	88	51.45	345	45	45	56	67	77	88	88	216	345
700.480	85	50.62	345	43	43	54	64	75	85	85	215	345
700.500	84	50.34	345	42	42	53	63	74	84	84	214	345

NOTE: Rt - thaw reduction factor

Table 6.18 Asphalt moduli of existing pavement used for highway 14X:02 DAMA analyses

Sta.	Asphalt modulus (FWD) (MPa)	Asphalt modulus AI formula (MPa)	Pavement temp. deg. C	Correction factor	Asphalt moduli @ temp.C (MPa) -17.8	-13.1	-8.3	-3.6	1.1	5.8	10.6	15.3	20	24.7	29.4	34.2
100.060	2789	4064	16	0.69	9392	9067	8310	7286	6135	4972	3885	2930	2136	1506	1028	680
100.100	2536	4064	16	0.62	8512	8218	7531	6604	5561	4507	3521	2656	1936	1365	932	616
100.140	3050	4064	16	0.75	10238	9883	9058	7943	6688	5420	4235	3194	2329	1642	1121	741
100.180	1837	4064	16	0.45	6166	5953	5456	4784	4028	3264	2551	1924	1403	989	675	446
100.220	3063	4064	16	0.75	10281	9926	9097	7977	6716	5443	4253	3208	2339	1649	1126	744
100.260	2382	4064	16	0.59	7995	7719	7074	6203	5223	4233	3307	2495	1819	1282	875	579
100.300	1012	4064	16	0.25	3397	3279	3005	2635	2219	1798	1405	1060	773	545	372	246
100.340	2700	4064	16	0.66	9063	8749	8019	7031	5920	4798	3749	2828	2061	1454	992	656
100.380	3297	4064	16	0.81	11067	10684	9791	8586	7230	5859	4578	3453	2517	1775	1212	801
100.420	2902	4064	16	0.71	9741	9404	8618	7557	6363	5157	4029	3039	2216	1562	1067	705

Table 6.19 Asphalt moduli of existing pavement used for highway 36:18 DAMA analyses

Sta.	Asphalt modulus (FWD) (MPa)	Asphalt modulus AI formula (MPa)	Pavement temp. deg. C	Correction factor	Asphalt moduli @ temp.C (MPa) -17.2	-12.8	-8.3	-3.9	0.6	5.0	9.4	13.9	18.3	22.8	27.2	31.7
0.340	3926	2251	25	1.74	27144	26080	23904	21033	17822	14573	11517	8808	6525	4686	3265	2208
0.360	4451	2440	24	1.82	28390	27278	25002	21998	18640	15242	12046	9212	6824	4901	3415	2309
0.380	2540	2440	24	1.04	16201	15566	14267	12553	10637	8698	6874	5257	3894	2797	1949	1318
0.400	4332	2440	24	1.78	27631	26548	24333	21410	18141	14834	11724	8966	6642	4770	3323	2248
0.420	3969	2440	24	1.63	25316	24324	22294	19616	16624	14591	10741	8215	6085	4370	3045	2059
0.440	4035	2440	24	1.65	25737	24728	22665	19942	16898	13817	10920	8351	6186	4443	3095	2094
0.460	3311	2440	24	1.36	21119	20291	18598	16364	13866	11338	8961	6853	5076	3646	2540	1718
0.480	5439	2440	24	2.23	34692	33333	30551	26881	22777	18625	14719	11257	8339	5989	4173	2822
0.500	2875	2440	24	1.18	18338	17619	16149	14209	12040	9845	7781	5950	4408	3166	2206	1492
0.520	4064	2440	24	1.67	25921	24906	22828	20086	17019	13916	10998	8411	6231	4475	3118	2109

Table 6.20 Asphalt moduli of existing pavement used for highway 45:06A DAMA analyses

Sta.	Asphalt modulus (FWD) (MPa)	Asphalt modulus AI formula (MPa)	Pavement temp. deg. C	Correction factor	Asphalt moduli @ temp.C (MPa) -17.2	-12.8	-8.3	-3.9	0.6	5.0	9.4	13.9	18.3	22.8	27.2	31.7
800.180	2885	2347	20	1.23	14307	13727	12542	10985	9253	7512	5887	4460	3268	2320	1595	1064
800.200	2213	2347	20	0.94	10975	10530	9621	8427	7098	5762	4516	3421	2507	1779	1224	816
800.220	6690	2347	20	2.85	33177	31831	29085	25474	21458	17420	13652	10341	75[illegible]	5379	3700	2467
800.240	1339	2347	20	0.57	6640	6371	5821	5099	4295	3487	2732	2070	1517	1077	740	494
800.260	1241	2347	20	0.53	6154	5905	5395	4725	3980	3231	2532	1918	1406	998	686	458
800.280	1923	2347	20	0.82	9336	9150	8360	7322	6168	5007	3924	2973	2179	1564	1063	709
800.300	1411	2347	20	0.60	6997	6714	6134	5373	4526	3674	2879	2181	1598	1134	780	520
800.320	824	2347	20	0.35	4086	3921	3582	3138	2643	2146	1682	1274	933	663	456	304
800.340	876	2347	20	0.37	4344	4168	3808	3336	2810	2281	1788	1354	992	704	484	323
800.360	1856	2347	20	0.79	9204	8831	8069	7067	5953	4833	3787	2869	2103	1492	1026	685

Table 6.21 Asphalt moduli of existing pavement used for highway 45:06B DAMA analyses

Sta.	Asphalt modulus (FWD) (MPa)	Asphalt modulus AI formula (MPa)	Pavement temp. deg. C	Correction factor	Asphalt moduli @ temp.C (MPa) -17.2	-12.8	-8.3	-3.9	0.6	5.0	9.4	13.9	18.3	22.8	27.2	31.7
400.240	7380	2008	22	3.68	42777	41043	37501	32846	27667	22461	17603	13334	9772	6935	4770	3182
400.260	1816	2008	22	0.90	10526	10099	9228	8082	6808	5527	4331	3281	2405	1707	1174	783
400.280	652	2008	22	0.39	3779	3626	3313	2902	2444	1984	1555	1178	863	613	421	281
400.300	998	2008	22	0.50	5785	5550	5071	4442	3741	3037	2380	1803	1321	938	645	430
400.320	360	2008	22	N/A	360	360	360	360	360	360	360	360	360	360	360	360
400.340	2303	2008	22	1.15	13349	12808	11703	10250	8634	7009	5493	4161	3049	2164	1489	993
400.360	1044	2008	22	0.52	6051	5806	5305	4646	3914	3177	2490	1886	1382	981	675	450
400.380	1942	2008	22	0.97	11257	10800	9868	8643	7280	5910	4632	3509	2571	1825	1255	837
400.400	835	2008	22	0.42	4840	4644	4243	3716	3130	2541	1992	1509	1106	785	540	360
400.420	1850	2008	22	0.92	10723	10288	9401	8234	6935	5630	4413	3342	2450	1739	1196	798

Table 6.22 Asphalt moduli of existing pavement used for highway 857:04 DAMA analyses

Sta.	Asphalt modulus (FWD) (MPa)	Asphalt modulus AI formula (MPa)	Pavement temp. deg. C	Correction factor	Asphalt moduli @ temp.C (MPa)											
					-17.2	-12.8	-8.3	-3.9	0.6	5.0	9.4	13.9	18.3	22.8	27.2	31.7
700.320	9152	4189	14	2.18	29752	28539	26065	23814	19200	15571	12188	9219	6746	4779	3281	2134
700.340	4293	4189	14	1.02	13956	13387	12226	10701	9006	7304	5717	4324	3164	2242	1539	1024
700.360	2945	4189	14	0.70	9574	9184	8387	7341	6178	5011	3922	2967	2171	1538	1056	703
700.380	3180	4189	14	0.76	10338	9916	9057	7927	6671	5410	4235	3203	2344	1661	1140	759
700.400	3195	4189	14	0.76	10386	9963	9099	7964	6703	5436	4255	3218	2355	1668	1145	762
700.420	3517	4189	14	0.84	11433	10967	10016	8767	7378	5984	4684	3543	2592	1837	1261	993
700.440	2689	4189	14	0.64	8742	8385	7658	6703	5641	4575	3581	2709	1982	1404	964	642
700.460	3985	4189	14	0.95	12955	12427	11349	9934	8360	6780	5307	4014	2937	2081	1429	951
700.480	3999	4189	14	0.95	13000	12470	11389	9969	8390	6804	5325	4028	2948	2088	1434	954
700.500	3929	4189	14	0.94	12773	12252	11190	9794	8243	6685	5232	3958	2896	2052	1409	938

Table [illegible] Overlay thicknesses using DAMA
+ Finn second fatigue criterion.

14X:02		36:18		45:06A		45:06B		857:04	
Sta.	Overlay (mm)	Sta.	Overlay (mm)	Sta.	Overlay (mm)	Sta.	Overlay (mm)	Sta.	Overlay (mm)
100.060	0	0.340	23	800.180	5	400.240	0	700.320	0
100.100	0	0.360	7	800.200	40	400.260	0	700.340	0
100.140	0	0.380	19	800.220	0	400.280	56	700.360	0
100.180	44	0.400	11	800.240	33	400.300	23	700.380	0
100.220	0	0.420	9	800.260	44	400.320	83	700.400	0
100.260	0	0.440	20	800.280	33	400.340	0	700.420	0
100.300	68	0.460	20	800.300	37	400.360	20	700.440	0
100.340	0	0.480	0	800.320	52	400.380	0	700.460	0
100.380	0	0.500	31	800.340	56	400.400	32	700.480	0
100.420	0	0.520	15	800.360	32	400.440	0	700.500	0
Mean	11.2		15.5		33.2		21.4		0
St.dev.	24.3		9.0		18.1		28.7		0
Design overlay (mm)	36		25		51		50		0

Table 6.24 Overlay thicknesses using DAMA
+ Danish fatigue criterion.

14X:02		36:18		45:06A		45:06B		857:04	
Sta.	Overlay (mm)	Sta.	Overlay (mm)	Sta.	Overlay (mm)	Sta.	Overlay (mm)	Sta.	Overlay (mm)
100.060	39	0.340	78	800.180	84	400.240	0	700.320	0
100.100	51	0.360	49	800.200	107	400.260	82	700.340	0
100.140	45	0.380	98	800.220	0	400.280	200	700.360	0
100.180	142	0.400	55	800.240	150	400.300	147	700.380	36
100.220	41	0.420	58	800.260	170	400.320	223	700.400	39
100.260	36	0.440	69	800.280	132	400.340	46	700.420	30
100.300	203	0.460	41	800.300	152	400.360	140	700.440	48
100.340	60	0.480	33	800.320	189	400.380	79	700.460	0
100.380	31	0.500	88	800.340	190	400.400	173	700.480	0
100.420	61	0.520	56	800.360	82	400.440	83	700.500	0
Mean	70.9		62.5		125.6		117.3		15.3
St.dev.	56.3		20.6		58.9		70.9		20.2
Design overlay (mm)	127		83		185		188		36

Table 6.25 Benkelman beam temperature corrected deflections (mils)

14X:02				36:18				45:06A				45:06B				857:04			
sta.	Def.	Temp.	Cor. def.	sta.	Def.	Temp.	Cor. def.	sta.	Def.	Temp.	Cor. def.	sta.	Def.	Temp.	Cor. def.	sta.	Def.	Temp.	Cor. def.
0.06	46	16	153.93	0.34	70	23	165.94	0.18	58	20	159.75	0.24	68	19	172.18	0.32	60	18	165.61
0.10	38	16	144.53	0.36	52	23	148.98	0.20	50	20	151.51	0.26	72	19	176.42	0.34	76	18	183.11
0.14	48	16	156.23	0.38	66	23	162.17	0.22	48	20	149.45	0.28	104	19	110.3	0.36	92	18	100.6
0.18	69	16	179.66	0.40	58	23	154.63	0.24	52	20	153.57	0.30	88	19	193.41	0.38	72	18	178.74
0.22	40	16	146.90	0.42	50	23	147.10	0.26	48	20	149.45	0.32	86	19	191.28	0.40	80	18	187.49
0.26	28	16	132.83	0.44	54	23	150.87	0.28	52	20	153.57	0.34	66	19	170.05	0.42	84	18	191.86
0.30	57	16	165.69	0.46	52	23	148.98	0.30	50	20	151.51	0.36	80	19	184.92	0.44	86	18	194.05
0.34	42	16	149.24	0.48	64	23	160.29	0.32	56	20	157.69	0.38	64	19	167.93	0.46	64	18	169.99
0.38	34	16	139.95	0.50	54	23	150.87	0.34	58	20	159.75	0.40	70	19	174.30	0.48	66	18	172.18
0.42	46	16	153.93	0.52	36	23	133.91	0.36	50	20	151.51	0.42	76	19	180.67	0.50	62	18	167.80
Avg.			52.29				52.37				53.78				82.16				81.14
st.dev.			13.25				9.09				3.94				13.14				12.17
Spring est. avg.			104.5				104.7				107.5				164.3				162.2
Spring est.st.dev.			26.5				18.18				7.88				26.28				24.34
Design defl. x + 2 * S			157.5				141.1				123.3				216.8				210.9

Table 6.26 Overlay thicknesses calculated using varies procedures (mm)

HIGHWAY SECTION / DESIGN METHOD	14X:02	36:18	45:06A	[illegible]	[illegible]:04
ELMOD	94	105	115	135	22
DAMA1 (+ Finn second fatigue criterion)	36	25	51	50	0
DAMA2 (+ Danish fatigue criterion)	127	84	185	188	36
Benkelman Beam	250	110	95	130	100

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CHAPTER 7

PROPOSED STRATEGIES FOR THE REHABILITATION OF FULL-DEPTH PAVEMENTS IN ALBERTA

7.1 General

As it was presented in Chapters 3, reviewing experiences with full-depth pavements in seasonal frost areas, these pavements perform, generally, adequately when compared with other flexible pavement structures. In Alberta, as described in Chapter 4, there have been some problems with full-depth pavements as indicated by Plewes and Millions (1). Evaluation of full-depth pavements in Region 3 of the Alberta Transportation Highway Network, carried out using the Department Pavement Management System, indicates that low structural adequacy indices (SAI) reflect on the pavements overall quality (PQI) (2). However a study which is underway have indicated an adequate condition of full-depth pavements in the Province (3). A preliminary survey undertaken as described in Chapter 6 appears to support the above statement.

In view of all the above, reconstruction, as it was believed in the past in Alberta, is not the only alternative that should be considered for full-depth pavements in the Province.

7.2 Rehabilitation Measures Proposed for Full-Depth Pavements

Three forms of rehabilitation measures for full-depth asphalt concrete pavements can be considered. These forms are:

1) overlay
2) partial cold milling plus overlay
    a) new material used for overlay
    b) recycled material used for overlay
3) reconstruction
    a) full-depth cold milling plus construction of a conventional type pavement with recycled material utilized
    b) as above but with construction of full-depth asphalt concrete pavement with utilization of the reclaimed asphalt concrete

Each of these rehabilitation measures has certain advantages and disadvantages. Each form will be evaluated subsequently.

7.2.1 Overlay of Full-Depth Pavements

Overlay of full-depth pavements is the easiest of the rehabilitation measures to perform. This fact is due to its simplicity of construction. Prior to overlay all the pavement visible defects and distresses as cracks and permanent deformations should be carefully repaired. Care should be taken to obtain an adequate bond between the existing surface and the overlay. The application of a tack coat should provide adequate bond.

Overlays have also certain disadvantages. They elevate the pavement grade and when of substantial thickness the pavement width is decreased. Also when a thick overlay is applied all guardrails, curbs and gutters have to be elevated. Another disadvantage of the overlays is the phenomena of reflective cracking. This means that the overlays may be adequate from a structural point of view but reflective cracking may develop. This may in a later time affect the bearing capacity of the pavement and lower the quality of ride.

7.2.2 Partial Cold-Milling and Overlay of Full-Depth Pavements

It seems that this form of rehabilitation of the existing asphalt concrete pavements will be gaining greater recognition from the pavement engineers and managers. By using this procedure pavement surface distortions and defects are removed. The cold-milled material can be recycled and, after adding some amount of virgin aggregate and asphalt cement, reused for the planned overlay. This method of rehabilitation usually does not elevate the existing pavement grade or if does the rise is not substantial. With such a method a question arises to what maximum depth the existing pavement may be cold-milled. It seems to be reasonable that this maximum cold-milled portion should not reach the fatigued or underside of the existing pavement. The fatigued or cracked thickness of the existing asphalt pavement may be estimated using relationships developed by Marchionna et. al. (4). After cold-milling and prior to the overlay placement a "cushion pad" or a layer which would retard propagation of cracks from the remained pavement portion to the overlay should be considered. There are many procedures developed to retard the cracks propagation. Reference (5) contains a good review of possible options.

In the analytical empirical design of such a measure the pavement layer remained after cold-milling should be treated as a very good aggregate layer. The moduli should be taken as applicable for such an aggregate course and be in the range of 500 MPa (70,000 psi). Due to fatigue cracks in the layer the moduli will not be temperature dependent. The required overlay thickness should be determined based on properties of asphalt concrete to be used for the overlay, the subgrade and the remaining layer supporting abilities and the expected traffic.

7.2.3 Reconstruction of Full-Depth Pavements

Reconstruction of a pavement is a very drastic form of pavement rehabilitation and should be used only in the case of very deteriorated pavements and when other means of rehabilitation are not expected to produce a pavement desirable performance.

Existing pavement layers provide a valuable source of material and should be reclaimed and reused. Reclaiming the existing pavement should not be to the full depth of the pavement to avoid mixing the reclaimed material with the subgrade. It is also wise to leave a thin layer of asphalt concrete as a "working table" for the equipment and as a temporary pavement to carry traffic. Prior to placement of new pavement layers the remained portion of asphalt concrete and the upper subgrade lift should be mixed up to obtain a uniform subgrade material. A good summary of reconstruction experience with full-depth pavements can be found in References (6) and (7).

The Pavement Engineer has two available procedures to follow. One alternative is to replace the existing full-depth asphalt concrete pavement with a conventional type pavement structure. This method seems to be recognized in Alberta as the desirable form of full-depth pavement reconstruction. The method has been applied for some projects in the Province as is presented in References (6),(7) and (8).

Another possible alternative would be to replace the existing full-depth pavement with the same structure type using the reclaimed and recycled asphalt concrete. Such a method has not been performed in the Province, probably due to the need to correct subgrade deficiencies. It some instances, where the subgrade is in good condition, this method may produce a good pavement performance based on the discussed adequate performance of full-depth pavements.

7.3 Mechanistic-empirical design of rehabilitation measures. Case study.

In this study a design of rehabilitation measures using the mechanistic-empirical method is presented. As a case example the northbound roadway of highway 14X:02 from km 1.79 to km 2.89 was considered.

Three alternatives of possible rehabilitation procedures were selected:

1. Overlay using recycled asphalt concrete.
2. Reconstruction using conventional pavement type with recycled asphalt concrete layer.
3. Partial cold milling of the existing pavement and overlay of the pavement with recycled asphalt concrete layer.

All the three cases were considered assuming a 25 year design period. The design thicknesses were calculated using the DAMA computer program with the second Finn fatigue equation incorporated.

7.3.1 Alternative 1 Overlay

In this alternative the moduli of the existing AC pavement and the subgrade were taken as calculated using the ELMOD computer program. The overlay layer moduli were obtained using the Asphalt Institute regression equation. Overlay thicknesses obtained are reported in Table 7.1. A design overlay thickness of the entire section was calculated utilizing the mean plus one standard deviation approach. Required overlay thickness from the structural point of view is 150 mm (6 in.). To prevent propagation of existing cracks in the pavement through a new layer a crack retardant layer 50 mm (2 in.) thick composed of open graded AC is proposed. This layer was not taken into account in the pavement structural analysis. Figure 7.1 explains this design concept in detail.

7.3.2 Alternative 2 Reconstruction

In the presented reconstruction alternative a concept of deep-strength structure was considered. It was decided to place 150 mm (6 in.) thick granular base on the improved subgrade.

Only one set of DAMA runs for the entire section was performed, assuming that the subgrade will be prepared so to have homogenous properties throughout. It was found that 320 mm (12.6 in.) of the asphalt concrete over the 150 mm granular base would be required for a design traffic. Figure 7.1 explains this design concept.

7.3.3 Alternative 3 Cold-milling and overlay

In this alternative it was decided to use partial cold-milling of the existing AC pavement to a depth of 150 mm (6 in.). The cold milling would take place only on the travelling lanes. The cold milled material would be cast on the pavement shoulders and after addition of virgin aggregate and virgin asphaltic emulsion would be mixed in-place and reused. The material would be utilized as 50 mm (2 in.) crack retardant layer in the cold-milled road bed and 50 mm (2 in.) layer on the shoulders. After construction of such layers the entire pavement width will be overlaid with a normal type hot-mix overlay.

In this analysis the remaining (not cold-milled) portion of the existing pavement was considered as partially fatigued and the modulus of 500 MPa (70,000 psi.) was assigned to it. The crack retardant layer was assigned the same modulus value. The subgrade moduli for the tested points were based on the ELMOD procedure. All the design thicknesses calculated for the selected stations are given in Table 7.2. Figure 7.1 explains this design concept. The calculated overlay thickness was selected based on the mean plus one standard deviation method and is 325 mm (13 in.).

7.4 Alternative Initial Cost Analysis

An initial cost of the three considered rehabilitation alternatives were calculated. Calculation of material quantities and initial construction costs are presented in Appendix G. The unit prices utilized in the calculation are the average unit prices from the highway contracts awarded in Alberta in 1988, or, in some cases, in 1987. This analysis shows that overlay of the existing pavement is the least expensive among the three alternatives selected. Reconstruction is the most expensive alternative, and is 2.25 times more expensive than the overlay. Alternative 3 - cold-milling and overlay cost $160,000/km.

Considering the expected performance of the three alternatives, despite equality of their structural strength, their order would probably be different.

Alternative 2 - reconstruction would probably perform the best and would need a major surface rehabilitation measure after about 15 to 17 years.

The second would be the alternative 3 - cold-milling and overlay. In this concept the subgrade defects would not be removed, but the pavement would receive a very substantial thickness of a new asphalt concrete layer. This would prevent development of non-load associated forms of distress. One may expect that this alternative will perform well, and any major surface rehabilitation will not be required for about 15 years.

Alternative 1 - overlay, which initial cost is the lowest, is expected to perform the poorest among the three concepts. Despite using a crack retardant layer, it is expected that some amount of reflective cracking would develop requiring quite frequent the surface rehabilitation. In a 25 year design period two or three major surface rehabilitation measures can be expected.

Taking everything into account the alternative 3 may be considered as a cost effective rehabilitation measure for some full-depth asphalt concrete pavements, however more through examination is needed. To find out more about an adequacy of this alternative a life-cycle analysis of the three alternatives should be performed.

7.5 Guidelines for Full-Depth Rehabilitation Strategies

This research and recent surveys performed in Alberta on full-depth pavements reveal that these pavements' performance is, generally, satisfactory. This fact disagrees w low Structural Adequacy Indices of these pavements. A limited survey performed by the author of the thesis o seven full-depth pavements in the Province indicates th the pavements shows no or little signs of permanent deforma tion. Low-temperature transverse cracking is the most visible form of the pavement surface distress. Some, very heavily loaded highways (15:04, 14X:02) shows signs of fatigue cracks developed in the longitudinal wheel paths. These cracks have not developed until 12 to 15 years of service.

To work out rehabilitation strategies for a particular full-depth pavement certain steps must be accomplished.

1. Data regarding the pavement construction should be obtained. These data should include: the subgrade soil type and densities achieved during construction, asphalt concrete densities, amount of air voids in the compacted mix, amount of asphalt cement in the mix, type of asphalt cement used, and data regarding aggregate gradation. These data can be obtained from the Alberta Transportation and Utilities Laboratory weekly construction reports. This information might indicate reasons for any abnormal pavement performance. If such data are impossible to obtain coring of the pavement should be performed. The coring may also be helpful in finding the pavement actual thickness and also the thickness of the fatigued portion of the pavement layer.

2. Non-destructive pavement testing along a pavement should be performed. The testing employing multi-sensor deflection devices e.g. Dynaflect or the Falling Weight Deflectometer are considered at present as the most reliable ones. The testing should be performed in the outer wheel paths with a spacing of 50 meters at most. Testing in between the wheel paths should also be considered. The moduli determined in between the wheel paths may

approximate the pavement moduli after construction.

The pavement moduli should be calculated, based on the deflection basin shape measurements. This calculation should preferably be performed using a self-iterative computer software. With the numerous test stations to consider, efficiency of the employed program is very important.

The calculated moduli should be utilized to determine pavement remaining life and to compute required overlay thickness. This calculation should be performed for each tested point rather than for an "average" moduli of a homogenous section.

A computer program for pavement life calculation should use fatigue equations and permanent deformation equations proper to materials locally employed and environmental conditions encountered. Taking the pavement subgrade non-linearity into account would be a definite asset of such a program.

3. A considered pavement should be carefully rated in terms of its visual conditions. This visual inspection will reveal the most pronounced forms of surface distress and allow to determine borders of weaker or more deteriorated pavement sections. It would be valuable to perform this inspection together with the structural capacity measurements. This way the two surveys could be closely correlated. During the visual inspection a pavement and its grade actual widths, shoulders condition, surface drainage, locations of objects limiting change of the pavement

elevation (bridges, overpasses, guardrails), and so on should be also determined.

4. Based on the collected information, several rehabilitation alternatives should be selected. Rehabilitation measures will depend on the structural strength of the pavement, its thickness, age, visual conditions, ride quality, availability of overlay, and so on.

Among the selected alternatives reconstruction of the entire pavement should be considered as the last possible resort, and only for thin (100 - 150 mm) and very badly deteriorated full-depth pavements. For thin pavements, but performing adequately, an overlay should be considered as a normal type rehabilitation procedure, unless it is uneconomical because of other than structural constraints. For medium thickness full-depth pavements (150 - 250 mm) overlay or partial cold-milling and overlay should be considered as an appropriate method of rehabilitation. Thick full-depth asphalt concrete pavements (above 250 mm) should be generally partially cold-milled and overlaid when a rehabilitation measure is necessary. Cold-milling should be generally limited to travelling lanes, unless the shoulders are in very bad condition.

Resuming, rehabilitation measures applied to full-depth pavements should be differentiated depending on the pavement structural, visual and riding conditions. The selected rehabilitation measures should be justified taking long term economics into account.

7.6 Summary

Three methods of full-depth pavements rehabilitation have been described in this Chapter. Advantages and disadvantages of these methods are also mentioned.

A mechanistic-empirical design procedure for rehabilitation measures was described in this Chapter. For the three rehabilitation methods selected, required pavement overlay thicknesses for a design traffic were calculated using the mentioned approach.

Required material quantities and the costs for each considered alternative were approximately calculated. It was estimated that alternative 1 - overlay cost $112,487 per kilometer, compared to $254,235 for alternative 2 - reconstruction and $156,349 for alternative 3 - cold-milling and overlay.

At last guidelines for full-depth AC pavement rehabilitation strategies has been described. The guidelines emphasize the need for obtaining proper information regarding the pavement materials, construction workmanship quality and proper evaluation of the pavement structural, visual and riding quality. Based on these and on a careful economical analysis rehabilitation measures should be selected. Some suggestions on how to rehabilitate various full-depth pavements were also mentioned in this Chapter.

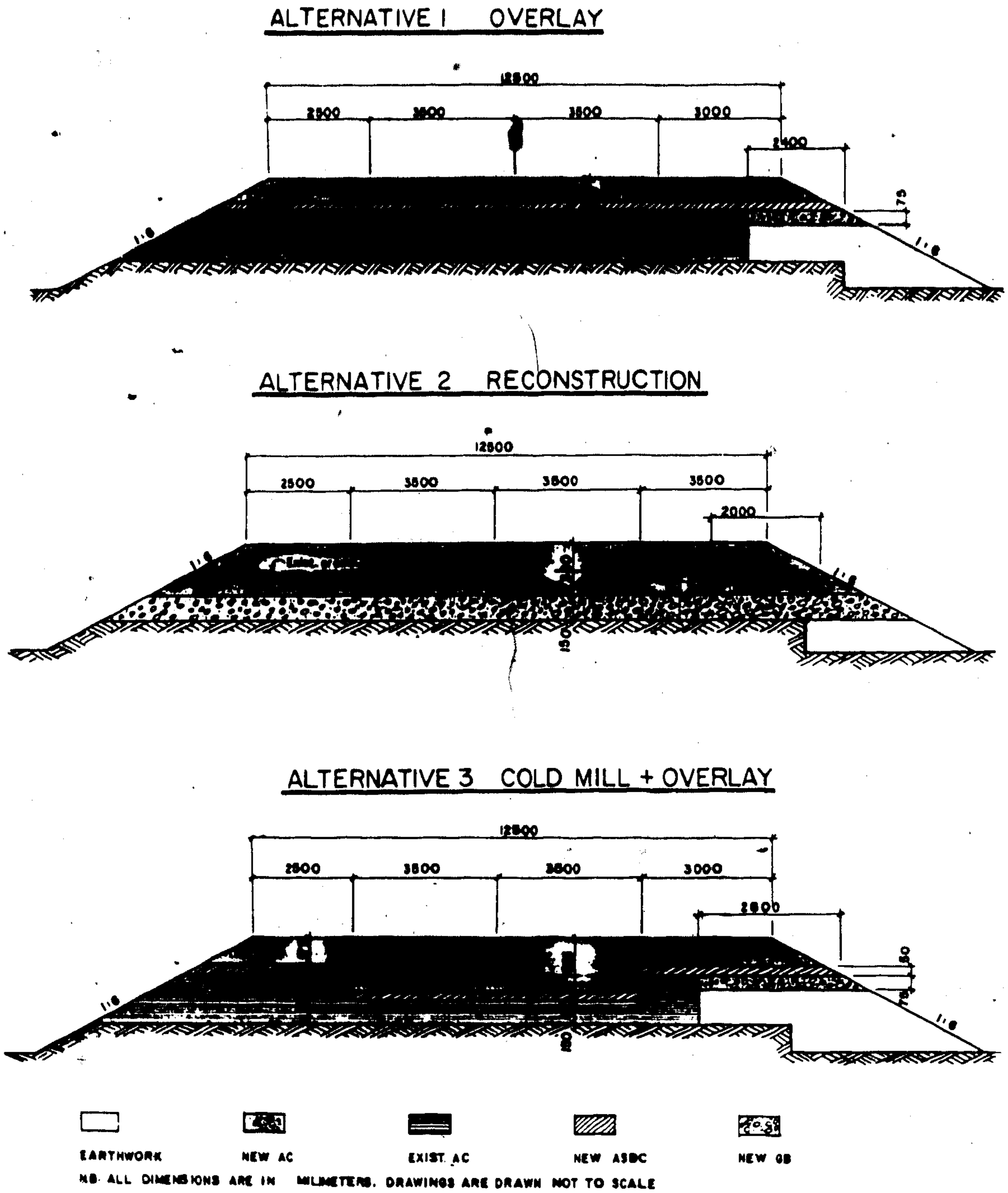

Figure 7.1 Alternative rehabilitation measure concepts for full-depth pavement. Highway 14X:02 km 1.79 to km 2.89.

Table 7.1 Hwy 14X:02 km 17.9 to
Alternative 1, overla

station	overlay (mm)
100.060	72
100.100	84
100.140	78
100.180	171
100.220	75
100.260	68
100.300	215
100.340	90
100.380	59
100.420	91
Mean	100
St. dev.	51
Design thick.	151

Table 7.2 Hwy 14X:02 km 17.9 to
Alternative 3, overla

station	overlay (mm)
100.060	305
100.100	305
100.140	315
100.180	336
100.220	313
100.260	279
100.300	315
100.340	312
100.380	310
100.420	320
Mean	311
St. dev.	14
Design thick.	325

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CHAPTER 8

SUMMARY, CONCLUSIONS, RECOMMENDATIONS

8.1 Summary

The main purpose of this research was to provide guidelines for rehabilitation of full-depth asphalt concrete pavements in Alberta. To achieve this more detailed objectives had to be fulfilled. First of all, experiences with the performance of full-depth pavements in seasonal frost areas, including the Province of Alberta, were reviewed. A comprehensive literature search indicates that full-depth asphalt concrete pavements are, generally, performing satisfactorily.

In order to properly evaluate the structural abilities of existing pavements, and full-depth pavements in particular, elastic moduli of each pavement layer were determined. The moduli can be obtained either in a destructive or non-destructive way. The non-destructive techniques, for obvious reasons, are gaining widespread acceptance. Pavement responses i.e. stresses, strains, and deflections can be calculated based on the theory of elasticity. The moduli backcalculation procedures using pavement deflection basins developed under a load, are gaining acceptance among pavement engineers and scientists. To obtain not one but many pavement deflections in each load location, deflectiontesting devices such as Dynaflect or the Falling Weight Deflectometer have been developed.

In this investigation two backcalculation computerized procedures were described. These are the ELMOD and FWDUT1S computer programs. These two procedures were applied to numerous deflection basins determined along selected full-depth pavements in the Province of Alberta. Both these procedures predicts reasonably well the asphalt concrete and the subgrade moduli of the selected full-depth AC pavements. It was found that ELMOD predicts lower subgrade moduli and higher asphalt concrete moduli when compared with FWDUT1S. Taking the ELMOD calculated moduli as 100, the FWDUT1S asphalt concrete moduli are, on average, 9 percent lower, and the FWDUT1S subgrade moduli, on average, 17 percent higher than these obtained using ELMOD.

The layer calculated moduli can be utilized in mechanistic-empirical procedures for estimation of pavement lives. In this study three existing semi-mechanistic methods were described and one method based on the DAMA computer program was developed.

The described backcalculation approach and the semi-analytical models were applied to five full-depth pavements selected from the Alberta Highway Network. The performed procedure utilized the ELMOD calculated moduli as an input, and the DAMA based semi-mechanistic procedure. Two different fatigue criteria were incorporated into the

DAMA program. A comparison of overlay thicknesses obtained for the selected pavements using four different design approaches was performed. These approaches are: ELMOD, DAMA with two different fatigue criteria and the RTAC Benkelman-beam deflection based method. It was found that the calculated thicknesses varied considerably from method to method.

In the last part of the thesis one full-depth asphalt concrete pavement was selected. Three rehabilitation measures were analyzed using the DAMA based pseudo-analytical model. Required thicknesses of the pavement layers were calculated and initial construction costs related to each rehabilitation alternative were computed. Some guidelines for rehabilitation of full-depth pavements were provided. The guidelines emphasize a type of data required and methods of the pavement evaluation to be used. Three rehabilitation methods appropriate for full-depth pavements were discussed. These are: overlay, reconstruction, and partial cold-milling and overlay. This investigation indicates that the latter method may be, in some cases, considered as a cost-effective one.

8.2 Conclusions

The following conclusions can be drawn from this study:

1. Full-depth asphalt concrete pavements appear to perform well provided they are properly designed and constructed. These pavements should be designed using distress criteria adequate to materials used. When full-depth pavements are constructed, a special attention should be paid to:

a. The subgrade proper preparation. The subgrade should be scraped, mixed, blended and adequately compacted to provide uniformity throughout a section to prevent differential heaving which may be harmful for full-depth pavements.

b. The pavements should be laid in thick lifts to prevent early development of low-temperature transverse cracking.

c. Asphalt concrete mix should be designed to provide its high stability. To achieve this a high proportion of crushed aggregate should be used. The last statement is especially true in the case of very heavily loaded roads.

d. Great attention should be paid to obtaining compaction of the mix in order to achieve high density and low amount of air voids.

2. Different methods for moduli backcalculation may produce different moduli. Care should be taken to see that the obtained moduli are reasonable and correlate with the laboratory obtained moduli, if available. It appears that

the ELMOD and FWDUT1S programs are suitable for backcalculation of pavement layer moduli. The moduli values are reasonable and close to the laboratory obtained moduli.

3. Both the ELMOD and DAMA based overlay procedures appear to have potential to reasonably predict necessary overlay thicknesses. An advantage of ELMOD is its great efficiency. Asphalt concrete fatigue criteria are the most significant factors that influence the pavement design life. Many different fatigue criteria are presently used and attention should be paid to utilize criterion proper to materials encountered.

4. The DAMA based overlay procedure may be used for analysis of various rehabilitation measures. It appears that this semi-analytical design procedure is suitable for application to pavement overlay design.

5. It appears that partial cold-milling and subsequent overlay may be considered as a cost-effective method for rehabilitation of thick full-depth asphalt concrete pavements.

8.3 Recommendations

1. Both ELMOD and FWDUT1S seem to be suitable for the pavement moduli backcalculation. However, further verification using several pavement sections should be performed before the final conclusion is drawn in this respect. Selection of the most suitable software should be based on the reasonableness of its results, least complexity and efficiency.

2. The DAMA based overlay design procedure seems to be a suitable tool for design of rehabilitation measures, but should be combined in one computer program.

3. Further investigation should be carried out to decide which asphalt concrete fatigue criterion is the most appropriate for the Alberta conditions and materials used.

4. The subgrade seasonal strength variations should be further investigated. The Falling Weight Deflectometer and a backcalculation method appear to be promising tools for such an investigation.

5. All moduli backcalculation techniques are very sensitive to the asphalt concrete temperatures and thicknesses of the pavement layers. Further, comprehensive, research is needed to find a proper temperature correction method for the

backcalculated moduli.

At present all sections having consistently higher modulus values than other sections of the pavement should be cored to define the layer thicknesses and material encountered. Development of a device or a method able non-destructively determine the pavement layer thicknesses would be of great value.

6. The concept of the Structural Adequacy Index as used in Alberta for full-depth pavements should be further evaluated.

7. Further research is needed to validate the newly introduced concept of thick full-depth pavement rehabilitation technique i.e. partial cold-milling and overlay.

APPENDIX A

EXAMPLE RUN OF FWDUT1S

M A P C O N

Some original routines by: The Pennsylvania Transportation Institute.
Updated, expanded and enhanced by: Austin Research Engineers, Austin, Tx.
Enhanced screen interaction by: SRA Technologies, Inc., Alexandria, VA.

Selection:

MAPCON
- 1. Safety Analysis, SAFE
- 2. Serviceability Analysis
- 3. Structural Capacity Analysis
- 4. Surface Condition Analysis
- 5. Surface Condition, Structural Capacity and Serviceability combined, 4CF
- 0. Exit - Return to DOS

M A F C O N

Structural Analysis

```
SELECTION:
                                      ┌-1. ELSYM5 Elastic Layer Theory Program
 [    ]            ┌-Modulus Data-----┤
                   │ Values           └-2. HCF Elastic Layer Theory Regression
                   │                           Equations Program
                   │                  ┌-Dynaflect Data----------3. additional braches
 Structural        │                  │
 Analysis----------┤                  │                    ┌-4. FWDUTIS Basin Matching
                   │                  ├-Falling Weight:----┼-5. FWDUTIS Basin Matching
                   │                  │ Deflectometer      │     -with FATLIF analysis
                   │                  │                    └-6. GENDEF Program*
  0.<-             │                  │                    ┌-7. RRFITIS Basin Matching
  Return to        └-Deflection-------┼-Road Rater Model---┼-8. RRFITIS Basin Matching
  MAFCON Menu        Data             │ 400,2000,or 2008   │     -with FATLIF analysis
                                      │                    └-9. GENDEF Program*
                                      └-Other Device-----------10. GENDEF Program*
                                                  *Standard deflection basin statistics
```

```
BASIN FITTING FOR  F W D  DEFLECTION DEVICE
FWDUTIS (VERSION 1.1 : 27 MARCH 1985) BY ARE INC.

IF YOU WANT TO STOP THE PLOT BEFORE IT GOES OFF
THE SCREEN, PRESS CONTROL-S.  TO START THE SCREEN
MOVING AGAIN, PRESS CONTROL-Q.

Enter Output DATA Filename [Default - FATLIF.DAT     ] -

Enter Output Filename (or device) [Default = FWDUTIS.OUT    ] -

BEGIN  F W D  DATA ENTRY.....

press ANY KEY to continue
```

DAFCON: FWD011S   Basin Fitting Program for FWD Devices

```
Enter peak force applied by the FWD:     9000.00     lbs.

Loading plate radius:    5.91          in.

Number of Sensors used on FWD: 7

Array of radial points where results are desired-
(radial distance of geophone location from the load, inches)

 1       .00
 2     12.00
 3     18.00
 4     24.00
 5     36.00
 6     48.00
 7     60.00
```

F1: Modify this screen; F2: Continue with next screen:

MAPCON: FNDUT1S - Basin Fitting Program for FWD Devices

Test Location Title (25 characters):

[857:04 sta. 500.100,   ]

Measured Deflection Basin (in mils)-

Sensor No.	Measured Deflection
1	12.7200
2	9.8000
3	7.9500
4	6.4600
5	4.2100
6	2.9500
7	2.2400

Number of Layers (2 to 6): 2

Layer No.	Poisson's Ratio
1	.3500
2	.4500

F1: Modify this screen; F2: Go to next screen; F3: Modify previous screen;

MnPCON: FWDUTIS Basin Fitting Program for FWD Devices

Surface type for section: 2 1 = Flexible (surface treated) 2 = Flexible (asphalt concrete) 3 = Rigid (jointed or jointed reinforced) 4 = Rigid (continuous reinf. PCC) 5 = Overlaid or other	Subgrade type for section: 3 1 = Clay 2 = Silty clay 3 = Clayey silt 4 = Sandy silt 5 = Sand 6 = Gravelly sand
(Flexible and No. of Layers >2 only) Base type for pavement: 1 = Granular 2 = Asphalt or bituminous stabilized 3 = Soil cement, cement treated or stabilized material	Subgrade quality with regard to stress sensitivity: 2 1 = Good 2 = Fair 3 = Poor

F1: Modify this screen; F2: Go to next screen; F3: Modify previous screen:

MAPCON: FWDDFIS Basin Fitting Program for FWD Device

Number of Layers= 2

Layer No.	Thickness (inches)	Young's (Elastic) Modulus (psi)
1	7.00	750955.
2	-	16408.

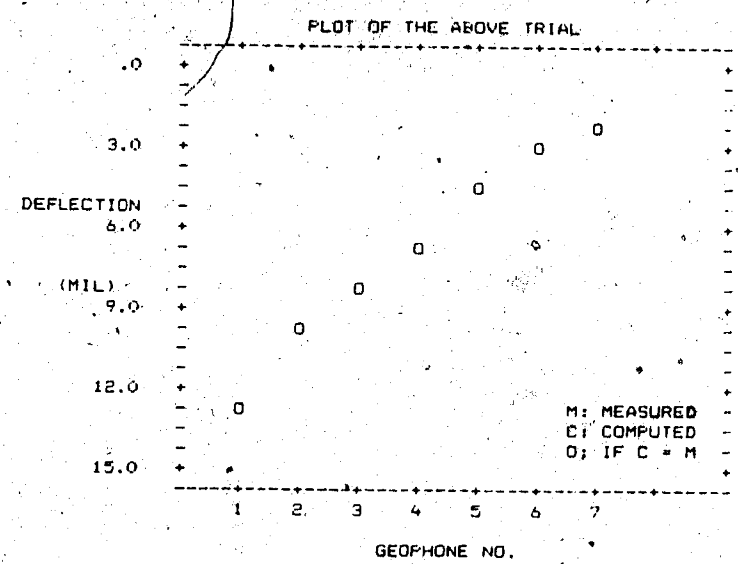
Interface between Subgrade and Semi-infinite Rock Layer:

1 = Full Friction (Rough).
2 = Perfectly Smooth

F1: Modify this screen; F2: Perform Analysis; F3: Modify previous screen

```
PROBLEM NUMBER :  1
PEAK FORCE (LBS) :     9000.00
DO YOU WANT A PLOT OF DEFLECTION BASINS ON SCREEN (Y/N)?
(THE PLOTS WILL SHOW ONLY SENSOR NO. ON THE ABSCISSA)

PLEASE WAIT FOR LAYER ANALYSIS TO FINISH.....

READY TO SHOW RESULTS.
WHEN SCROLLING PRESS CONTROL-S TO STOP SCROLL AND CONTROL-Q TO BEGIN AGAIN.
PRESS ENTER TO SEE RESULTS.
```

```
FWDUTIS --  F W D  DEFLECTION BASIN FITTING PROGRAM
CENTER FOR TRANSPORTATION RESEARCH, U.T. AUSTIN,TEXAS
VERSION 1.1 : 27 MARCH 1985 BY ARE INC, 2600 DELLANA LANE, AUSTIN, TX

PROBLEM NUMBER :  1
PEAK FORCE (LBS) :    [illegible]
TEST LOCATION OF MEASURED [illegible]CTION BASIN : 057:04 sta. 500.100

TRIAL NO.   1
THICKNESSES(IN):        [illegible]
POISSONS RATIOS:         .3500      .4500
E.MODULI (PSI):        750955.     16408.
--> COMPUTED DEF. (MILS):      12.50      9.57      7.94      6.54      4.48
--> COMPUTED DEF. (MILS):       3.21      2.45
MEASURED DEF. (MILS):       12.72      9.80      7.95      6.46      4.21
MEASURED DEF. (MILS):        2.95      2.24
```



APPENDIX B

STATISTICAL COMPARISON OF ASPHALT CONCRETE AND SUBGRADE MODULI OBTAINED USING FWDUT1S AND ELMOD COMPUTER PROGRAMS

GENERAL ASSUMPTIONS

1) Data obtained using both programs are normally distributed

2) Data are paired i.e. paires of moduli obtained with the use of the two programs are truly correlated

3) Null hypothesis for asphalt moduli is formulated as follows: asphalt moduli obtained usig the two different methods of moduli backcalculation are not significantly different. $E_{1E} = E_{1F}$

4) Alternative hypothesis for asphalt moduli is as follows: asphalt moduli predicted using FWDUT1S computer program are significantly lower than these obtained using ELMOD computer program. $E_{1F} < E_{1E}$

5) It is seen from the above that the statistical test is one-sided.

6) Null hypothesis for subgrade moduli is as follows: subgrade moduli predicted using both programs are not significantly different. $E_{2E} = E_{2F}$

7) Alternative hypothesis for subgrade moduli is as follows: subgrade moduli obtained with ELMOD are significantly lower than subgrade moduli obtained with FWDUT1S. $E_{2E} < E_{2F}$

8) Again it is seen that this statistical test is one-sided.

9) The t-test for paired data will be used.

PROCEDURE

$$x_2' = \Sigma x_2/n_2$$

$$x_1' = \Sigma x_1/n_1$$

$$y = x_2 - x_1$$

$$y' = \Sigma y/n$$

standard deviation of y is:

$$s = (\Sigma(y - y')^2/(n-1))^{0.5}$$

The standard deviation of y' is:

$$s_{y'} = s/\sqrt{n}$$

The t-test is applied to y' by comparing it to a zero mean difference:

$$t = |\ y' - 0\ |/s_{y'}$$

The number of degree of freedom is determined as follows: $v = n-1$ and tabulated t value is read off for an appropriate level of significance. If $t_{calc} > t_{tab}$ the difference is statistically significant.

HIGHWAY 14X:02

A. ASPHALT MODULI

$n = 20;\ v = 20 - 1 = 19;$

$y' = 338 \qquad s = 300$

$s_{y'} = 300/\sqrt{20} = 67.08$

$t_{calc} = 338/67.08 = 5.04$

$t_{tab} = 3.883$ with probability of 0.0005

<u>Conclusion</u> Asphalt moduli obtained using FWDUT1S are lower that these obtained using ELMOD with the level of significance 0.0005.

B. SUBGRADE MODULI

$n = 20\ ;\ v = 19;$

$y' = 30 \qquad s = 24$

$s_{y'} = 24/\sqrt{20} = 5.37$

$t_{calc} = 30/5.37 = 5.59$

$t_{tab} = 3.883$ with probability of 0.0005

<u>Conclusion</u> Subgrade moduli obtained using ELMOD are lower that these obtained using FWDUT1S with the level of significance 0.0005.

HIGHWAY 36:18

A. ASPHALT MODULI

$n = 13;\ v = 13 - 1 = 12;$

$y' = 327 \qquad s = 538$

$s_{y'} = 538/\sqrt{13} = 149.21$

$t_{calc} = 327/149.21 = 2.19$

$t_{tab} = 2.179$ with probability of 0.025

<u>Conclusion</u> Asphalt moduli obtained using FWDUT1S are lower that these obtained using ELMOD with the level of significance 0.025.

B. SUBGRADE MODULI

$n = 13\ ;\ v = 12;$

$y' = 9 \qquad s = 12$

$s_{y'} = 12/\sqrt{13} = 3.33$

$t_{calc} = 9/3.33 = 2.70$

$t_{tab} = 2.179$ with probability of 0.025

<u>Conclusion</u> Subgrade moduli obtained using ELMOD are lower that these obtained using FWDUT1S with the level of significance 0.025.

HIGHWAY 45:06A

A. ASPHALT MODULI

$n = 25; v = 25 - 1 = 24;$

$y' = 584 \qquad s = 748$

$s_{y'} = 748/\sqrt{25} = 149.6$

$t_{calc} = 548/149.6 = 3.90$

$t_{tab} = 3.745$ with probability of 0.0005

Conclusion Asphalt moduli obtained using FWDUT1S are lower that these obtained using ELMOD with the level of significance 0.0005.

B. SUBGRADE MODULI

$n = 25 ; v = 24;$

$y' = 15 \qquad s = 15$

$s_{y'} = 15/\sqrt{25} = 3.00$

$t_{calc} = 15/3.00 = 5.00$

$t_{tab} = 3.745$ with probability of 0.0005

Conclusion Subgrade moduli obtained using ELMOD are lower that these obtained using FWDUT1S with the level of significance 0.0005.

HIGHWAY 45:06B

A. ASPHALT MODULI

$n = 19;\ v = 19 - 1 = 18;$

$y' = 579 \qquad s = 566$

$s_{y'} = 566/\sqrt{19} = 129.85$

$t_{calc} = 579/129.85 = 4.46$

$t_{tab} = 3.992$ with probability of 0.0005

<u>Conclusion</u> Asphalt moduli obtained using FWDUT1S are lower that these obtained using ELMOD with the level of significance 0.0005.

7.

B. SUBGRADE MODULI

$n = 19;\ v = 18;$

$y' = 16 \qquad s = 12$

$s_{y'} = 12/\sqrt{19} = 2.75$

$t_{calc} = 16/2.75 = 5.81$

$t_{tab} = 3.992$ with probability of 0.0005

<u>Conclusion</u> Subgrade moduli obtained using ELMOD are lower that these obtained using FWDUT1S with the level of significance 0.0005.

HIGHWAY 857:04

A. ASPHALT MODULI

$n = 69$; $v = 69 - 1 = 68$;

$y' = 125$ $\quad$ $s = 412$

$s_{y'} = 412/\sqrt{69} = 49.60$

$t_{calc} = 125/49.60 = 2.52$

$t_{tab} = 2.0$ with probability of 0.025

Conclusion Asphalt moduli obtained using FWDUT1S are lower that these ootained using ELMOD with the level of significance 0.025.

B. SUBGRADE MODULI

$n = 69$; $v = 68$;

$y' = 12$ $\quad$ $s = 8$

$s_{y'} = 8/\sqrt{69} = 0.96$

$t_{calc} = 12/0.96 = 12.46$

$t_{tab} = 3.46$ with probability of 0.0005

Conclusion Subgrade moduli obtained using ELMOD are lower that these obtained using FWDUT1S with the level of significance 0.0005.

ALL HIGHWAYS TESTED

A. ASPHALT MODULI

$n = 146;\ v = 146 - 1 = 145;$

$y' = 310 \qquad s = 535$

$s_{y'} = 535/\sqrt{146} = 44.28$

$t_{calc} = 310/44.28 = 7.00$

$t_{tab} = 3.291$ with probability of 0.0005

Conclusion Asphalt moduli obtained using FWDUT1S are lower that these obtained using ELMOD with the level of significance 0.0005.

B. SUBGRADE MODULI

$n = 146\ ;\ v = 145;$

$y' = 15 \qquad s = 14$

$s_{y'} = 14/\sqrt{146} = 1.16$

$t_{calc} = 15/1.16 = 12.95$

$t_{tab} = 3.291$ with probability of 0.0005

Conclusion Subgrade moduli obtained using ELMOD are lower that these obtained using FWDUT1S with the level of significance 0.0005.

APPENDIX C

TRAFFIC CALCULATIONS

HIGHWAY 14X:02

Estimated traffic growth i=0.045 a year.

Cumulative no. of ESAL to 1984 3438*365*0.85 = 1,066,640

Cumulative no. of ESAL to 1985 4281*365*0.85 = 1,328,180

ESAL/day (1,328,180-1,066,640)/365 = 716.5 ESAL/day

ESTIMATED TRAFFIC IN 1987

Used formula:

$$(ESAL/day)*365*0.85*[(1+i)^{n}-1]/\ln(1+i) + (ESAL\ in\ 1985)$$

where .85 is a design lane factor (only for multilane roads

n - design period in years

i - yearly traffic growth

$$716.5*365*.85*[(1+0.045)^{2}-1]/\ln(1+0.045)+1,328,180 =$$

1,792,925

ESTIMATED TRAFFIC IN 2002

$$716.5*365*.85*[(1+0.045)^{17}-1]/\ln(1+0.045)+1.328,180 =$$

6,950,957

ESTIMATED TRAFFIC IN THE DESIGN PERIOD 1987-2002

6,950,957-1,792,925 = 5,158,032

ESAL/month 5,158,032/12*15 = 28656

HIGHWAY 36:18

Estimated traffic growth i=0.04 a [illegible]r.

Cumulative no. of ESAL to 1985 [illegible]365 = 145,270

Cumulative no. of ESAL to 1986 497*365 = 181,405

ESAL/day (145,270-181,405)/365 = 99 ESAL/day

ESTIMATED TRAFFIC IN 1987

Used formula:

$(ESAL/day)*365*[(1+i)^{n}-1]/\ln(1+i) + (ESAL\ in\ 1986)$

where

n - design period in years

i - yearly traffic growth

$99*365*[(1+0.04)^{1}-1]/\ln(1+0.04)+181,405 \doteq 218,258$

ESTIMATED TRAFFIC IN 2002

$99*365*[(1+0.04)^{16}-1]/\ln(1+0.04)+181,405 = 985,704$

ESTIMATED TRAFFIC IN THE DESIGN PERIOD 1987-2002

985,704-218,258 = 767,446

ESAL/month 767,446/12*15 = 4264

HIGHWAY 45:06A

Estimated traffic growth i=0.04 a year.

Cumulative no. of ESAL to 1984   542*365 = 197,830

Cumulative no. of ESAL to 1985   602*365 = 219,730

ESAL/day   (219,730-197,830)/365 = 60 ESAL/day

ESTIMATED TRAFFIC IN 1987

Used formula:

$(ESAL/day)*365*[(1+i)^{n}-1]/\ln(1+i) + (ESAL\ in\ 1985)$

where

n - design period in years

i - yearly traffic growth

$60*365*[(1+0.04)^{2}-1]/\ln(1+0.04)+219{,}730 = 265{,}294$

ESTIMATED TRAFFIC IN 2002

$60*365*[(1+0.04)^{17}-1]/\ln(1+0.04)+219{,}730 = 749{,}017$

ESTIMATED TRAFFIC IN THE DESIGN PERIOD 1987-2002

749,017-219,730 = 483,732

ESAL/month   483,732/12*15 = 2687

HIGHWAY 45:06B

Estimated traffic growth i=0.04 a year.

Cumulative no. of ESAL to 1984   544*365 = 198,560

Cumulative no. of ESAL to 1985   604*365 = 220,460

ESAL/day   (220,460-198,560)/365 = 60 ESAL/day

ESTIMATED TRAFFIC IN 1987

Used formula is

$(ESAL/day)*365*[(1+i)^{n}-1]/\ln(1+i) + (ESAL\ in\ 1985)$

where

n - design period in years

i - yearly traffic growth

$60*365*[(1+0.04)^{2}-1]/\ln(1+0.04)+220,460 = 266,024$

ESTIMATED TRAFFIC IN 2002

$60*365*[(1+0.04)^{17}-1]/\ln(1+0.04)+220,460 = 749,747$

ESTIMATED TRAFFIC IN THE DESIGN PERIOD 1987-2002

749,747-266,024 = 483,732

ESAL/month   483,732/12*15 = 26[illegible]7

HIGHWAY 857:04

Estimated traffic growth i=0.04 a year.

Cumulative no. of ESAL to 1985 107*365 = 39,055

Cumulative no. of ESAL to 1986 124*365 = 45,260

ESAL/day (45,260-39,055)/365 = 17 ESAL/day

ESTIMATED TRAFFIC IN 1987

Used formula:

$$(ESAL/day)*365*[(1+i)^{n}-1]/\ln(1+i) + (ESAL\ in\ 1986)$$

where

n - design period in years

i - yearly traffic growth

$$17*365*[(1+0.04)^{1}-1]/\ln(1+0.04)+45,260 = 51,588$$

ESTIMATED TRAFFIC IN 2002

$$17*365*[(1+0.04)^{16}-1]/\ln(1+0.04)+45,260 = 183,372$$

ESTIMATED TRAFFIC IN THE DESIGN PERIOD 1987-2002

183,372-45,260 = 138,112

ESAL/month 138,112/12*15 = 767

APPENDIX D

EXAMPLE RUN OF SELECTED PROCEDURE FOR OVERLAY DESIGN

45:06A sta. 800.340 run A

LAYER AND MATERIAL PROPERTIES

LAYER NUMBER	MATERIAL TYPE	POISSON'S RATIO	THICKNESS (IN.)
1	ASPH. CONC.	0.35	6.00
2	SUBGR. SOIL	0.45	

CURING CONDITIONS

LAYER NUMBER	MATERIAL TYPE	CURE TIME (MONTHS)	MONTH OPENED TO TRAFFIC	MONTHS CURED BEFORE OPENING
1	ASPH. CONC.	0.0	SEPT.	0

TRAFFIC CONDITIONS

NUMBER OF REPETITIONS PER MONTH = 1730

ENVIRONMENTAL CONDITIONS
(MEAN MONTHLY AIR TEMPERATURES, DEG. F)

JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
-1.1	8.1	16.9	31.8	52.0	58.8	62.6	58.7	50.2	39.7	22.3	7.2

LOAD CONFIGURATION AND COMPUTATIONAL POINTS

LOAD PER TIRE = 4500. LBS
CONTACT PRESSURE = 95.00 PSI
RADIUS OF LOAD = 3.88 IN.
LOAD SPACING = 13.50 IN.

COMPUTATIONAL POINT 1 X = 0.0 IN. (CENTER OF ONE TIRE)
COMPUTATIONAL POINT 2 X = 3.88 IN. (EDGE OF ONE TIRE)
COMPUTATIONAL POINT 3 X = 6.75 IN. (MIDPOINT OF TWO TIRES)

MODULI CONDITIONS

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
1	ASPH.CONC.				
		1	1.0	628910	
		2	9.0	604368	
		3	17.0	562217	
		4	25.0	483666	
		5	33.0	407407	
		6	41.0	330746	

7	48.0	268206
8	57.0	196343
9	65.0	143898
10	73.0	102127
11	81.0	70241
12	89.0	46849

MODULI CONDITIONS

SUBGRADE LAYER

LAYER NUMBER	MATERIAL TYPE	MONTH	MODULUS
2	SUBGR.SOIL		
		JAN.	50000
		FEB.	50000
		MAR.	50000
		APRIL	6786
		MAY	6786
		JUNE	8352
		JULY	9914
		AUG.	11484
		SEPT.	13050
		OCT.	13050
		NOV.	31625
		DEC.	50000

***************DESIGN LIFE OF PAVEMENT****************

LAYER	DAMAGE TYPE	CUMULATIVE DAMAGE	CRITICAL POSITION	DESIGN LIFE (YEARS)	DESIGN REPETITIONS
1	FATIGUE	1.000	1	3.6	0.7221E+05
2	DEFORMATION	1.000	1	1.5	0.3123E+05

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LAYER AND MATERIAL PROPERTIES

LAYER NUMBER	MATERIAL TYPE	POISSON'S RATIO	THICKNESS (IN.)
1	ASPH. CONC.	0.35	1.00
2	ASPH. CONC.	0.35	6.00
3	SUBGR. SOIL	0.45	

CURING CONDITIONS

LAYER NUMBER	MATERIAL TYPE	CURE TIME (MONTHS)	MONTH OPENED TO TRAFFIC	MONTHS CURED BEFORE OPENING

1	ASPH. CONC.	0.0	SEPT.	0
2	ASPH. CONC.	0.0	SEPT.	0

TRAFFIC CONDITIONS

NUMBER OF REPETITIONS PER MONTH 2435

ENVIRONMENTAL CONDITIONS
(MEAN MONTHLY AIR TEMPERATURES, DEG. F)

JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
-1.1	8.1	16.9	31.8	52.0	58.8	62.6	58.7	50.2	39.7	22.3	7.2

LOAD CONFIGURATION AND COMPUTATIONAL POINTS

LOAD PER TIRE = 4500. LBS
CONTACT PRESSURE = 95.00 PSI
RADIUS OF LOAD = 3.88 IN.
LOAD SPACING = 13.50 IN.

COMPUTATIONAL POINT 1 X = 0.0 IN. (CENTER OF ONE TIRE)
COMPUTATIONAL POINT 2 X = 3.88 IN. (EDGE OF ONE TIRE)
COMPUTATIONAL POINT 3 X = 6.75 IN. (MIDPOINT OF TWO TIRES)

MODULI CONDITIONS

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
1	ASPH.CONC.				
		1	1.0	2049568	
		2	9.0	1963566	
		3	17.0	1788372	
		4	25.0	1568898	
		5	33.0	1305203	
		6	41.0	1051786	
		7	48.0	817164	
		8	57.0	612898	
		9	65.0	444250	
		10	73.0	311468	
		11	81.0	211381	
		12	89.0	138974	

MODULUS PARAMETERS: NU = 0.150 P200 = 6.0 FREQ = 10 HZ VV = 7.0 VB = 13.1

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
2	ASPH.CONC.				
		1	1.0	628910	
		2	9.0	604368	
		3	17.0	562217	
		4	25.0	483666	
		5	33.0	407407	
		6	41.0	330746	
		7	48.0	268206	
		8	57.0	196343	
		9	65.0	143898	
		10	73.0	102127	
		11	81.0	70241	
		12	89.0	46849	

MODULI CONDITIONS

SUBGRADE LAYER

LAYER NUMBER	MATERIAL TYPE	MONTH	MODULUS
3	SUBGR.SOIL		
		JAN.	50000
		FEB.	50000
		MAR.	50000
		APRIL	6786
		MAY	6786
		JUNE	8352
		JULY	9914
		AUG.	11484
		SEPT.	13050
		OCT.	13050
		NOV.	31625
		DEC.	50000

***************DESIGN LIFE OF PAVEMENT***

LAYER	DAMAGE TYPE	CUMULATIVE DAMAGE	CRITICAL POSITION	DESIGN LIFE (YEARS)	DESIGN REPETITIONS
1	FATIGUE	1.000	3	183.6	0.5657E+07
2	FATIGUE	1.000	2	5.9	0.1729E+06
3	DEFORMATION	1.000	1	4.0	0.1176E+06

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LAYER AND MATERIAL PROPERTIES

LAYER NUMBER	MATERIAL TYPE	POISSON'S RATIO	THICKNESS (IN.)
1	ASPH. CONC.	0.35	3.00
2	ASPH. CONC.	0.35	6.00
3	SUBGR. SOIL	0.45	

CURING CONDITIONS

LAYER NUMBER	MATERIAL TYPE	CURE TIME (MONTHS)	MONTH OPENED TO TRAFFIC	MONTHS CURED BEFORE OPENING
1	ASPH. CONC.	0.0	SEPT.	0
2	ASPH. CONC.	0.0	SEPT.	0

TRAFFIC CONDITIONS

NUMBER OF REPETITIONS PER MONTH 2435

ENVIRONMENTAL CONDITIONS
(MEAN MONTHLY AIR TEMPERATURES, DEG. F)

JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
-1.1	8.1	16.9	31.8	52.0	58.8	62.6	58.7	50.2	39.7	22.3	7.2

LOAD CONFIGURATION AND COMPUTATIONAL POINTS

LOAD PER TIRE = 4500. LBS
CONTACT PRESSURE = 95.00 PSI
RADIUS OF LOAD = 3.88 IN.
LOAD SPACING = 13.50 IN.

COMPUTATIONAL POINT 1 X = 0.0 IN. (CENTER OF ONE TIRE)
COMPUTATIONAL POINT 2 X = 3.88 IN. (EDGE OF ONE TIRE)
COMPUTATIONAL POINT 3 X = 6.75 IN. (MIDPOINT OF TWO TIRES)

MODULI CONDITIONS

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
1	ASPH.CONC.				
		1	1.0	2049568	
		2	9.0	1963566	
		3	17.0	1788372	
		4	25.0	1568898	
		5	33.0	1305203	

6	41.0	1051786
7	48.0	817164
8	57.0	612898
9	65.0	444250
10	73.0	311468
11	81.0	211381
12	89.0	138974

MODULUS PARAMETERS: NU = 0.150   P200 = 6.0   FREQ = 10 HZ   VV = 7.0

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
2	ASPH.CONC.				
		1	1.0	628910	
		2	9.0	604368	
		3	17.0	562217	
		4	25.0	483666	
		5	33.0	407407	
		6	41.0	330746	
		7	48.0	268206	
		8	57.0	196343	
		9	65.0	143898	
		10	73.0	102127	
		11	81.0	70241	
		12	89.0	46849	

MODULI CONDITIONS

SUBGRADE LAYER

LAYER NUMBER	MATERIAL TYPE	MONTH	MODULUS
3	SUBGR.SOIL		
		JAN.	50000
		FEB.	50000
		MAR.	50000
		APRIL	6786
		MAY	6786
		JUNE	8352
		JULY	9914
		AUG.	11484
		SEPT.	13050
		OCT.	13050
		NOV.	31625
		DEC.	50000

******************DESIGN LIFE OF PAVEMENT**

LAYER	DAMAGE TYPE	CUMULATIVE DAMAGE	CRITICAL POSITION	DESIGN LIFE (YEARS)	DESIGN REPETITIONS
1	FATIGUE	1.000	1	361.6	0.1116E+08
2	FATIGUE	1.000	3	21.1	0.6168E+06
3	DEFORMATION	1.000	1	29.6	0.8658E+06

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LAYER AND MATERIAL PROPERTIES

LAYER NUMBER	MATERIAL TYPE	POISSON'S RATIO	THICKNESS (IN.)
1	ASPH. CONC.	0.35	5.00
2	ASPH. CONC.	0.35	6.00
3	SUBGR. SOIL	0.45	

CURING CONDITIONS

LAYER NUMBER	MATERIAL TYPE	CURE TIME (MONTHS)	MONTH OPENED TO TRAFFIC	MONTHS CURED BEFORE OPENING
1	ASPH. CONC.	0.0	SEPT.	0
2	ASPH. CONC.	0.0	SEPT.	0

TRAFFIC CONDITIONS

NUMBER OF REPETITIONS PER MONTH 2435

ENVIRONMENTAL CONDITIONS
(MEAN MONTHLY AIR TEMPERATURES, DEG. F)

JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
-1.1	8.1	16.9	31.8	52.0	58.8	62.6	58.7	50.2	39.7	22.3	7.2

LOAD CONFIGURATION AND COMPUTATIONAL POINTS

LOAD PER TIRE = 4500. LBS
CONTACT PRESSURE = 95.00 PSI
RADIUS OF LOAD = 3.88 IN.
LOAD SPACING = 13.50 IN.

COMPUTATIONAL POINT 1 X = 0.0 IN. (CENTER OF ONE TIRE)
COMPUTATIONAL POINT 2 X = 3.88 IN. (EDGE OF ONE TIRE)
COMPUTATIONAL POINT 3 X = 6.75 IN. (MIDPOINT OF TWO TIRES)

MODULI CONDITIONS

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
1	ASPH.CONC.				
		1	1.0	2049568	
		2	9.0	1963566	
		3	17.0	1788372	
		4	25.0	1568898	
		5	33.0	1305203	
		6	41.0	1051786	
		7	48.0	817164	
		8	57.0	612898	
		9	65.0	444250	
		10	73.0	311468	
		11	81.0	211381	
		12	89.0	138974	

MODULUS PARAMETERS: MU = 0.150   P200 = 6.0   FREQ = 10 HZ   VV = 7.0

ASPHALT STABILIZED LAYER

LAYER NUMBER	MATERIAL TYPE	POINT NUMBER	TEMP. (DEG. F)	MODULUS EI (PSI)	MODULUS EF (PSI)
2	ASPH.CONC.				
		1	1.0	628910	
		2	9.0	604368	
		3	17.0	562217	
		4	25.0	483666	
		5	33.0	407407	
		6	41.0	330746	
		7	48.0	268206	
		8	57.0	196343	
		9	65.0	143898	
		10	73.0	102127	
		11	81.0	70241	
		12	89.0	46849	

MODULI CONDITIONS

SUBGRADE LAYER

LAYER NUMBER	MATERIAL TYPE	MONTH	MODULUS
3	SUBGR.SOIL		
		JAN.	50000
		FEB.	50000
		MAR.	50000
		APRIL	6786

MAY	6786
JUNE	8352
JULY	9914
AUG.	11484
SEPT.	13050
OCT.	13050
NOV.	31625
DEC.	50000

***************DESIGN LIFE OF PAVEMENT**

LAYER	DAMAGE TYPE	CUMULATIVE DAMAGE	CRITICAL POSITION	DESIGN LIFE (YEARS)	DESIGN REPETITIONS
1	FATIGUE	1.000	1	438.7	0.1282E+08
2	FATIGUE	1.000	3	65.8	0.1921E+07
3	DEFORMATION	1.000	2	141.2	0.4125E+07

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Overlay vs load repetitions

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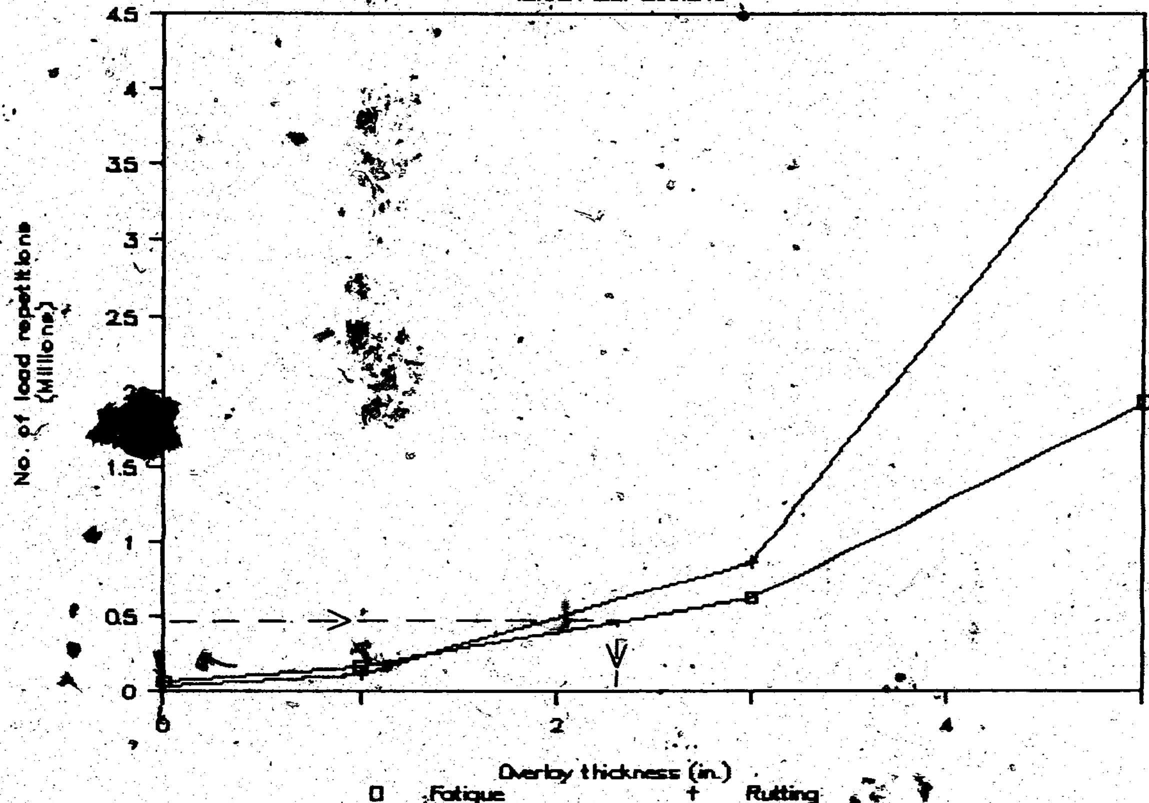

APPENDIX E

PHOTOGRAPHS OF SELECTED PAVEMENT SECTIONS

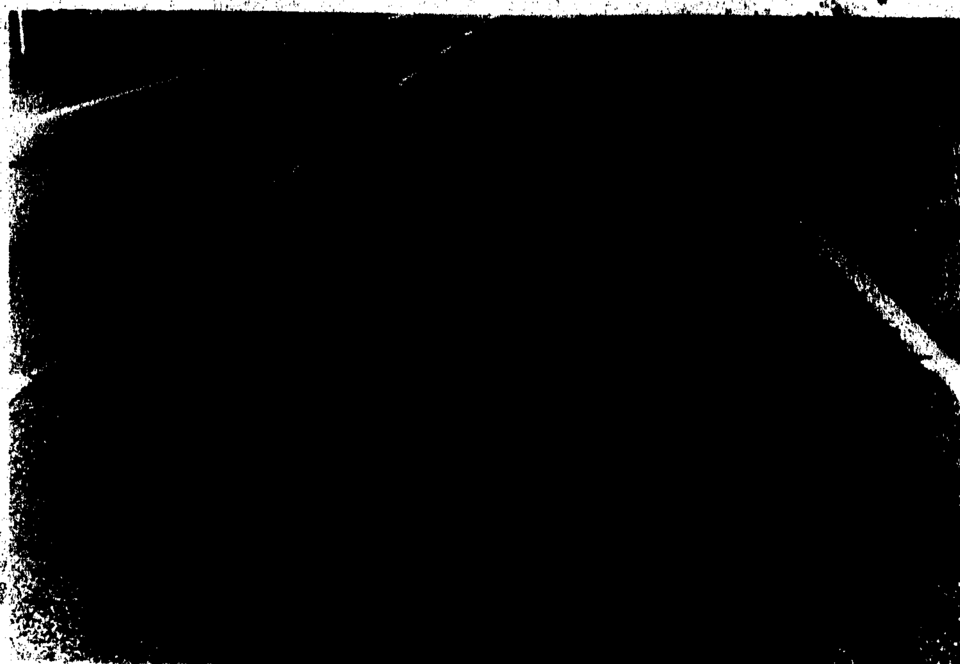

Photo E-1 Highway 14X:02. Cracking in longitudinal wheel paths. Pavement age - 13 years. Without overlay. Very heavy traffic

Photo E-2 Highway 14X:02. Cracking in longitudinal wheel path. Detailed view.

Photo E-3 Highway 15:04, Manning Freeway. General view. Pavement age - 16 years. Overlaid 1982. Very heavy traffic.

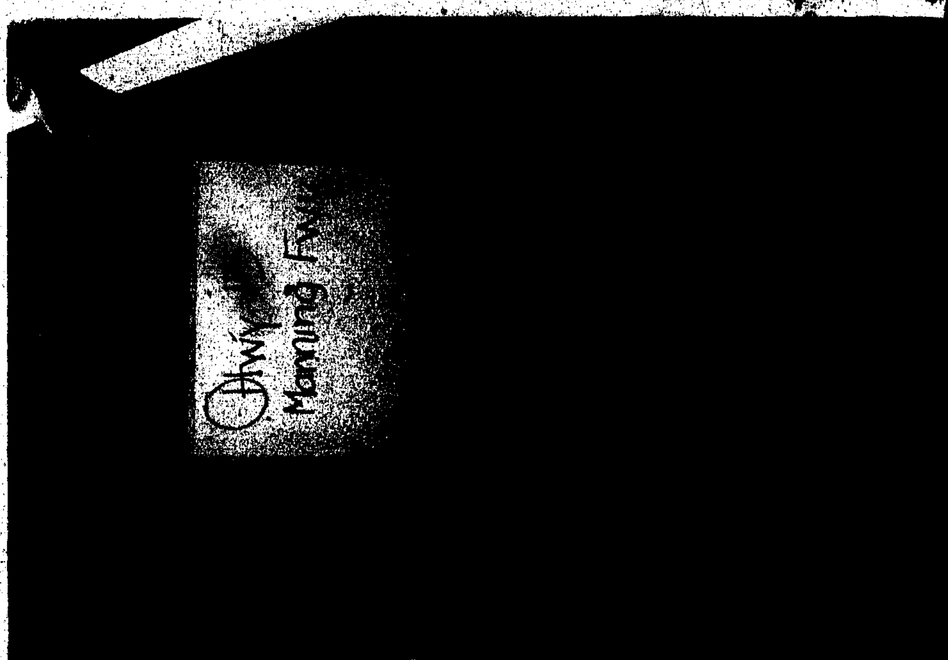

Photo E-4 Highway 15:04, Manning Freeway. Cracking developed in longitudinal wheel paths. Detailed view.

Photo E-5 Highway 36:[illegible] thin full-depth pavement (115 [illegible] Unusually good performance. [illegible]ent age - 14 years. No overlay. Low traffic.

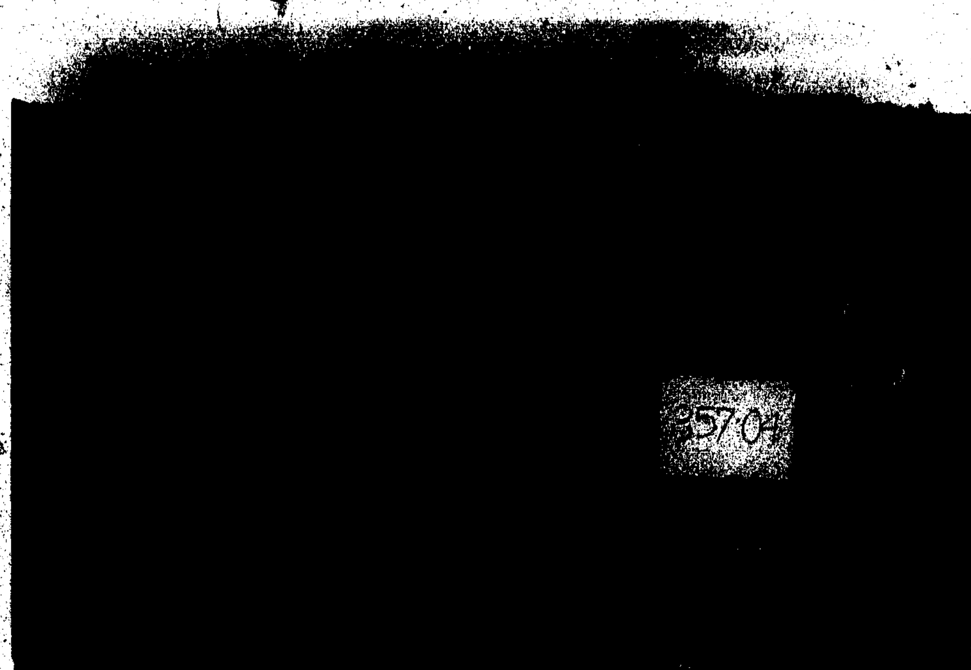

Photo E-6 Highway 857:04. Example of an excellent full-depth pavement. Pavement age - 9 years. Very low traffic.

Photo E-7 Highway 857:04. The only weak area along the highway. The subgrade bearing capacity loss probably due to frost action.

Photo E-8 Highway 45:06B. Example of overlaid full-depth pavement. Pavement age - 17 years. Overlay age - 10 years. Medium traffic.

Photo E-9 Highway 45:06B. Detailed view of the overlay (65mm)

Photo E-10 Highway 45:06. General view. Low - temperature transverse cracking

APPENDIX F

OVERLAY CALCULATION USING THE BENKELMAN BEAM DEFLECTION BASED RTAC METHOD

1. HIGHWAY 14X:02 km 1.80 to km 2.90 lane 1.

a) Maximum spring Benkelman Beam deflection measured in 1987 was on the average 2.67 mm (0.105 in.) - (see Table 6.25) with a standard deviation 0.69 mm (0.027 in.),

b) Section is uniform in distress evidence, soils, traffic and geometrics.

c) An overlay should be sufficient for a design period of 15 years. Estimated traffic volume from 1987 to 2002 is 5,158,032 equivalent single axle loads - see Appendix C for traffic details.

d) From Figure 6.2 a design Benkelman Beam deflection of 0.635 mm (0.025 in.) is required to carry the future traffic. The existing facility has a design deflection of 2.67 + 2 * 0.69 = 4.05 mm (0.159 in.).

e) From Figure 6.3, for the above deflection value of 4.05 mm (0.159 in.) and the design deflection of 0.635 mm (0.025 in.) the read off additional thickness of gravel layer is 625 mm (24.6 in.) or <u>250 mm (9.8 in.) of asphalt concrete layer</u>.

2.HIGHWAY 36:18 km 1.04 to km 1.60.

a) Maximum spring Benkelman Beam deflection measured in 1987 was on the average 2.67 mm (0.105 in.) - (see Table 6.25) with a standard deviation 0.46 mm (0.018 in.).

b) Section is uniform in distress evidence, soils, traffic and geometrics.

c) An overlay should be sufficient for a design period of 15 years. Estimated traffic volume from 1987 to 2002 is 767,446 equivalent single axle loads - see Appendix C for traffic details.

d) From Figure 6.2 a design Benkelman Beam deflection of 1.22 mm (0.048 in.) is required to carry the future traffic. The existing facility has a design deflection of 2.67 + 2 * 0.46 = 3.59 mm (0.141 in.).

e) From Figure 6.3, for the above deflection value of 3.59 mm (0.141 in.) and the design deflection of 1.22 mm (0.048 in.) the read off additional thickness of gravel layer is 270 mm (10.5 in.) or <u>110 mm (4.3 in.) of asphalt concrete layer</u>.

3.HIGHWAY 45:06A km 13.43 to km 13.98.

a) Maximum spring Benkelman Beam deflection measured in 1987 was on the average 2.74 mm (0.108 in.) - (see Table 6.25) with a standard deviation 0.20 mm (0.008 in.).

b) Section is uniform in distress evidence, soils, traffic and geometrics.

c) An overlay should be sufficient for a design period of 15 years. Estimated traffic volume from 1987 to 2002 is 483,732 equivalent single axle loads - see Appendix C for traffic details.

d) From Figure 6.2 a design Benkelman Beam deflection of 1.27 mm (0.05 in.) is required to carry the future traffic. The existing facility has a design deflection of 2.74 + 2 * 0.20 = 3.15 mm (0.124 in.).

e) From Figure 6.3, for the above deflection value of 3.15 mm (0.124 in.) and the design deflection of 1.27 mm (0.05 in.) the read off additional thickness of gravel layer is 240 mm (9.35 in.) or 95 mm (3.75 in.) of asphalt concrete layer.

4.HIGHWAY 45:06B km 23.27 to km 23.82.

a) Maximum spring Benkelman Beam deflection measured in 1987 was on the average 4.17 mm (0.164 in.) - (see Table 6.25) with a standard deviation 0.66 mm (0.026 in.).

b) Section is uniform in distress evidence, soils, traffic and geometrics.

c) An overlay should be sufficient for a design period of 15 years. Estimated traffic volume from 1987 to 2002 is 483,732 equivalent single axle loads - see Appendix C for traffic details.

d) From Figure 6.2 a design Benkelman Beam deflection of 1.27 mm (0.05 in.) is required to carry the future traffic. The existing facility has a design deflection of 4.17 + 2 * 0.66 = 5.51 mm (0.217 in.).

e) From Figure 6.3, for the above deflection value of 5.51 mm (0.217 in.) and the design deflection of 1.27 mm (0.05 in.) the read off additional thickness of gravel layer is 330 mm (13.0 in.), or <u>130 mm (5.2 in.) of asphalt concrete layer.</u>

5.HIGHWAY 857:04 km 12.25 to km 12.80.

a) Maximum spring Benkelman Beam deflection measured in 1987 was on the average 4.11 mm (0.162 in.) - (see Table 6.25) with a standard deviation 0.62 mm (0.024 in.).

b) Section is uniform in distress evidence, soils, traffic and geometrics.

c) An overlay should be sufficient for a design period of 15 years. Estimated traffic volume from 1987 to 2002 is 138,112 equivalent single axle loads - see Appendix C for traffic details.

d) From Figure 6.2 a design Benkelman Beam deflection of 1.78 mm (0.07 in.) is required to carry the future traffic. The existing facility has a design deflection of 4.11 + 2 * 0.62 = 5.36 mm (0.211 in.).

e) From Figure 6.3, for the above deflection value of 5.36 mm (0.211 in.) deflection and the design deflection of 1.78 mm (0.07 in.) the read off additional thickness of gravel layer is 250 mm (9.8 in.) or <u>100 mm (4.0 in.) of asphalt concrete layer</u>.

APPENDIX G

ALTERNATIVE CONSTRUCTION QUANTITIES AND INITIAL COSTS

HIGHWAY 14X:02 KM 0.00 TO KM 3.22 1 KILOMETER BASIS

QUANTITIES

ALTERNATIVE 1 OVERLAY

1. Pavement edge cut 1000 m
2. Reclaim edge of asphalt concrete pavement
   0.30*0.30*6*0.5*1000*2.294 = 619 tonnes
3. Earthwork ([(0.30-0.075)*6+2.40+2.40]/2*(0.30-0.075)+
   (1.00*6+2.40+2.40)/2*1.00)*1000 = 6092 cu.m
4. Subgrade preparation 2.40*1000 = 2400 sq.m
5. Crushed gravel base course from pit to sp. 2 - 25
   (0.075*6+2.40+2.40)/2*0.075]*1000*1.7*1.1 = 369 tonnes
6. Prime coat 2.40*1000 = 2400 sq.m
7. Tack coat 12.50*1000 = 12500 sq.m
8. Asphalt stabilized base course (ASBC)(12.50+0.15*6*2+
   12.50+0.20*6*2)/2*1000*0.05*2.294 = 1675 tonnes
9. Recycled asphalt concrete pavement (RACP)
   (12.50+12.50+0.15*6*2)/2*0.15*1000*2.294 = 4611 tonnes

ALTERNATIVE 2 RECONSTRUCTION

1. Reclaim existing ACP
   (12.50+12.50+0.28*6*2)/2*0.28*1000*2.294 = 9108 tonnes
2. Earthwork
   (2.00+2.00+1.00*6)/2*1.00*1000 = 5000 cu.m
3. Subgrade preparation 16.10*1000 = 16100 sq.m
4. Subgrade excavation 0.02*(12.50+0.28*6*2+

12.50+0.30*6*2)/2*1000 = 32 cu.m

5. Crushed gravel base course from pit to sp. 2 -25

(12.50+0.32*6*2+12.50+0.47*6*2)/2*0.15*1000*1.7*1.1

= 4836 tonnes

6. Prime coat (12.50+0.32*6*2)*1000 = 16340 sq.m

7. Recycled asphalt concrete pavement (RACP)

(12.50+12.50+0.32*6*2)/2*0.32*1000*2.294 = 10585 tonne

ALTERNATIVE 3 COLD MILLING AND OVERLAY

1. Pavement edge cut 1000 m

2. Reclaim of asphalt concrete edge

0.30*6*0.30*0.5*1000*2.294 = 619 tonnes

3. Earthwork {[(0.30-0.075)*6+2.80+2.80]/2*(0.30-0.075)+

(1.00*6+2.80+2.80)/2*1.00}*1000 = 6582 cu.m

4. Subgrade preparation 2.80*1000 = 2800 sq.m

5. Crushed gravel base course from pit to sp. 2 -25

(0.075*6+2.80+2.80)/2*0.075*1000*1.7*1.1 = 425 tonnes

6. Prime coat 2.80*1000 = 2800 sq.m

7. Tack coat (12.50+0.175*6*2)*1000 = 14600 sq.m

8. Cold milling existing ACP

0.15*7.00*1000*2.294 = 2409 tonnes

9. Haul of unnecessary cold mill 1709/3 = 570 tonnes

10. ASBC in place mixed and compacted [(0.05*7.00)+(2.50

+0.175*6+2.50+0.225*6)/2*0.05+(3.00+0.175*6+3.00

+0.225*6)/2*0.05]*1000*2.294 = 1709 tonnes

11. Crushed gravel to mix with remained RAP

0.95*570*1.1 = 596 tonnes

12. Asphalt cement 0.05*570 = 28.5 tonnes

13. Recycled asphalt concrete pavement (RACP)[(0.15*7.0[illegible]+

(12.50+0.175*6*2+12.50)/2*0.175]*1000*2.294 = 7848 tonnes

COSTS

The following cost calculations are based on average unit prices in Alberta in 1988. Numbers in brackets designate the item number according to the Alberta Transportation and Utilities unit prices - highway and road contracts.

ALTERNATIVE 1 OVERLAY

1. Pavement edge cut (P520) 1000m*$1.50/m = $1,500
2. Reclaim edge of ACP (P150) 619t*$5.208/t = $3,274
3. Haul Reclaimed ACP BLF (P160) 619t*$.726/t = $ 449
4. Haul Reclaimed ACP haul (P161)

   619t*15km$.113t.km = $1,049
5. Common borrow excavation to trucks (G138)

   6092cu.m*$2.741/cu.m = $16,698
6. Truck haul com. borrow exc. (G154)

   6092cu.m*15km*$.180/cu.m.km = $16,448
7. Subgrade preparation (B111, B112, B113, B114)

   2400sq.m*$.57/sq.m = $1,368

8. Crushed pit gravel to sp.2-25 (B314)	369t*$2.056/t =	$759
9. Haul gran. mat. BLF (B341)	369t*$.747/t =	$276
10. Haul gran. mat. haul (B342)	369t*15km*$.114t.km =	$631
11. Excavate and stockpile (B340)	369t*$1.058/t =	$390
12. Prime coat (B686)	2400sq.m*$0.028/sq.m =	$67
13. Prime coat mat. (X401)	2400sq.m*2kg/sq.m*$.176/kg =	$885
14. Tack coat (B688)	12500sq.m*$.012/sq.m =	$150
15. Tack coat mat. (X401)	12500sq.m*.3kg/sq.m*$.176/kg =	$660
16. ASCB from PRSP 2-16 (B622)	1675t*$8.010/t =	$13,417
17. Haul ASCB BLF (B640)	1675t*$.743/t =	$1,245
18. Haul ASCB haul (B641)	1675t*15km*$.112t.km =	$2,814
19. RACP 1-12.5 (P142)	4611t*$6.450/t =	$29,741
20. RACP BLF (P160)	4611t*$.743/t =	$2,624
21. RACP haul (P161)	4611t*15km*$.113/t.km =	$7,816

SUBTOTAL = $102,261

10% FIELD ENGEENERING = $10,226

TOTAL = $112,487

ALTERNATIVE 2 RECONSTRUCTION

1. Reclaim existing ACP (P150)		
	9108t*$5.208/t =	$47,434
2. Haul RAP BLF (P160)	9108t*$.726/t =	$6,612
3. RAP haul (P161)	9108t*15km*$.113/t.km =	$15,438
4. Common borrow exc.to truck (G138)		
	(5000-32)cu.m*$2.741/cu.m =	$13,617
5. Truck haul com. bor. (G154)		
	(5000-32)cu.m*15km*$.180/cu.m.km =	$13,414
6. Subgrade excavation (B100)		
	32cu.m*$1.360/cu.m =	$44
7. Subgrade preparation (B110, B111, B112, B113)		
	16,100sq.m*$.57/sq.m =	$9,177
8. Crushed pit gravel to sp. 2-25 (B314)		
	4396t*$2.056/t =	$9,942
9. Haul gran. mat. BLF (B341)	4836t*$.747/t =	$3,612
10. Haul gran. mat. haul (B342)		
	4836t*15km*$.114/t.km =	$8,269
11. Excavation and stockpile gran. mat. (B340)		
	4836t*$1.058 =	$5,116
12. Prime coat (B686)	16340sq.m*$.028 =	$458
13. Prime coat mat. (X401)		
	16340sq.m*.3kg/sq.m*$.176 =	$5,752
14. RACP 1 - 12.5 (P142)	10585t*$6.450/t =	$68,273
15. RACP BLF (P160)	10585t*$.569/t =	$6,023
16. RACP haul (P161)	10585t*15km*$.113/t.km =	$17,942

SUBTOTAL =	$231,123
10% FIELD ENGINEERING =	$23,112

TOTAL = $254,235

ALTERNATIVE 3 COLD MILL AND OVERLAY

1.	Pavement edge cut (P520)	1000m*$1.50/m =	$1,500
2.	Reclaim edge of ACP (P150)	619t*$5.208/t =	$3,274
3.	Haul Reclaimed ACP BLF (P160)	619t*$.726 =	$449
4.	Haul Reclaimed ACP BLF (P161)	619t*$.113 =	$1,049
5.	Common borrow excavation to trucks (G138)	6582cu.m*$2.741/cu.m =	$18,041
6.	Truck haul com. borrow exc. (G154)	6582cu.m15km*$.180/cu.m.km =	$17,771
7.	Subgrade preparation (B111, B112, B113, B114)	2800sq.m*$.57/sq.m =	$1,596
8.	Crushed pit gravel to sp.2-25 (B314)	425t*$2.056/t =	$873
9.	Haul gran. mat. BLF (B341)	425t*$.747/t =	$317
10.	Haul gran. mat. haul (B342)	425t*15km*$.114t.km =	$726
11.	Excavate and stockpile (B340)	425t*$1.058/t =	$449
12.	Prime coat (B686)	2800sq.m*$0.028/sq.m =	$78
13.	Prime coat mat. (X401)	2800sq.m*2kg/sq.m*$.176/kg =	$986

14. Tack coat (B688) 14600sq.m*$.012/sq.m = $175

15. Tack coat mat. (X401)

14600sq.m*.3kg/sq.m*$.176/kg = $771

16. Cold mill exist. ACP (P150) 2409t*$5.208/t = $12,546

17. Cold mill BLF (P160) 570t*$.726/t = $414

18. Cold mill haul (P161) 570t*15km*$.113/t.km = $966

19. Crushed gravel from pit to sp. 1 - 12.5 (B302)

596t*$4.423/t = $2,636

20. Crushed gravel from pit BLF (B341)

596t*$.747/t = $445

21. Crushed gravel from pit haul (B342)

596t*15km*$.114/t.km = $1,019

22. Excavate and stockpile gravel (B340)

596t*$1.058/t = $631

23. Asphalt SS-1 (X400) 28.5t*$176.218/t = $3,807

24. Mixing, placing and compaction of ASBC (see B117)

[(2409/2.294)+(28.5/1.01)]t*$3.000/t = $3,235

25. RACP 1 - 12.5 (P142) 7848t*$6.450/t = $50,620

26. RACP BLF (P160) 7848t*$.569/t = $4,466

27. RACP haul (P161) 7848t*15km*$.113/t.km = $13,302

SUBTOTAL = $142,142

10% FIELD ENGINEERING = $14,214

TOTAL = $156,356