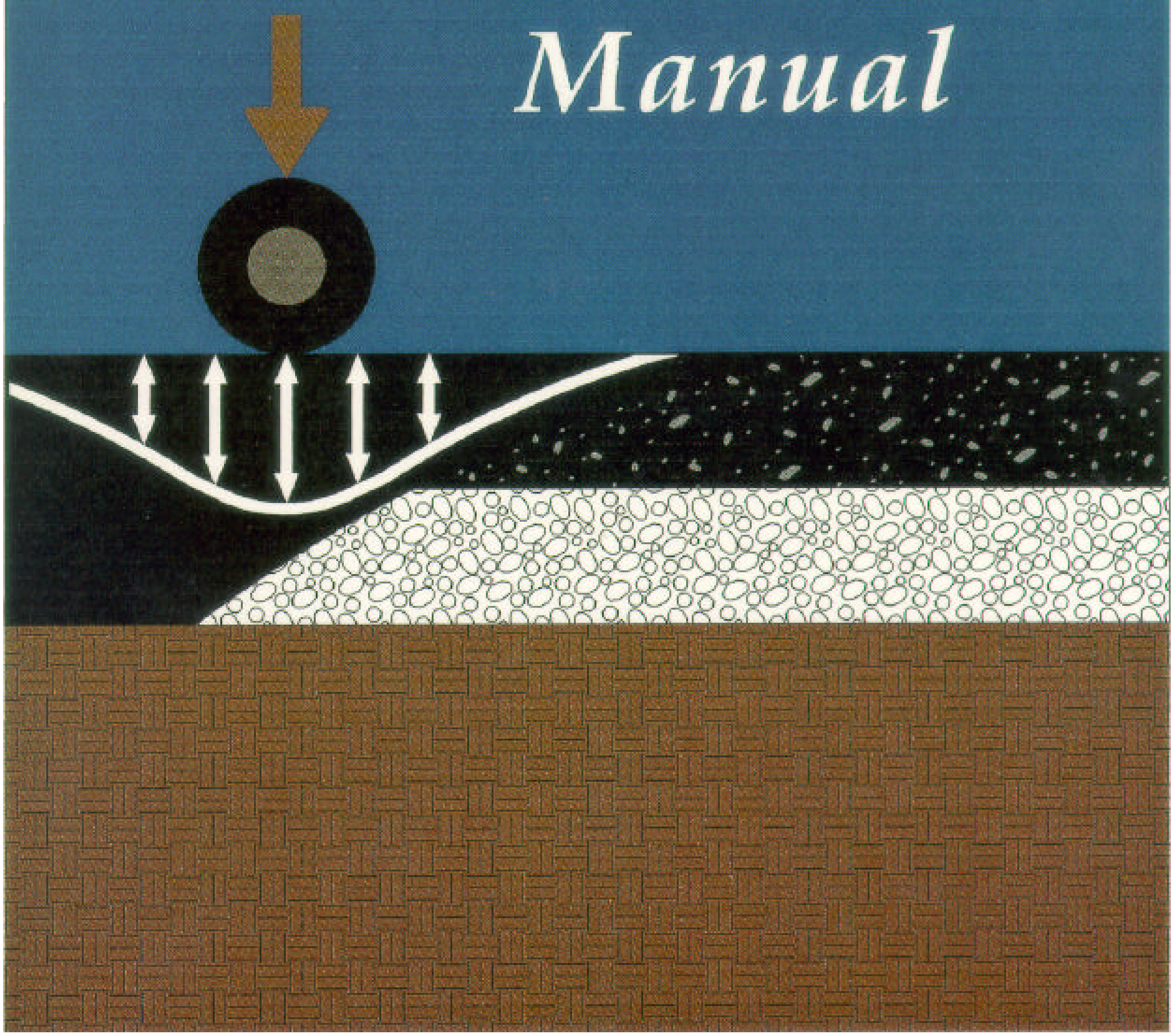
Alberta
TRANSPORTATION
AND UTILITIES

Pavement Design Manual



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TRANSPORTATION
AND UTILITIES

Pavement Design Manual

Edition 1 *June 1997*

FOREWORD

The Pavement Design Manual is published by Alberta Transportation and Utilities (AT&U) for use in the Province of Alberta to promote uniformity of pavement design and to achieve long lasting quality pavements in a cost-effective manner, that will contribute to traffic safety and efficient roads for the well-being of the travelling public.

In general, the Manual reflects past AT&U design practices that have resulted in decades of cost-effective pavement performance conducive to Alberta environmental, traffic and materials conditions. It reflects the most appropriate design methodologies and strategies, adapted for Alberta conditions and experience, that are available at the present time. Changes in technology related to design and construction practices will necessitate revisions to the Manual. It provides the engineering consultant industry a guideline for pavement design for new roadway construction, final pavements and pavement rehabilitation. The pavement design methodologies apply to the surfacing of rural highways only. Use of these design practices will provide cost-effective pavement designs to suit the specific requirements of each project.

This Manual is not a textbook or a substitute for engineering knowledge, experience or judgement. It is a guiding Manual that must be used in conjunction with AAASHTO Guide for Design of Pavement Structures, 1993".

The information presented in this Manual was carefully researched, compiled and presented. However, no warranty, expressed or implied, is made on the accuracy of the contents or their extraction from referenced publications. Alberta Transportation and Utilities, AGRA Earth & Environmental and their sub-consultants assume no responsibility for errors or omissions or possible misinterpretation that may result from use and interpretation of the material herein contained.

June 1997

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The Manual was prepared by **AGRA Earth & Environmental Limited** (AEE) under the direction of the Steering Committee, Alberta Transportation & Utilities. The Steering Committee included members from the Planning, Programming & Technical Services Division and Regional Services Division.

The following people were directly involved in the preparation of the Manual and their contribution towards the development of this Manual is appreciated.

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1 INTRODUCTION

1.1 Objective

This Manual provides a comprehensive guideline to be followed by engineering consultants for pavement design for new roadway and final stage pavement construction, including reconstruction and widening, and rehabilitation. Engineering consultants are responsible for the surfacing strategy and detailed structural pavement design on roadway design project assignments. For the purposes of this Manual, pavement refers to all layers of the pavement structure above the subgrade.

An objective of the Manual is to ensure a degree of consistency in designs provided by engineering consultants by following specific structural design methodologies within a general framework. At the same time the design process provides sufficient flexibility to allow for the judgement and innovation by experienced pavement design engineers to address the specific conditions of each project.

Alberta Transportation and Utilities (AT&U) will continue to be the custodian of all pavement evaluation, management and inventory data. These data will be available for use by engineering consultants. AT&U's role in the design process will be to review pavement designs provided by consultants for completeness, conformance to the design philosophies and methodologies outlined in the Manual, and to ensure that the design is supported by appropriate engineering investigation and evaluation.

1.2 Scope and Limitations

The methodologies provided in this Manual apply to the design of flexible (granular base course) and semi-rigid (cement stabilized base course) pavement structures on Alberta Primary Highways and Secondary Highways. These methodologies apply to the design of rural highways only and may not be directly transferable to urban roadways where traffic speeds, drainage conditions etc. may be different. The design of seal coat, slurry seal and micro-surfacing applications, which are generally considered to be maintenance or preservation strategies, are not addressed.

The Manual reflects the most appropriate design methodologies, adapted for Alberta conditions and experience, that are available at the present time. Changes in technology related to non-destructive pavement evaluation testing, laboratory testing and analysis, mechanistic pavement design and SHRP SUPERPAVE new materials; new maintenance practices; and changing traffic conditions and loadings will all influence the future performance of pavements and will result in necessary changes to the Manual in the future.

The Manual is not all encompassing in terms of addressing all factors that may influence the design and performance of a pavement. Pavement designers will need to address these factors on a project-by-project basis and, where necessary, will have to carry out additional research to ensure appropriate and cost-effective design solutions are provided.

It is important that the design engineer have ready access to background publications and the research of others (eg. Asphalt Institute [AI], American Association of State Highway and Transportation Officials [AASHTO], Federal Highway Administration [FHWA], Transportation Research Board [TRB], Transportation Association of Canada [TAC], Association of Asphalt Paving Technologists [AAPT], Canadian Technical Asphalt Association [CTAA], etc.) that form the technical background to the design and performance of flexible pavement structures. This Manual must be used in conjunction with the AASHTO Guide [AASHTO 93]. The details and background included within the Guide are extensively referenced within the Manual.

In general, this Manual reflects past AT&U design practices that have resulted in decades of cost-effective pavement performance experience under Alberta environmental, traffic and materials conditions. AT&U chooses to design new pavement structures to last 20 years before rehabilitation becomes necessary according to Department standards of acceptance and performance expectations. The Department places a high priority on the ride quality and serviceability of pavements. Pavements are design and constructed as economically as possible and, on a network basis, the occasional failure for structural reasons is deemed acceptable and a demonstration that pavement structures are not being over designed.

The general philosophy to stage the design and construction of new construction pavements has been maintained. The Manual includes state-of-the-art methods for the structural design of new roadway construction, final stage pavements and rehabilitation. The Manual will allow for the use of granular subbase and cement stabilized base courses where proven cost-effective.

Rehabilitation design will require life cycle cost analysis in order to assess various alternative strategies and to identify the preferred alternative.

1.3 Organization of the Manual

The system outlined in the Manual for pavement design is presented in Figure 1.1. Also shown in the figure are the referenced sections within the Manual. Within the Manual, brief descriptions of the significant factors affecting the design and performance of pavements are provided.

The background, design inputs, and outline of the pavement design methodology, including worked through examples, are provided for:

- new construction
- final stage pavement
- rehabilitation - structural overlays
- non-structural rehabilitation.

Application of life cycle cost analysis to rehabilitation design is presented. The design of pavement structures for low volume roads is addressed.

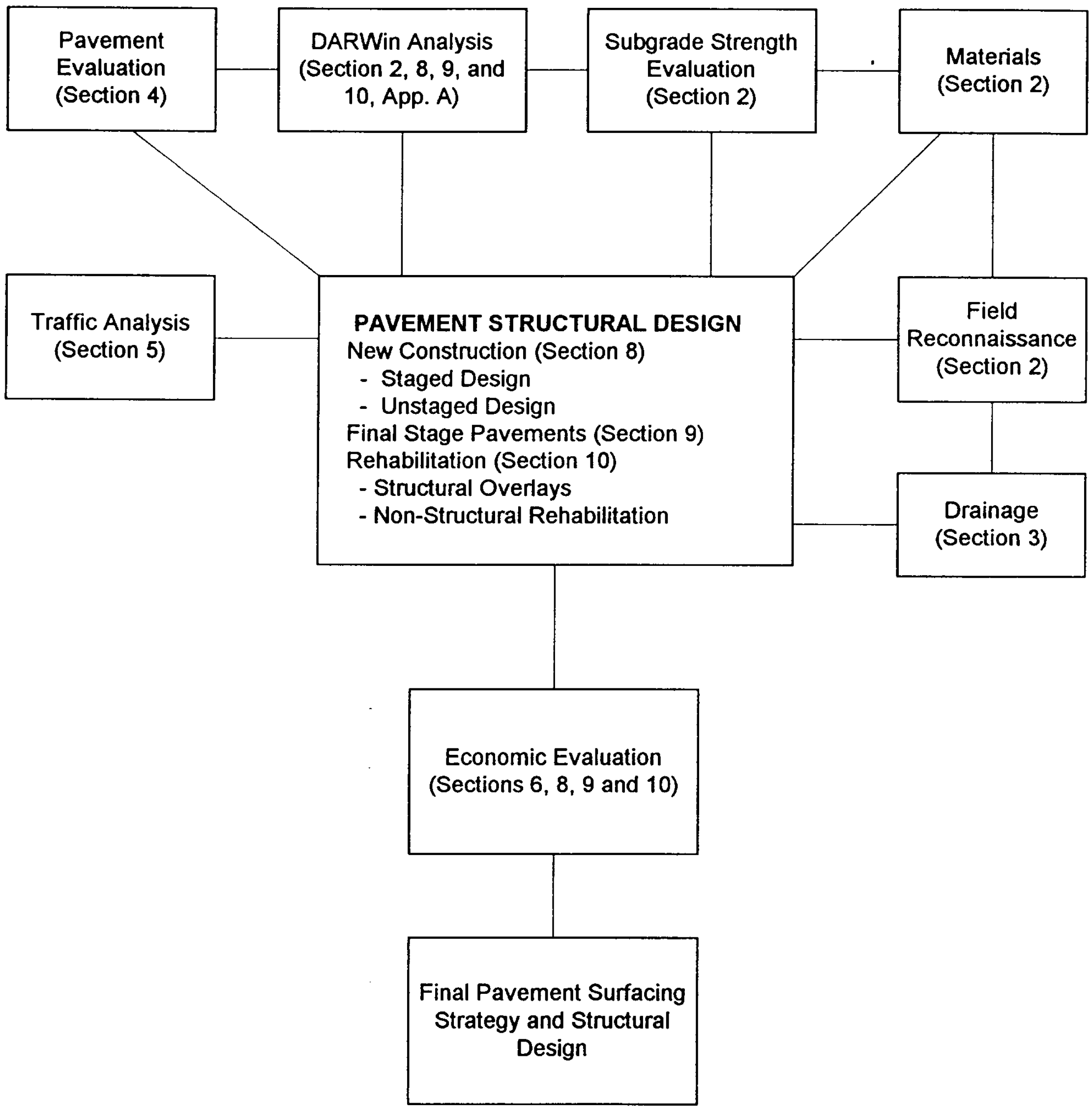


Figure 1.1 Pavement Design System

2 MATERIALS

2.1 Introduction

The components and definitions of materials essential to Alberta flexible pavement structures include the subgrade, a granular base course, and a surfacing of asphalt concrete. Granular base pavement structures comprise about 75 percent of the secondary and primary highway network. To a lesser extent, cement stabilized base courses (soil cement) or full depth asphalt concrete pavements have been designed and constructed in the past. These latter two pavement types together would represent about 25 percent of the secondary and primary highway network pavement structures.

The subgrade is comprised of the uppermost materials placed in the road bed embankment or the soil remaining at the base of a cut. The subgrade soil is often referred to as the foundation or road bed soil. This foundation component is usually constructed of native inorganic soil often in combination with imported soils from select borrow sources, and would be compacted to a specified density and moisture content.

The granular base course (GBC) is that material placed immediately above the prepared subgrade. The GBC used in Alberta consists of a well graded crushed gravel with a maximum particle size varying from 20 mm to 40 mm. On occasion the GBC is separated from the subgrade by a granular subbase course of lower quality and less expensive material. Granular subbase course (GSBC) generally consists of pit run gravel fill with a maximum particle size of 80 mm to 125 mm.

Asphalt stabilized base courses (ASBC) were traditionally used as temporary wearing courses on first stage granular base or cement stabilized base course projects. Based upon AT&U analysis, 50 mm of ASBC has been superseded by 60 mm ACP. Presently ASBC is only used under special circumstances by some municipalities.

The top layer of the flexible pavement structure is comprised of a densely graded, hot mix, asphalt concrete pavement (ACP). In addition to functioning as a structural component of the pavement structure, the ACP must also resist the abrasive forces of traffic and climatic and environmental conditions, minimize surface moisture infiltration to the underlying pavement structure, provide a skid resistant surface, and provide a smooth riding surface. The selection of asphalt concrete mix types has been previously developed for Alberta traffic volume and temperature conditions and will be subsequently discussed.

Specific requirements relating to physical and quality attributes of materials utilized in pavement construction are included in Table 3.2.3.1 Specification for Aggregate in the AT&U Standard Specifications For Highway Construction [AT&U 97].

2.2 Subgrade Soils

2.2.1 Soil Classification

The basic components of soils are differentiated on the basis of grain size as follows:

Cobbles	plus 75 mm
Gravel	75 mm to 5 mm
Sand	5 mm to 0.075 mm
Silt	0.075 mm to .002 mm
Clay	minus .002 mm

Fine grained soils are defined as materials having more than 50 percent of the dry mass smaller than the 0.075 mm particle size. Although size limitations are arbitrary, such limitations allow standardization by definition. It is necessary to understand as well that plasticity is an extremely important property to differentiate between silt and clay, and to predict behaviour. The pavement design engineer is most interested in the strength of the soil and the extent to which this strength varies with climate, environment and drainage effects.

A typical soil profile for highway design purposes consists of three horizons. The surface or 'A' horizon materials will consist of organic soils, followed by 'B' horizon which is a semi-weathered zone followed by 'C' horizon which represents the parent material type.

A significant portion of Alberta's near surface soils utilized for road building purposes originated from the glaciation process. As the glaciers advanced and retreated, materials were mixed, segregated and deposited. Some materials became homogeneous, some materials were deposited as granular outwash, while others settled out in still waters and formed some of Alberta's lacustrine clays.

The Modified Unified Soil classification system originally developed by Casagrande is the basis for the system utilized in Alberta. This system uses plasticity to differentiate between silts and clays. A plasticity chart presented in Figure 2.1 follows which correlates liquid and plastic limit test results to Soil Group symbols used for soil description.

For all new construction it is very important that in-situ moisture contents, Atterberg limits and grain size analysis of subgrade soil materials be determined to assess subgrade soil characteristics and to infer resilient modulus (M_R) values.

2.2.2 Subgrade Strength Evaluation

The characteristic material property of subgrade soils used for pavement design is the resilient modulus (M_R). The resilient modulus is defined as being a measure of the elastic property of a soil recognizing selected non-linear characteristics. Methods for the determination of M_R are described in AASHTO T294-92 test method. For many years, standard California Bearing Ratio (CBR) tests were utilized to measure the subgrade strength parameter as a design input.

For roadbed materials, the AASHTO Guide [AASHTO 93] recommends that the resilient modulus be established based on laboratory testing of representative samples in stress and moisture conditions simulating the primary moisture seasons. Alternatively, the seasonal resilient modulus values may be determined based on correlations with soil properties.

Since the resilient modulus test equipment is currently not present in many laboratories, researchers have developed correlations to converting CBR values to approximate M_R values. The correlation considered reasonable for fine grained soils with a soaked CBR of 10 or less is:

$$M_R\ (\text{MPa}) = 10.3 * (\text{CBR}) \quad [\text{AASHTO 93}]$$

Since 1991, AT&U has used the Falling Weight Deflectometer (FWD) to obtain deflection data. The ELMOD (Evaluation of Layer Moduli and Overlay Design) computer program was used to analyze the FWD deflection data. With the recent adoption of the AASHTO method for the design of pavement structures by AT&U, a computer program called DARWin 3.0 (Design, Analysis and Rehabilitation for WINdows) has been adapted to analyze FWD deflection data and to establish a backcalculated subgrade modulus.

For the purposes of this Manual, the backcalculated subgrade modulus is used to represent the in-situ subgrade resilient modulus which in turn is an input for the design of final stage pavements and overlays.

For the design of new construction pavement structures, the subgrade resilient modulus is estimated using an existing representative roadway located near the new project, with similar subgrade soils and drainage conditions, as a prototype. The prototype should preferably meet the following criteria [TAC 97]:

- be a minimum of 3 years old
- be a minimum of 0.5 km in length
- be reasonable free of structural distress
- be slightly under-designed for the loading conditions on the new highway.

The prototype can be tested with the FWD and the deflection data analyzed with DARWin 3.0 to determine the backcalculated subgrade modulus. This value can then be used as an approximation of the strength of the subgrade materials that would exist in the new subgrade.

2.2.3 Seasonal Variations

One of the most critical conditions that develops in a seasonal frost area such as Alberta is the weakening of the subgrade during the spring thaw period. This weakening results from the melting of ice segregation within the subgrade soils and, to a lesser extent, due to higher moisture contents during this period associated with reduced drainage.

Seasonal variations of subgrade strength is a difficult factor to model. The task is one of determining the extent to which the subgrade strength is reduced during or immediately following the thaw period. Nevertheless, the seasonal variations model is important to the structural design of pavements.

A study of subgrade strength seasonal variations was conducted by AT&U from 1989 to 1994 [Kurlanda 94]. This study was based on FWD deflection testing of several pavement test sections and subsequent analysis using the ELMOD computer program. Although the strength reductions were contingent upon several factors, for many subgrades the reduction was up to 50 percent of the summer strength. Pavements with cement stabilized base courses generally indicated less reduction in subgrade strength. For full-depth asphalt concrete pavements the reduction was similar to that of granular base pavements but the period of minimum subgrade strength occurred in the late spring to early summer rather than early spring as for the other base types. As a result of this research, subgrade seasonal variation factors were developed that were used in the ELMOD analysis.

The AASHTO Guide [AASHTO 93] outlines guidelines for determining the seasonal variation of the subgrade modulus based either upon laboratory resilient modulus testing or from backcalculated moduli determined from FWD deflection data. A procedure is described which allows the effective roadbed soil resilient modulus to be determined based on the estimated relative damage that corresponds to the seasonally adjusted subgrade modulus for each month of the year.

Table 2.1 provides an example of the application of the procedure used to estimate the effective M_R. In this example, an average backcalculated subgrade modulus of 82 MPa was determined using the results of FWD test data collected on a prototype section in June and an acceptable backcalculation program. This value is presented in bold in the Table. This value was adjusted for other months, using the seasonal factors developed by AT&U.

These seasonally adjusted values were then multiplied by 0.33 to obtain corrected values for the subgrade modulus.[1] These reduced subgrade moduli were then used to estimate Relative Damage following the methods used in the AASHTO Guide (Part II Figure 2.3) for estimating the effective roadbed soil resilient modulus. The Relative Damage, u_f, was calculated using the equation:

$$u_f = 1.18 \times 10^8 \times M_R^{-2.32} \qquad \text{where } M_R \text{ is in psi.}$$

Since this empirically derived equation is in psi, the conversion from MPa to psi was made using the factor of 1 MPa = 145 psi. The average u_f was used to obtain an M_R value which corresponded to the Relative Damage, u_f. This can be done by solving the above equation for M_R, or by using the M_R/u_f scale on Figure 2.3. These calculations should be performed using spreadsheet methods which

[1] AASHTO methodology requires that M_R values backcalculated from FWD deflection data be adjusted to be consistent with the values used to represent the AASHO Road Test subgrade in the development of the flexible design equation. A correction factor of C=0.33 is recommended within the Guide to adjust backcalculated M_R values to design M_R values. For purposes of this Manual, the term "design M_R" is equivalent to the AASHTO term of Effective Roadbed Soil Resilient Modulus.

A recent FHWA Design Pamphlet prepared by Brent Rauhut Engineering Inc. [BRE 97] as part of a research study entitled "Backcalculation Of Pavement Layered Moduli In Support Of The 1993 AASHTO Guide For The Design Of Pavement Structures" recommended a C-value of 0.35 for subgrade soil below a pavement with an unbound granular base layer. This study in essence supports the use of the AASHTO correction factor of 0.33. The Executive Summary of this report is contained in Appendix C.

have been verified using the given data in the AASHTO Guide (Part II, Figure 2.4). The Effective Roadbed M_R , in psi, was then converted to MPa.

Table 2.1 was imported from a spreadsheet developed in Excel with the Seasonal Adjusted Modulus values in MPa. Reduced Soil Modulus values were converted to both MPa units as well as psi units, in order to verify the calculations. This would explain the large number of digits reported for the M_R in psi which are not considered significant.

Table 2.1 CHART FOR ESTIMATING EFFECTIVE ROADBED M_R

Month	AT&U Seasonal Factors	Seasonal Adjusted Modulus M_R (MPa)	Roadbed Soil Modulus M_R (psi)	Reduced Soil Modulus M_R (MPa)	Reduced Soil Modulus M_R (psi)	Relative Damage U_f
Jan	5.00	410	59462	135	19623	0.01
Feb	5.00	410	59462	135	19623	0.01
Mar	2.75	226	32704	74	10792	0.05
Apr	0.625	51	7433	17	2453	1.61
May	0.875	72	10406	24	3434	0.74
June	1.00	**82**	11892	27	3925	0.54
July	1.00	82	11892	27	3925	0.54
Aug	1.00	82	11892	27	3925	0.54
Sept	1.00	82	11892	27	3925	0.54
Oct	1.00	82	11892	27	3925	0.54
Nov	2.00	164	23785	54	7849	0.11
Dec	4.00	328	47570	108	15698	0.02
				Summation:		5.25
				Average:	$¬_f$ =	0.44
			Effective	Roadbed	M_R (psi)	4300
			Effective	Roadbed	M_R (MPa)	**30**

In order to provide a simplified method of determination of the design M_R for use by designers in Alberta, a parametric study was undertaken for the Southern, Central and Peace River regions of the Province using the seasonal factors developed by AT&U. Such factors were considered representative of Central Alberta and were adjusted subjectively to reflect seasonal differences in southern and northern Alberta.

The 0.33 reduced backcalculated M_R values were used to estimate relative damage and determine the effective Roadbed Soil Resilient Modulus following the methods used in Figure 2.4 of the AASHTO Guide and demonstrated in the previous example.

For FWD testing performed during the months of June to October, the calculated Effective Roadbed Soil Resilient Modulus was approximately 10 percent higher than the Reduced soil Modulus values, over the range of 20 to 150 MPa. This relationship is dependent upon the particular seasonal parameters assumed and therefore should be considered as climatic/geographic specific. It is suggested that this be considered a regional adjustment factor (C_{REG}) of 1.10 for Alberta. Combining the adjustment factor (C) of 0.33 with this regional adjustment factor (C_{REG}) of 1.10, the Effective Roadbed Resilient Modulus for design purposes can be determined by the following equation:

$$\text{Design } M_R = 0.36 \text{ x (backcalculated } M_R)$$

This combined adjustment factor would only apply to pavements tested by the FWD during the months of June through October when the subgrade is in a relatively stable and unfrozen condition.

2.2.4 Swelling Soil Potential

Excessively expansive soils such as highly plastic clays or bentonitic shales require special attention particularly when in close proximity to the surface of the road embankment. These materials contain minerals which result in volume changes (swelling and shrinking) with changes in moisture content.

Utilization of swelling materials in only lower portions of the embankment is often undertaken in order to minimize these effects. Compaction of this soil type at moisture contents slightly in excess of optimum moisture content will also often result in reduced swelling potential. Alternatively, the use of soil modifiers such as lime or Portland cement have been utilized as effective and economical solutions to reduce the swelling potential of these soils.

The need to control the intrusion of moisture into such soils is of major importance in order to mitigate swelling. Special considerations should be directed at pavement surface cracks and joints as well as at culvert locations.

2.2.5 Frost Susceptibility

The Alberta climate results in freezing of near surface subgrade soils for several months each year. The depth of frost penetration generally increases from the south to the north of the province.

Although some volumetric expansion occurs due to the freezing, a more significant issue relates to the spring melt period. The thaw will release excess water which causes a loss of subgrade strength and potential damage to the roadway pavement structure if the structure has not been designed to account for weakened subgrade support.

The term frost heaves refers to the upward vertical movement of a pavement surface as a direct result of the formation of ice lenses in a frost susceptible subgrade. For true frost heave to occur the following three factors must be present:

1. A frost susceptible soil.
2. Slowly depressed air temperatures.
3. A supply of water.

The removal of any one of the three factors will usually be sufficient to significantly reduce the potential for frost heaving and resulting surface distress. Differential frost heave can be mitigated at the design and construction stages by selective utilization of embankment and subgrade soil types.

Several methods have been developed for the characterization of frost susceptible soils. Casagrande initially formulated a guideline relating frost susceptibility to the percentage of particles by mass finer than 0.02 mm. The U.S. Corps of Engineers expanded the system as shown in the following table with F1 soils the least frost susceptible and F4 soils the most frost susceptible.

U.S. Corps of Engineers Frost Design Soil Classification

Frost Group	Soil Type	Percentage finer than 0.02 mm, by weight	Typical soil types under Unified Soil Classification System
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	a) Gravelly soils	>20	GM, GC
	b) Sands, except very fine silty sands	>15	SM, SC
	c) Clays, PI > 12	--	CL, CH
F4	a) All silts	--	ML, MH
	b) Very fine silty sands	>15	SM
	c) Clays, PI < 12	--	CL, CL-ML
	d) Varved clays and other fined-grained, banded sediments	--	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

2.2.6 Organics

The extremely compressible nature of highly organic materials in the subgrade often leads to problems related to pavement performance. These problems are further intensified when the depths and the properties of the organic deposits are non-uniform. Prudent practice includes removal of the organic layer, and stockpiling for future use, particularly where the deposits are shallow (0 - 2.5 m).

Deeper and/or more extensive peat or muskeg deposits require considerations such as displacement, surcharge embankments for preloading often with special drainage provisions, or the use of geo-synthetics.

2.3 Granular Base and Subbase

Base courses and granular subbase courses are used in flexible pavements to increase the load supporting capacity of the structure. Secondary benefits related to the use of untreated granular materials include improved drainage and added protection against frost action. As described earlier the base course is constructed near the pavement surface and is required to possess a high resistance to deformation. Subbase materials placed between the base and the subgrade can be of lower quality and are generally a less expensive material. Subbase materials used in Alberta in the past generally have been limited to the occasional use of pitrun aggregates.

Base course materials designed for maximum stability must possess high internal friction which is a function of particle size distribution, particle shape and density. Aggregates with little or no fines are also desirous due to being pervious (free draining) and less frost susceptible.

In general, it has been determined for Alberta conditions that performance and economy are well balanced when the largest maximum aggregates size are utilized assuming the crush count criteria has been achieved.

2.4 Asphalt Concrete

High quality asphalt concrete mixtures are capable of being produced in Alberta largely due to the availability of good quality aggregates and the high quality and consistency of locally produced asphalt cements. Air temperature affects asphalt concrete pavement with respect to low temperature cracking, fatigue cracking, and rutting. In general, softer grades of asphalt cements tend to be used for colder climates and/or lower traffic volumes, while harder grades of asphalt cements are more suitable for warmer climates and/or higher traffic volumes. Specific asphalt concrete selections coupling both temperature and traffic considerations have been developed based on experience for Alberta conditions. These selections of asphalt concrete mix types includes the asphalt cement selection. A design map for the selection of mix types, Figure 2.2, and the accompanying Table 2.2 follow. Table 2.3 presents the most recent AT&U specification requirements for asphalt concrete mix types and characteristics [AT&U 97].

SUPERPAVE (SUperior PERforming asphalt PAVEments) is a product of the Strategic Highway Research Program (SHRP) and incorporates performance-based asphalt materials characterization and new mix design procedures to improve rutting, low temperature cracking and fatigue cracking performance of asphalt concrete pavements. The SUPERPAVE system comprises specifications, test methods and selection criteria for binders and asphalt mixtures, and a mix design process using the gyratory compactor.

SUPERPAVE Level 1 mix design methodology and the use of modified binders are being evaluated or assessed by AT&U. The application of these new technologies should be considered where project conditions eg. improved performance, reduced life cycle costs etc., would justify their use.

2.5 Field Reconnaissance For Design

Field reconnaissance is an integral component of pavement design. The reconnaissance should be carried out in conjunction with the assessment other pavement evaluation information obtained from the Pavement Management System (PMS), the Maintenance Management System (MMS) and FWD testing. During the reconnaissance some additional information which is not identified during the preliminary stage and which may be important for rehabilitation design can be obtained. Such information may include:

$ Identification of the predominant pavement distress mode, the potential causes of the distress and the influence the distress may have on the performance of proposed alternative rehabilitation treatments. Attention should be especially given to the extent and cause of pavement rutting as different rehabilitation measures may be needed for different causes of rutting. Assessment of frequency and severity of transverse cracking is also important. Pre-overlay repair of severe transverse cracking may be a cost-effective treatment to reduce the severity and occurrence of reflective cracks.

$ Identification of localized areas of fatigue distress, severe settlements and frost heaves as such areas may require removal and replacement.

$ Location of pavement areas experiencing drainage or subdrainage problems. The subdrainage problem areas may initially be located by assessing the FWD data but the field inspection will often reveal moisture-related distresses. Distresses such as asphalt stripping, structural rutting, depressions, fatigue cracking and potholes may be good indications of subdrainage problems.

$ Identification of the need for levelling course or re-profiling to re-establish pavement cross-section, superelevation, rut-filling, treatment of existing crack filler, or other remedial work.

Coring and drilling may be required after the reconnaissance to confirm pavement layer thicknesses, structure and material types, and to characterize and evaluate subgrade soil conditions. In addition, coring is required for projects where asphalt recycling and Hot In-place Recycling (HIR) are considered as potential rehabilitation strategies.

2.6 Laboratory Evaluation And Materials Characterization

Laboratory evaluation and material characterization are usually conducted for rehabilitation projects where recycling, HIR, or pavement reconstruction involving reuse of existing pavement and granular materials is considered.

In cases where recycling of the asphalt concrete pavement is being considered, the asphalt cement rheology of the existing pavement is characterized. This usually involves determination of the recovered asphalt cement penetration and viscosity. In addition, the asphalt content, in-situ pavement density and air voids, aggregate gradation and degree of asphalt stripping are evaluated. These results are used to determine deficiencies in the asphalt concrete pavement properties and to assess the potential for recycling of the pavement.

Laboratory testing of the reclaimed granular material may involve such tests as sieve analysis, Los Angeles abrasion loss, petrographic number, plasticity index and the percentage of crushed particles. These tests help to determine if there is any degradation of the in-situ granular material and to classify it for reuse either as granular base or subbase.

Subgrade soil investigation usually involves particle size analysis, moisture content determination, Atterberg Limits and moisture/density relationship (Proctor test). In addition to those standard tests some other specialized tests may be needed as well. Specialized testing may include the California Bearing Ratio (CBR) test and the resilient modulus test which can provide information on the strength of the subgrade material.

2.7 Summary

One of the most important inputs into the structural pavement design process is the evaluation of the subgrade strength. The continued philosophy and practice to use stage design and construction mitigates the inherent difficulty in determining the effective roadbed soil resilient modulus (design M_R).

Recommendations for determination of subgrade strength design inputs are as follows:

1. For final stage paving and overlays, the design M_R should be determined by conducting FWD testing on the actual roadway and analysing the deflection data with DARWin 3.0 to determine the backcalculated subgrade modulus.

2. For new construction roadways, the design M_R should be determined by evaluating a suitable prototype roadway in the vicinity of the project and analysing FWD deflection data with DARWin 3.0 to determine the backcalculated subgrade modulus.

   This evaluation should be supplemented with an assessment of the results of laboratory testing to determine soil classification, moisture content and plasticity index of materials proposed for subgrade construction of the new roadway if available.

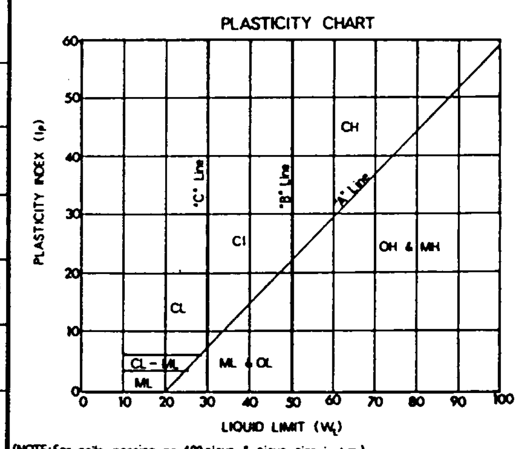
3. If suitable prototypes are unavailable, the results of laboratory testing to determine soil classification, moisture content and plasticity index of materials proposed for subgrade construction should be used to estimate the design M_R. Consideration should be given to environmental conditions in selecting the design M_R.

Alberta TRANSPORTATION

UNIFIED SOIL CLASSIFICATION SYSTEM
(Modified by PFRA)

4.4 Unified Soil Classification System

MAJOR DIVISIONS			GROUP SYMBOLS	LOG SYMBOLS	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (More than half of material is larger than No. 80 sieve size)	GRAVELS (More than half of coarse fraction is larger than No. 5000 sieve size)	CLEAN GRAVELS (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine amount of sand and gravel from grained size curve. Depending on percent of fines (fraction smaller than No. 80 sieve size), coarse-grained soils are classified as follows: Less than 5% ---- GW, GP, SW, SP; More than 12% ---- GM, GC, SM, SC; 5 to 12% ---- Borderline cases requiring dual symbols**	$C_u = \frac{D_{60}}{D_{10}}$ is greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ is between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		GRAVELS WITH FINES (Appreciable amount of fines)	GM* d / u		Silty gravels, gravel-sand-silt mixtures		Atterberg Limits below "A" Line or P.I. is less than 4	Above "A" Line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg Limits above "A" Line with P.I. greater than 7	
	SANDS (More than half of coarse fraction is smaller than No. 5000 sieve size)	CLEAN SANDS (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ is greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ is between 1 & 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradations requirements for SW	
		SANDS WITH FINES (Appreciable amount of fines)	SM* d / u		Silty sands; sand-silt mixtures		Atterberg Limits below "A" Line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
			SC		Clayey sands; sand-clay mixtures		Atterberg Limits above "A" Line with P.I. greater than 7	
FINE GRAINED SOILS (More than half of material is smaller than No. 80 sieve size)	CLAYS (Above "A" Line on PLASTICITY CHART, negligible organic content)	W_L<30%	CL		Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays	PLASTICITY CHART (see below)		
		30<W_L<50 %	CI		Inorganic clays of medium plasticity, gravelly clays, sandy clays, silty clays			
		W_L>50%	CH		Inorganic clays of high plasticity, fat clays			
	SILTS (Below "A" Line, negligible organic content)	W_L<50%	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity			
		W_L>50%	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
	ORGANIC SILTS AND CLAYS (Below "A" Line)	W_L<50%	OL		Organic silts and organic silty clays of low plasticity			
		W_L>50%	OH		Organic clays of medium to high plasticity, organic silts			
HIGHLY ORGANIC SOILS			Pt		Peat and other highly organic soils	Strong colour or odor and fibrous textures		



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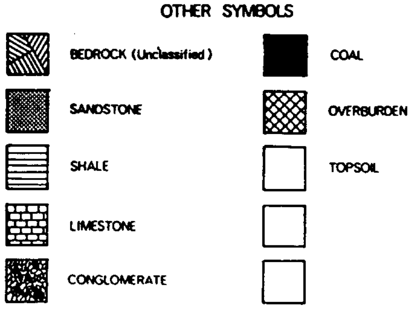
OTHER SYMBOLS

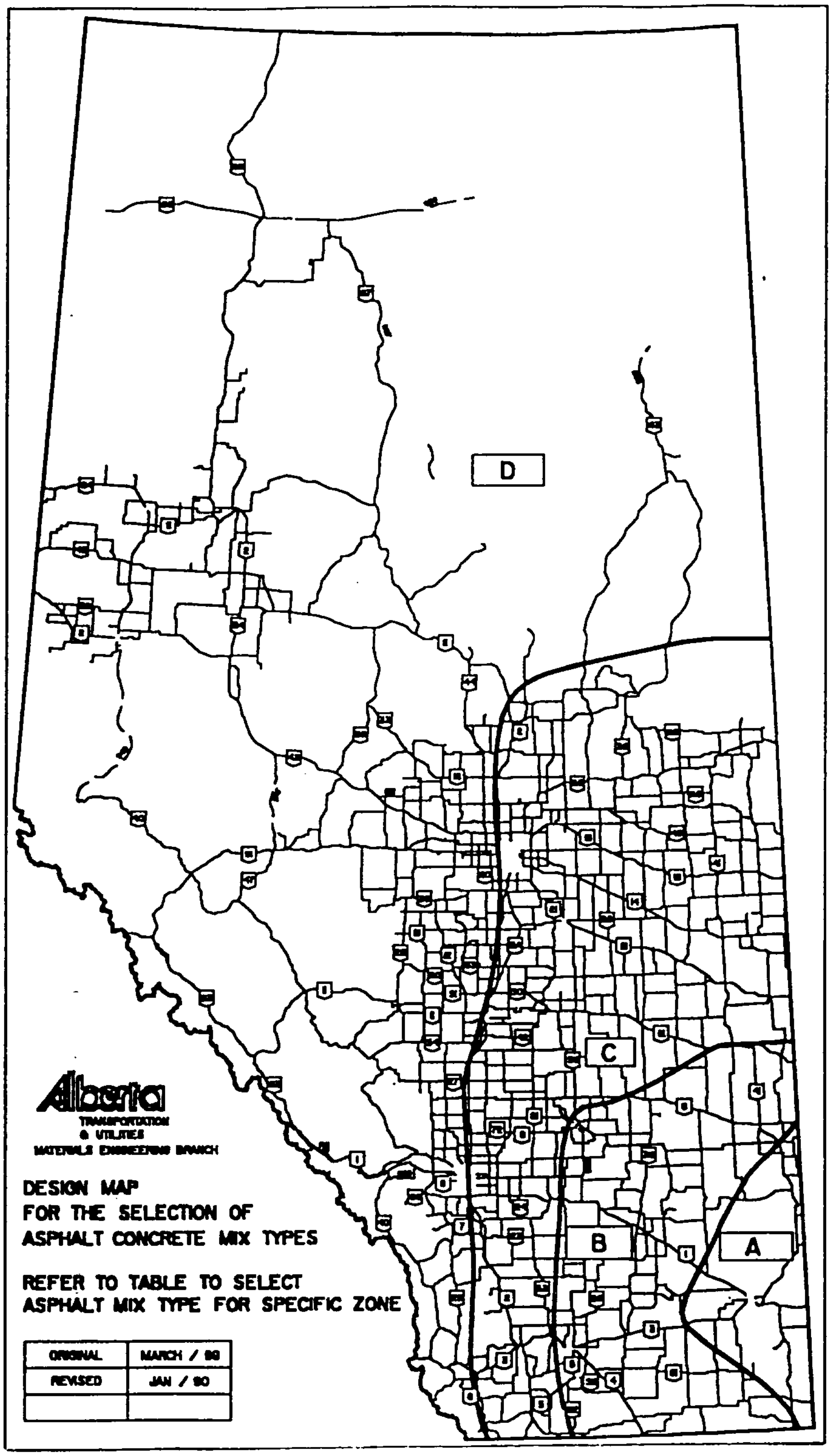


NOTE

* Division of GM and SM groups into subdivisons of d and u are for roads and airfields only. Subdivison is based on Atterberg Limits; suffix d is used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

** Borderline classifications, used for soils processing characteristics of two groups, are designated by combination of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

Figure 2.1 Modified Unified Classification System for Soils



DESIGN MAP FOR THE SELECTION OF CLIMATIC ZONE
FOR ASPHALT MIX TYPE DETERMINATION

Figure 2.2 AT&U Design Map for the Selection of Asphalt Concrete Mix Types

TABLE 2.2 AT&U ESAL CRITERIA FOR SELECTION OF ASPHALT CONCRETE MIX TYPES

ZONE	ASPHALT MIX TYPE					
	1	2	3 or 4	5	6	7[1]
A	>2.0	1.0 - 2.0	0.5 - 1.0	0.3 - 0.5	<0.3	-
B	>3.0	1.5 - 3.0	0.7 - 1.5	0.4 - 0.7	<0.4	-
C	>4.0	2.0 - 4.0	1.0 - 2.0	0.5 - 1.0	0.2 - 0.5	<0.2
D	>5.0	2.5 - 5.0	1.5 - 2.5	0.8 - 1.5	0.3 - 0.8	<0.3

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[1] Also for Community Airports.

ESAL criteria given are the total ESALs ($x10^6$) in the design lane that will be applied to the asphalt pavement over its design life.

TABLE 2.3 AT&U ASPHALT CONCRETE MIX TYPES AND CHARACTERISTICS

TABLE 3.50.3.2
ASPHALT CONCRETE MIX TYPES AND CHARACTERISTICS

Asphalt Concrete Mix Type	Class for Des 1 Aggregate	% MF, -5000 (Min) (Note 1)	% Fractures +5000 (2 faces)* (Min)	Asphalt Cement Grade	Marshall Stability N (Min)	No. of Blows	Air Voids %	VMA % (Min.) by % Air Voids			Voids Filled with Asphalt %	Flow mm	Retained Stability % (Min.)
								3	4	5			
1	16	80	98 (one face) 90 *	150-200A	12 000	75	3 to 5	12.5	13.5	14.5	65 to 75	2 to 3.5	70
2	16	70	70 *	150-200A	12 000	75	3 to 5	12.5	13.5	14.5	65 to 75	2 to 3.5	70
3	16	40	60 *	150-200A	8 000	75	3 to 5	12.5	13.5	14.5	65 to 75	2 to 3.5	70
4	12.5	50	60 *	150-200A	8 000	75	3 to 5	13	14	15	65 to 75	2 to 3.5	70
5	12.5	Note 2	60 *	200-300A	8 000	75	3 to 5	13	14	15	65 to 75	2 to 3.5	70
6	12.5	Note 2	60 *	200-300A	5 300	50	3 to 5	13	14	15	65 to 78	2 to 4	70
7	12.5	Note 2	60 *	300-400A	5 300	50	Note 5	13	14	15	65 to 78	2 to 4	70
8	10	Note 2	60 *	Note 3	5 300	Note 4	3 to 5	14	15	16	65 to 78	2 to 4	70

Note 1 - The Percentage of Manufactured Fines in the -5000 Portion of the Combined Aggregate.
Note 2 - All fines manufactured by the process of crushing shall be incorporated into the mix for Asphalt Mix Types 5, 6, 7 and 8.
Note 3 - Use the same asphalt grade as for the lift above.
Note 4 - Use the same number of blows as for the surface course.
Note 5 - Air Voids of 3 to 5 % except for 2 to 4 % on community airports, (VMA at 2 % air voids shall be a minimum of 12 %).
Note 6 - Mix types 1 to 5 shall have a minimum Theoretical Film Thickness value of 6.0 μm and mix types 6 and 7 shall have a minimum Theoretical Film Thickness value of 6.5 μm. The Theoretical Film Thickness value shall be established in accordance with TLT-311.

General requirements for mix design:

- It is recommended that the design asphalt content be determined at 4 % air voids (3% for community airports), which is the mid-point of the design air voids. The test properties at this asphalt content are then checked to ensure compliance with the respective criteria.
- A minimum of four specimens shall be prepared at each asphalt content
- Theoretical maximum specific gravity shall be determined in duplicate for at least three asphalt contents
- Retained Stability after 24 hour soaking at 60° C to be run at the recommended design asphalt content.

3 DRAINAGE

3.1 Cross-section Geometry

Highway engineers have long recognized the critical necessity of good drainage in the design and construction of pavements. Drainage is an important feature in determining the ability of a given pavement structure designed for specific traffic conditions to withstand the effects of traffic and the environment. Drainage affects the strength and behaviour of both the subgrade soil and the granular base course, and to a lesser extent, the durability of the asphalt concrete.

Highway drainage may be considered in two categories, surface and subsurface, each of which are separately treated. Surface drainage, as a result of rainfall and/or snow melt, is dealt with by incorporating the following minimum requirements:

1. Cross slope on driving lanes and shoulders of 2%.
2. Utilization of densely graded asphalt concrete mixtures with low permeabilities.
3. Routine maintenance procedures to seal surface cracks.
4. Design of roadway subgrade crown minimum of 1.0 m above ditch elevation.
5. Utilization of free draining granular materials within the pavement structure.
6. Lateral extension of the granular materials through the shoulder to drain out onto the side slope.

In rehabilitation projects where road widening and/or flattening of side slopes is a part of the rehabilitation strategy, lateral drainage is most important to maintain. In order to provide this drainage the granular drainage layer must be extended to at least the depth of the existing subgrade. The use of granular sub-base such as pit run is an appropriate materials.

More detailed information is provided in the AT&U 1996 Highway Geometric Design Guide [AT&U 96].

Subsurface drainage is a somewhat less required circumstance in Alberta. However on occasion the need arises in selected situations involving a high water table, active springs, or significant infiltration of surface water.

3.2 Properties of Materials

The permeability of granular materials used as base course or subbase course is dependent upon grain size distribution, grain shape and relative density. Of these factors, grain size including the percentage of fines is of major significance. In general, the lower the percentage passing the 80 µm sieve size while still allowing sufficient fines for construction compactibility, the better the drainage characteristics of the material. It is also mandatory that the granular material be drained through the shoulder so as not to impede the escape of water from the granular base course or subbase zone. Notwithstanding the above criteria, the AASHTO pavement design procedures for new construction takes into account duration of the time required to drain the GBC layer as well as the frequency and time period that the pavement materials are exposed to saturated conditions. AASHTO provides the following definitions corresponding to various drainage levels from the pavement structure:

Quality of Drainage	Water Removal Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	no drainage

3.3 Moisture and Rainfall

Moisture that exists within a pavement structure or within the subgrade soils beneath a pavement may be generated from many sources including:

1. Cracks in the pavement surface.
2. A permeable pavement surface.
3. Pavement side slope.
4. Lateral movement from the shoulder.
5. Near surface water table.

The effect of rainfall on performance is related to intensity since surface runoff is lowest and moisture absorption highest under conditions of prolonged low intensity rainfall. A map showing 30 year precipitation normals for Alberta is presented in Figure 3.1

Spring rainfalls coupled with freezing temperatures typically represent a negative set of conditions. Although rainfall per se is not a design criteria, the design must account for weakened subgrade conditions taking into consideration the time required for drainage of gravel base layers as described above .

Additional measures currently employed in Alberta include the utilization of adequate (2%) cross slopes, paved shoulders, and daylighting of GBC to the side slopes.

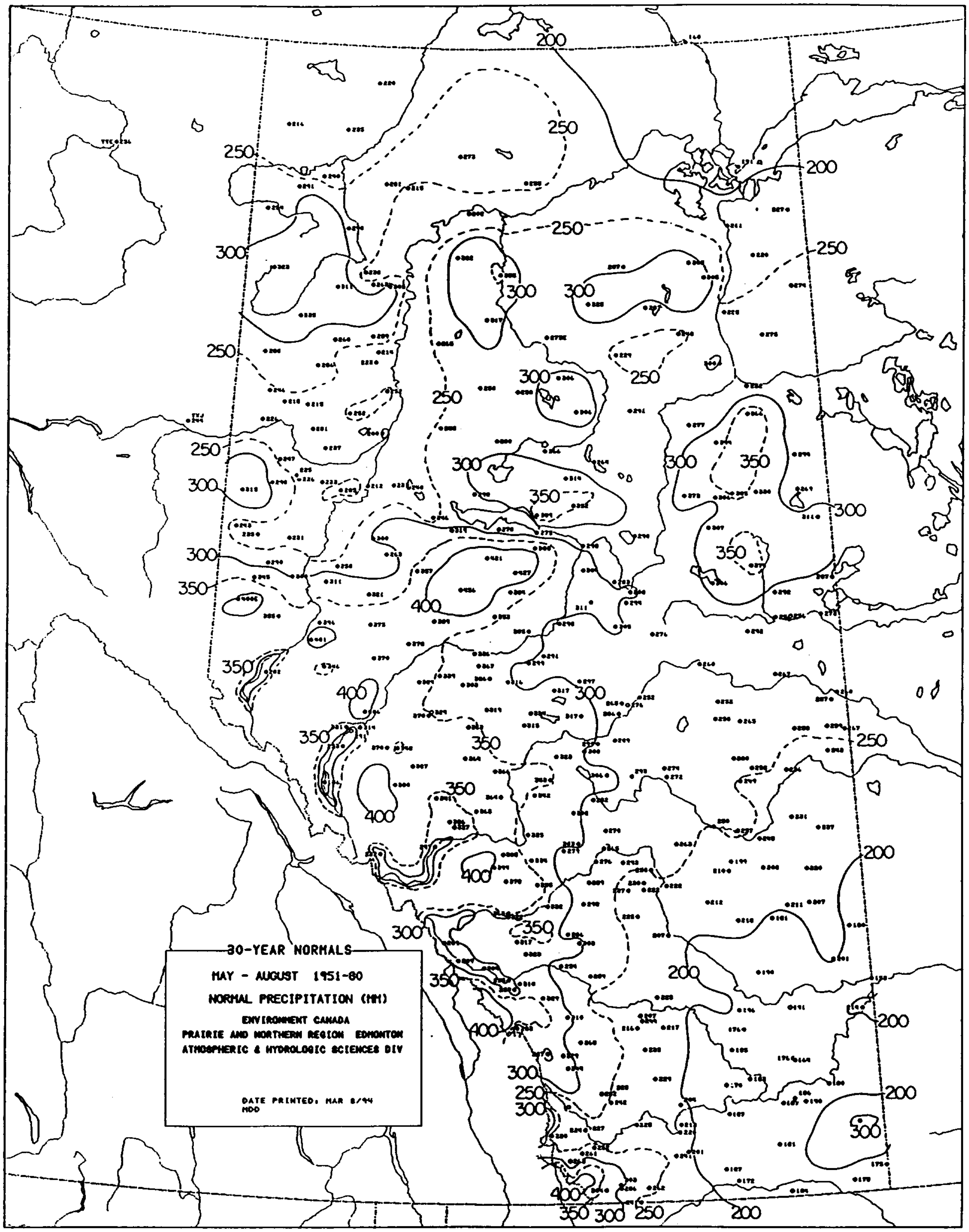

Figure 3.1 30-Year Precipitation Normals

4 PAVEMENT EVALUATION

4.1 Introduction

In order to carry out design of final stage pavements or pavement rehabilitation, the existing pavement condition must be evaluated. Such an evaluation usually involves the assessment of the existing pavement structural adequacy, surface distress, roughness, rutting, and to lesser extent, skid resistance. The design of final stage pavements may only involve the assessment of pavement structural adequacy because the first stage pavement is usually not old enough to exhibit distresses related to traffic loading and the environment.

Pavement evaluation techniques differ between different highway agencies. Within AT&U, emphasis has been placed on carrying out structural evaluation of pavements using non-destructive deflection testing. Another consideration which should be recognized is that AT&U was one of the first highway agencies in North America to develop and implement a Pavement Management System (PMS). The AT&U PMS database provides a wealth of information about the pavement network and individual pavement section performance. This information should not be overlooked during the pavement evaluation stage of pavement rehabilitation design.

At present AT&U is maintaining the following pavement performance data:

- $ structural strength information
- $ roughness information
- $ visual distress data
- $ rut data

This information is collected to provide input for:

- $ final stage pavement and rehabilitation design
- $ the review of the structural and functional performance of various pavement sections
- $ operational research
- $ the Pavement Management System
- $ Rehabilitation Programming.

AT&U=s current Pavement Management System uses Riding Comfort Index (RCI), Structural Adequacy Index (SAI), Visual Condition Index (VCI) and Pavement Quality Index (PAI) to measure, monitor and predict the condition of highway network. The Pavement Quality Index (PQI) is a composite index that incorporates RCI, SAI and VCI into one index in the following relationship.

$$PQI = 1.1607 + 0.0596\ (RCI \times VCI) + 0.4264\ (RCI \times \log_{10} SAI).$$

The PMS uses PQI for identifying pavement sections that are in need of rehabilitation and for monitoring and predicting network performance and budgetary needs.

All four indices are scaled from 0 to 10; 0 being very poor to absolute minimum, and 10 being very good to near perfect. AT&U’s PMS uses minimum acceptable levels (also known as critical levels) of these indices to flag sections needing rehabilitation or treatment. The values for minimum acceptable levels are:

PQI	4.7
RCI	5.5
SAI	3.0
VCI	3.5

4.2 Pavement Evaluation

4.2.1 Pavement Structural Adequacy

In Alberta, the evaluation of the existing pavement strength has been carried out using pavement deflections. Deflection testing has been a standard evaluation procedure by AT&U for the last 35 years. The oldest of the deflection testing techniques was with the Benkelman beam. This procedure was discontinued in 1991. Since the mid-1960s the Department was also using Dynaflect testing to measure pavement deflections. The Dynaflect deflections were converted to the equivalent Benkelman beam rebounds using AT&U developed correlation equations.

In 1989 AT&U acquired its first Falling Weight Deflectometer (FWD) and since 1992 pavement structural evaluation has been exclusively performed based on FWD deflection data. At that time a decision was also made to replace the structural overlay design system based upon Benkelman beam rebound deflections with a simplified mechanistic-empirical design procedure based on the interpretation of the FWD deflection basin using the ELMOD computer program.

The Falling Weight Deflectometer (FWD) is a trailer-mounted device that measures the deflection of the pavement under a simulated vehicular load. The load is obtained by dropping a weight on a specially designed spring system and a steel plate placed on the pavement. Such a drop can produce an impact load of 25 - 30 millisecond duration, and a peak force of up to 120 kN. Research and practice indicates that stress and strain conditions developed in the pavement by the FWD are very similar to the conditions under a heavy wheel load [Ullidtz 87].

At present there are several FWD manufacturers: Dynatest, KUAB, Foundation Mechanics, and others. All FWD units used on AT&U projects are of the Dynatest 8000 type.

The FWD testing procedure is fully automated and computerized. The unit is run by one technician, who operates the machine through a computer in the pulling vehicle. Dynatest uses several software (field programs) to operate the units. The units operated in Alberta are currently using the 25.03 release of the Dynatest Field Program. The collected data are stored in computer files. The structure of the files is in test form and corresponds to the release of the field program and the selected format of the data output.

The testing procedures used for various types of FWD testing is described fully in the AT&U Falling Weight Deflectometer (FWD) Field Operation Manual Version 2.1 [AT&U 92].

The FWDs used in Alberta are equipped with 9 deflection sensors. The sensors are located in the following offset distances from the centre of the load plate: 0, 200, 300, 450, 600, 900, 1200, 1500 and 1800 millimetres. For both inventory and project level testing the load plate radius used is 150 mm. At each tested location four FWD drops from different heights are performed. The first drop is for seating the plate on the pavement and is not recorded. The following three drops are performed from such heights to result in the following loads being applied to the pavement: 26.7 kN, 40.0 kN, and 53.3 kN. All these loadings are recorded with targeted load tolerances of +/- 10 percent.

Presently ten FWD tests per kilometre are performed for inventory and project level testing. The tests are taken in the outside wheel path. The tests are performed with 100 metre intervals and are staggered on both lanes of the highway. On divided highways, the tests are staggered over the inside and outside travel lanes.

During the FWD testing, the in-situ pavement temperatures and air temperatures are recorded. The in-situ pavement temperature is obtained by drilling a 50 mm hole in a pavement, filling the hole with oil and inserting a thermometer. The reading is recorded after a minimum of 15 minutes and input into a FWD field program. The air temperature is measured automatically by installed sensors and automatically input into the field program.

The ELMOD computer program was used by AT&U to analyze the FWD deflection data for the design of final stage pavements and overlays. With the adoption of the AASHTO method by AT&U, the DARWin 3.0 [AASHTO 97] computer program has been adapted to process and analyze the FWD deflection data,. DARWin stands for Design, Analysis, and Rehabilitation for Windows. DARWin 3.0 is designed to operate under the Windows 95/NT environment. It will be necessary for consultants engaged in the preparation of pavement designs to acquire the DARWin 3.0 software. The program and Users= guide can be purchased through ERES Consultants, Inc., Toll Free (888)443-2794, FAX (217)356-3088 or E-mail darwin@eresnet.com. The procedures for the design of final stage pavement and rehabilitation projects are described in Sections 9 and 10 respectively.

4.2.2 Pavement Ride Quality (Roughness)

Roughness is an indication of the longitudinal irregularities of the pavement surface which influence vehicle ride. Roughness is a good indicator of how well the road is serving the travelling public.

Studies have shown that roughness affects the costs of vehicle operation and the life cycle costs of pavements. A significant portion of pavements are rehabilitated due to an unacceptable ride rather than the structural problems. For this reason roughness monitoring constitutes an important rehabilitation design input.

AT&U collects the Riding Comfort Index (RCI) data by utilizing the James Cox Roadmeter. In addition, the Automatic Road Analyzer (ARAN) was used from 1993 to 1995 to provide RCI data. The RCI is an index on the scale of 0 to 10, with 10 representing a smooth pavement providing an excellent ride. The Department is in the process of changing to the International Roughness Index (IRI).

The Cox Roadmeter is mounted on a trailer and towed by a van. The van is equipped with an operator=s control panel which enables the control and monitoring of the testing operation. The data are collected continuously at a traffic speed of 80 km/hr. All pertinent information for the section of roadway being tested is preprogrammed prior to conducting the test. The data collected are recorded on a cassette tape. The field data from the cassette tape are subsequently down loaded onto a main frame computer and analysed. The analysed information is usually averaged over 500 metres and presented in graphical and tabular format. The RCI data can be averaged over different intervals, if required.

It was found in Alberta that the RCI of new pavements is close to 8.5 - 9, while older pavement sections rarely have RCI values lower than 4. The RCI trigger value for rehabilitation due to roughness has been set at 5.5 by AT&U.

Typical RCI plots are presented in the design examples in Section 10.

4.2.3 Pavement Condition (Distress) Survey

The surface condition of a pavement at any time reflects the degree of damage caused by traffic and the environment based upon a visual evaluation of the pavement surface. The surface condition rating is useful as an input for predicting the remaining life of a pavement. It also assists in the preliminary evaluation and programming of appropriate maintenance and rehabilitation treatments.

Currently two procedures of pavement condition survey are used by AT&U. These procedures provide Visual Condition Index (VCI) for the PMS and quantities and severities of rutting, cracking and transverse cracking for the Maintenance Management System (MMS).

These condition surveys carried out by the Department would not normally be used by the pavement designer. It would be necessary for the designer to carry out a field evaluation at the time of the design of the pavement to confirm pavement performance and to collect other inputs required for the detailed design and estimation of quantities for the project.

4.2.4 Pavement Safety Evaluation

Pavement Rutting

Ruts are longitudinal depressions in the pavement wheel paths created by repeated repetitions of heavy traffic. Ruts can result from the densification of pavement layers and the subgrade under the traffic loads. More often, rutting can result from the deformation and shoving of the asphalt concrete layers under truck traffic. There are several possible causes of rutting including unstable granular and subgrade material, unstable asphalt mixes, and unstable shoulder material. Rutting may be accompanied by longitudinal cracking in the wheel paths. Rutting may be considered as both structural distress and a safety issue.

In Alberta, rut measurements are carried out for the purposes of pavement evaluation and maintenance activities. The procedures and the interval of rut measurements are different for each purpose.

Network rut measurements were carried out prior to 1996 with use of ARAN. The rut bar of ARAN operated by sending ultrasonic signals toward the pavement surface. The signals reflected back to a receiver and based on the time elapsed from sending and receiving the signal the distance from the rut bar to the pavement surface was calculated. All this information was stored on the onboard ARAN computer. The rut bar consisted of seven ultrasonic sensors distributed at one-foot intervals along the bar. In addition, extension wings with 2 or 3 sensors could be attached to the main bar. AT&U used the rut bar with two wings (2-foot on the centerline side and 3-foot on the shoulder side). In such a rut bar configuration there were 12 ultrasonic sensors. The distance interval between measurement was typically 10 metres. The rut depth output was reported as the mean rut depth in each wheel path over a distance equal to the sample interval and averaged over a specified 100 metres length. All rut measurements were carried out at a speed of 80 km/h. Based on ARAN rut data, a plot of rut depth in each wheel path over the length of pavement section was produced. Examples of the ARAN produced rut profiles are presented in the design examples in Section 10.

In 1996, AT&U collected the roughness data and rut data with the RT 3000 van. This equipment has a rut bar equipped with ultrasonic sensors and receivers, which can simultaneously collect rut measurements and the IRI (International Roughness Index).

For PMS purposes the rut depth is classified as:

$	minimal	0 to 3 mm
$	minor	4 to 6 mm
$	moderate	7 to 12 mm
$	major	13 to 25 mm
$	severe	over 25 mm.

For MMS purposes 10 measurements are taken in each 50-metre long gauging unit with a 1.2 metre straight edge and a wooden wedge.

The severity of rutting for MMS purposes is classified as:

- $ slight rut depth of 5 to 10 mm
- $ moderate rut depths of 11 to 15 mm
- $ extreme rut depths greater than 15 mm.

Pavement Skid Resistance

When the aggregate in the surface of pavement becomes smooth or when there is bleeding or flushing, i.e. the presence of free asphalt binder on the pavement surface, the pavement skid resistance may be reduced. Several methods are used by agencies to evaluate the skid resistance of in-service pavements. AT&U has used a Mu-Meter skid trailer for this purpose.

The trailer is equipped with two freely rotating test wheels that are angled to the direction of the motion. The test is conducted at a constant speed of approximately 65 km/h. During the test the pavement is wetted under the testing wheels. The friction factor measurements are reported as the Skid Number. The Skid Number is equal to 100 times the friction factor and can be described as:

- $ 46 or greater adequate friction characteristics
- $ 45 to 30 medium friction characteristics
- $ 29 or lower poor friction characteristics.

4.2.5 PMS Reports

Several reports can be produced by AT&U=s PMS. Out of these reports three are very commonly used by pavement engineers. These reports are:

- $ PMS Data Base
- $ PMS Cross-sections
- $ Status Summary Report

PMS Data Base

The PMS Data Base contains extensive historical information regarding construction, performance and rehabilitation of the Alberta=s Primary and Secondary Highway Systems. The database includes information dating back to 1959 and is probably the most comprehensive pavement database in North America. The database contains the following information:

- $ highway control section number, length, and location
- $ pavement inventory section from - to and width of pavement
- $ ST soil type (soil classification is Unified System as modified by P.F.R.A {similar to ASTM D-2487})
- $ pavement layer types and thicknesses (all thicknesses in inches):
  - GB granular base thickness
  - SC soil-cement base thickness
  - CS cement-treated base thickness
  - A asphalt stabilized base course thickness
  - AC asphalt concrete thickness
  - T type of asphalt cement used
  - SA sulphur-asphalt layer thickness
- $ traffic data
  - ESAL cumulative Equivalent Single Axle Loads per day per direction

(these data should not be used for design purposes)

  - AADT Annual Average Daily Traffic
- $ pavement evaluation data
  - RCI Riding Comfort Index
  - VCR Visual Condition Rating
  - Benkelman Beam Data (since 1992, FWD central deflections have been converted to equivalent Benkelman beam rebounds):
    - MM month when deflection testing was carried out
    - DD day of month when deflection testing was carried out
    - PT pavement temperature (/C)
    - DBAR mean equivalent Benkelman beam rebound for a section
    - DMAX mean plus two standard deviation of equivalent Benkelman beam rebounds for a pavement section
  - SKD Mu-Meter skid data
  - RUT ARAN rut data (since 1989)
- $ CM digital comments

A typical example of the PMS data base is presented in Figure 4.1.

PMS Cross-sections

In addition to the PMS Data Base, AT&U also updates the pavement cross-section information. The updates have been performed manually and provide useful information to the pavement designer. The cross-section information is especially valuable to identify cases of pavement widening, shifting of centerline, where asphalt concrete inlays were constructed, pavements with different thicknesses for each lane etc.

PMS Status Summary

The PMS Status Summary is produced by AT&U every year based on yearly update of the PMS inventory data base and Pavement Inventory Need System report (PINS).

These reports include the following information:

Field(s)	**Description**	**Explanation**
INV	Inventory Sections	are specific to data base and may vary from year to year. They are simply a method of indicating a change in structure or other attribute.
LN	Lane	is used to sort out the lanes in multi-lane highways. A blank here means a two-lane highway and the A&@ character means that all lanes are combined for one direction of a multi-lane highway.
SOIL TYPE		refers to AT&U records of the general type of soil to be expected in the subgrade of the given Inventory Section.
GB, SC, CS SA, OB, FIRST ACP, O/L	Layer Depths	Since the data base still requires entries to be made in Imperial units, the figures shown in here have been converted to metric. A base depth of A254 mm@, for example, represents A10 inches@.

Field(s)	**Description**	**Explanation**
PRESENT CONDITION		should be interpreted with extreme caution. The various indices are brought up to date for each Inventory Section when it is tested or re-tested.
SKID	Skid Numbers	represents the results of spot-testing in wet conditions with the AMu-Meter@, and gives an idea of the dynamic Coefficient of Friction between the road surface and a standardized tire. The collection of skid data has been discontinued since 1993.
RUT	Rut Depths	refer to average for the given Inventory Section. They will not reflect isolated deep ruts.
PREDICTED NEED		These are open to misinterpretation. Here Aneeds@ means simply that one or more of the performance indices for the given Section is expected to move down to the Aminimum acceptable value@ in the year quoted. This may or may not be the year for rehabilitation.
ESAL AND		Traffic figures are taken directly (and electronically) AADT from the database maintained by Planning and Programming Branch. They are updated when new information becomes available. In the case of ESALs, this is about once every two years.

This report is a quick reference, for use on field trips or in conversations over the phone. The need year is not intended for making decisions on construction or rehabilitation matters without referring to other, much more reliable and detailed information.

A typical example of a PMS Status Summary is presented in Figure 4.2.

```
42938     8 06  0.00 16.58        JCT 22 TO WCL CALGARY       1  4   .
          ===================================================================
          YY|ST|GB|SC|CS|A| AC |T| AC |T|ESAL|RCI|VCR|MM|DD|PT|DBAR|DMAX|SA| AADT|SKD|CM|RUT|

42939     0.00  6.00   (DESIGN) 5723/96                                    12.5 1
42940     81 21 10              4.3 1     35
42941     82                              70           6  7 24 .031 .041      2050
42942     83                             130 7.3  71                          2730 58
42943     85                             291           8 14 18 .028 .038      3390
42944     86                             393 6.5                              3390 70 1
42945     87                             532 6.7  65                          4180
42946     89                             757 6.4                              4820
42947     90                             843           6 11 20 .040 .066      4820 81
42948     91                             943 6.5                              5600        5
42949     94                            1597 5.1       9 26 17 .038 .047      5880
42950     95                            2141 5.3                              6310        6
42951     96                    4.7 1   2685 9.0  90                          6310
42952     97                            2686           8 15 25 .030 .040      6310
42953     6.00  9.00   (DESIGN) 5723/96                                    12.8 1
42954     81 21 10              4.3 1     35
42955     82                              70           6  7 24 .031 .041      2050
42956     83                             130 7.3  71                          2730 58
42957     85                             291           8 14 18 .028 .038      3390
42958     86                             393 6.5                              3390 70 1
42959     87                             532 6.7  65                          4180
42960     89                             757 6.4                              4820
42961     90                             843           6 11 20 .040 .066      4820 81
42962     91                             943 6.5                              5600        5
42963     94                            1597 5.1       9 26 17 .038 .047      5880
42964     95                            2141 5.3                              6310        6
42965     96                    3.5 1   2685 9.0  90                          6310
42966     97                            2686           8 15 25 .030 .040      6310
42967     9.00 14.40   (DESIGN) 5723/96                                    11.9 1
42968     81 21 10              4.3 1     35
42969     82                              70           6  7 24 .031 .041      2050
42970     83                             130 7.3  71                          2730 58
42971     85                             291           8 14 18 .028 .038      3390
42972     86                             393 6.5                              3390 70 1
42973     87                             532 6.7  65                          4180
42974     89                             757 6.4                              4820
42975     90                             843           6 11 20 .040 .066      4820 81
42976     91                             943 6.5                              5600        5
42977     94                            1597 5.1       9 26 17 .038 .047      5880
42978     95                            2141 5.3                              6310        6
42979     96                    3.5 1   2685 9.0  90                          6310
42980     97                            2686           8 15 25 .030 .040      6310
42981     14.40 15.80  (DESIGN) 5723/96                                    11.9 1
42982     81 15 10              6.3 1     35
42983     82                              70           6  9 23 .039 .070      3000
42984     83                             130 6.2  69                          2730 58
42985     85                             291           8 14 19 .026 .031      3390
42986     86                             393 6.5                              3390 70 1
42987     87                             532 6.9  75                          4180
          YY|ST|GB|SC|CS|A| AC |T| AC |T|ESAL|RCI|VCR|MM|DD|PT|DBAR|DMAX|SA| AADT|SKD|CM|RUT|
```

Figure 4.1 Example of AT&U PMS Data Base

ALBERTA TRANSPORTATION AND UTILITIES
ROADWAY ENGINEERING

PRIMARY HIGHWAY STATUS SUMMARY (1994 PMS DATA)

HWY	CS	INV	LN	FROM KM	TO KM	LENGTH IN KM	WIDTH IN M	SOIL TYPE	GB MM	SC MM	CS MM	SA MM	OB MM	YR BASED	FIRST ACP MM	YR FIRST PVED	O/L MM	YR O/L	YR SEAL COAT	PRESENT CONDITION POI	RCI	SAI	VCI	SKID	RUT MM	PREDICTED NEED YR	NEED PARM	PRESENT ESAL (/DAY)	AADT
7	08				JCT 22	TO JCT 2A																							
7	08	1		0.00	0.82	0.82	6.7	CL	51	0	0	0	102	51	102	56	51	67		4.5	4.8	4.8	5.6	63	5	1993	RCI	112	2760
7	08	2		0.82	1.94	1.12	11.9	CL	254	0	0	0	0	89	109	89	0			6.0	6.2	4.1	7.8		7	2001	RCI	112	2760
7	08	3		1.94	2.40	0.46	11.9	CL	203	0	0	0	51	55	102	56	51 79	67 89		7.0	6.8	6.5	7.2		5	2007	RCI	112	2760
7	08	4		2.40	4.26	1.86	11.9	CL	254	0	0	0	0	89	109	89	0			6.1	6.7	3.9	7.2		7	2003	SAI	112	2760
7	08	5		4.26	5.52	1.26	11.9	CL	203	0	0	0	51	55	102	57	51 79	67 89		6.7	6.7	6.1	6.9		4	2007	POI	112	2760
7	08	6		5.52	6.22	0.70	11.9	CL	254	0	0	0	0	89	109	89	0			6.0	6.6	3.8	7.1		5	2002	SAI	112	2760
7	08	7		6.22	6.96	0.74	11.9	CL	203	0	0	0	51	55	102	57	51 79	67 89		6.9	6.8	6.7	7.0		5	2007	RCI	112	2760
7	08	8		6.96	8.76	1.80	11.9	CL	254	0	0	0	0	89	109	89	0			5.7	6.6	3.2	7.0		6	1998	SAI	112	2760
7	08	9		8.76	9.64	0.88	11.9	CL	203	0	0	0	51	55	102	57	51 79	67 89		6.7	6.7	6.6	6.7		5	2006	RCI	112	2760
7	08	10		9.64	10.27	0.63	11.9	CL	254	0	0	0	0	89	109	89	0			6.2	6.7	3.8	7.5		5	2002	SAI	112	2760
7	08	11		10.27	11.22	0.95	11.9	CL	51	0	0	0	152	51	102	57	51 79	67 89		6.8	6.8	6.1	7.0		4	2007	RCI	112	2760
7	08	12		11.22	14.52	3.30	11.9	CL	254	0	0	0	0	89	109	89	0			6.1	6.6	4.4	7.0		4	2004	POI	112	2760
7	08	13		14.52	23.27	8.75	11.9	CL	229	0	0	0	0	87	119	87	0			6.5	6.6	4.9	7.5	61	5	2006	POI	108	2387
7	08	14		23.27	25.40	2.13	11.9	CL	229	0	0	0	0	92	119	92	0			6.9	6.4	6.1	8.2		3	2003	RCI	114	2001
7	08	15		25.40	25.91	0.51	11.9	CL	203	0	0	0	0	87	119	87	0			5.6	5.5	5.3	7.3	60	5	1995	RCI	291	3330
7	08					25.91																							
7						25.91																							
8	06				JCT 22	TO WCL	CALGARY																						
8	06	1		0.00	14.40	14.40	12.5	CI-CH	254	0	0	0	0	81	109	81	0		86	4.9	5.6	4.1	5.7	81	5	1996	RCI	110	5880
8	06	2		14.40	15.80	1.40	12.5	CL	254	0	0	0	0	81	160	81	0		86	5.7	5.7	6.6	6.0	84	4	1997	RCI	110	5880
8	06	3		15.80	20.05	4.25	12.5	CI	254	0	0	0	0	81	109	81	0		86	5.0	5.5	5.1	5.6	78	5	1995	RCI	110	5880
8	06					20.05																							
8						20.05																							

Figure 4.2 Example of AT&U PMS Status Summary

5 TRAFFIC ANALYSIS

5.1 Introduction

Traffic-related data, which includes axles loads, axle configurations and number of applications, are required for both new construction and rehabilitation pavement structural design.

Cars and light truck traffic produce only small stresses in normal pavement structures and therefore truck traffic is the major consideration in the structural design of pavements. The project design ESALs are expressed as the cumulative Equivalent Single Axle Loads (ESALs) in the design lane for the design period..

The results of the AASHO Road Test indicated that the damaging effect on the pavement structure of an axle load of any mass can be represented by a number of 80 kN ESALs. For example, one application of a 100 kN single axle load would result in the same damage as 2.5 applications of an 80 kN single axle load. Conversely it would take about 62 applications of a 50 kN single axle load to result in the same damage as one 80 kN single axle load. This relationship characterizing the relative destructive effect of various axle loads in terms of equivalent 80 kN single axle loads is sometimes referred to as the AFourth Power Rule@. Load equivalency factors allow any axle group configuration and loading to be converted into an equivalent number of ESALs.

5.2 Estimating Design ESALs

5.2.1 Sources of Traffic Data

Statistics for traffic volumes and Single Unit Trucks (SUT) and Tractor Trailer Combinations (TTC) on the Primary Highway System are published annually by AT&U [AT&U 96]. This report provides values for ESALs per day per direction for all Traffic Control Sections (TCS) monitored on the primary highway system. An example of a typical report is presented in Figure 5.1.

The determination of the ESALs per day per direction is related to the AADT:

$$\text{ESALs/day/direction} = \frac{\text{AADT}}{2}\left[\frac{(\%\text{SUT})}{100}\times 0.881+\frac{(\%\text{TTC})}{100}\times 2.073\right]$$

where AADT, Average Annual Daily Traffic, is the average daily two way traffic expressed as vehicles per day for the period of January 1 to December 31, and where the traffic volumes are assumed to be split 50:50 for both directions.

These data can be used directly as initial inputs to determine design ESALs. However, the following should be noted:

$ Although all types of trucks are counted, they are sorted into only two categories: Single Unit Trucks and Tractor Trailer Combinations.

$ The factors, in ESALs per unit, used to convert the SUT and TTC counts are 0.881 and 2.073 respectively and were recently updated [KPMG 95]. The previous factors used were established in 1983 and were 0.56 and 1.37 respectively. Legal load limits were changed in 1987/88 and it is probable that ESAL factors probably started to change shortly after.

$ Traffic statistics on Secondary Highways were not collected by AT&U since 1993 for the period of 1994 to 1996.

5.2.2 Lane Distribution Considerations

The reported ESALs/day/direction are based upon a 50:50 split between directions. There may be special cases where this does not hold true (such as more loaded trucks in one direction and more empty trucks in the other). In these special cases it may be necessary to confirm actual distribution through a count survey.

The statistics reported are for total ESALs/day/direction. On multilane highways, the total would represent all lanes in one direction. For 4 lane divided highway, an 85/15 split between the outside and inside lane can be used as guideline to determine the design. This split may require adjustment on some highways located near major urban areas.

In cases where directional distribution factors are used and the pavement structure is designed on the basis of distributed traffic, consideration should be given to the design of variable cross-sections. This is addressed in more detail in Section 8.

5.2.3 Traffic Growth

The average growth in traffic on the Alberta primary highway network is about 2.3% per annum. Guidelines for growth factors to be used to estimate design ESALs for a project are:

New Construction	5% per annum compounded
Rehabilitation and Twinning Projects	3% per annum compounded

These values may require adjustment based upon local conditions, future development, etc.

Growth can be accounted for in design using the Traffic Growth Factors presented in Table D.20 [AASHTO 93] or by calculation:

$$\text{Traffic Growth Factor (TGF)} = [(1 + g)^n - 1]/g$$

where g = rate/100 and is not zero and n is the design period in years. If the annual rate is zero, the growth factor is equal to the design period.

Alternatively, for a 20 year design period, the following factors can be used directly:

- for 3% growth TGF = 26.87
- for 5% growth TGF = 33.06.

These factors multiplied by the first-year ESALs estimate will give the total number of ESAL applications expected during the 20 year design period.

5.3 Summary

The process of determining the design ESALs to be used on a particular project is:

1. Determine the Design Period (n).
2. Determine present ESALs/day/direction.
3. Adjust ESALs/day/direction based on lane distribution considerations.
4. Select Traffic Growth Factor (TGF).
5. Calculate Design ESALs/lane for the design period = ESALs/day/direction x 365 x TGF.

ALBERTA PRIMARY HIGHWAY TRAFFIC VOLUME, VEHICLE CLASS, TRAVEL and ESAL REPORT 1996
Alberta Transportation and Utilities
Planning and Programming Branch

Produced 10-Mar-1997 By Intertech Systems Inc.

Hwy	CS	TCS	From	To	Length in Km	Traffic Volumes WAADT	Traffic Volumes WASDT	%PV	%RV	%BU	%SU	%TT	%CM	Travel MVKm Annual	Travel MVKm Summer	ESALs per direction SU	ESALs per direction TT	ESALs per direction Total
6	0	4	U.S. BORDER	WATERTON LAKES PARK BOUNDARY	23.330	280	590	74.6	15.1	1.0	8.2	1.1	10.3	2.4	2.1	10.1	3.2	13.3
6	0		U.S. BORDER	WATERTON LAKES PARK BOUNDARY	23.330	280	590	74.6	15.1	1.0	8.2	1.1	10.3	2.4	2.1	10.1	3.2	13.3
6	4	4	WATERTON LAKES PARK BOUNDARY	S OF 505 NW OF TWIN BUTTE	24.189	840	1100	81.6	2.3	2.0	7.3	6.8	16.1	7.4	4.1	27.0	59.2	86.2
6	4	8	N OF 505 NW OF TWIN BUTTE	PINCHER CREEK S.C.L.	16.991	990	1290	81.9	2.2	2.1	7.2	6.6	15.9	6.9	3.7	31.4	67.7	99.1
6	4	12	PINCHER CREEK S.C.L.	S OF 507 AT PINCHER CREEK SJ	0.620	2540	3090	84.6	6.1	0.5	5.8	3.0	9.3	0.6	0.3	64.9	96.1	161.0
6	4	16	N OF 507 AT PINCHER CREEK SJ	E OF 507 AT PINCHER CREEK NJ	1.659	3440	4190	88.7	2.5	0.9	4.0	3.9	8.8	2.1	1.1	60.6	139.1	199.7
6	4	20	N OF 507 AT PINCHER CREEK NJ	S OF 3 AT PINCHER CREEK	2.975	2870	3500	85.4	5.6	1.0	4.8	3.0	8.8	3.1	1.6	60.7	89.2	149.9
6	4		WATERTON LAKES PARK BOUNDARY	S OF 3 AT PINCHER CREEK	46.434	1134	1453	83.2	2.9	1.7	6.5	6.7	13.9	20.1	10.6	32.5	67.0	99.5
6			U.S. BORDER	S OF 3 AT PINCHER CREEK	71.764	856	1172	82.3	4.2	1.6	6.7	5.2	13.5	22.5	12.9	25.3	46.1	71.4
7	8	4	E OF 22 AT BLACK DIAMOND	W OF 2A & 783 S OF OKOTOKS	19.341	3440	3760	84.9	4.0	1.9	5.3	3.9	11.1	24.4	11.1	60.3	139.1	219.4
7	8	8	E OF 2A & 783 S OF OKOTOKS	W OF 2A N OF ALDERSYDE	5.951	2560	2830	83.0	3.4	0.7	7.8	5.1	13.6	5.6	2.6	88.6	136.4	225.0
7	8	12	E OF 2A N OF ALDERSYDE	W OF 2 & 547 N OF ALDERSYDE	0.809	4220	4750	79.9	1.9	1.0	8.8	8.6	18.2	1.2	0.6	159.9	376.2	536.1
7	8		E OF 22 AT BLACK DIAMOND	W OF 2 & 547 N OF ALDERSYDE	26.101	3268	3579	84.4	3.8	1.6	5.9	4.3	11.8	31.2	14.3	84.9	145.7	230.6
7			E OF 22 AT BLACK DIAMOND	W OF 2 & 547 N OF ALDERSYDE	26.101	3268	3579	84.4	3.8	1.6	5.9	4.3	11.8	31.2	14.3	84.9	145.7	230.6
8	6	4	E OF 22 NE OF BRAGG CREEK	CALGARY W.C.L.	16.560	6100	7150	87.2	2.7	0.5	3.6	6.0	10.1	37.0	18.1	96.7	379.4	476.1
8	6		E OF 22 NE OF BRAGG CREEK	CALGARY W.C.L.	16.560	6100	7150	87.2	2.7	0.5	3.6	6.0	10.1	37.0	18.1	96.7	379.4	476.1
8			E OF 22 NE OF BRAGG CREEK	CALGARY W.C.L.	16.560	6100	7150	87.2	2.7	0.5	3.6	6.0	10.1	37.0	18.1	96.7	379.4	476.1
9	2	4	N OF 1 & 797 N OF LANGDON	S OF 564 E OF DELACOUR	12.968	1660	1960	73.1	1.9	1.1	10.2	13.7	25.0	7.9	3.9	74.6	235.7	310.3
9	2	8	N OF 564 E OF DELACOUR	S OF 566 E OF KATHRYN	6.482	1690	2250	73.7	1.4	0.8	12.0	12.1	24.9	4.5	2.2	99.9	237.0	336.9
9	2	12	N OF 566 E OF KATHRYN	S OF 567 S OF IRRICANA	14.250	1930	2310	78.7	2.3	1.0	9.6	6.4	19.0	10.1	5.0	81.6	168.0	249.6
9	2	16	N OF 567 S OF IRRICANA	S OF 72 & 806 NE OF BEISEKER	11.563	2490	3010	80.6	3.6	1.1	6.2	6.1	15.4	10.6	5.3	68.0	209.1	277.1
9	2	20	E OF 72 & 806 NE OF BEISEKER	W OF 21 E OF BEISEKER	19.565	2320	3090	79.9	6.5	0.7	5.8	7.1	13.6	16.6	9.3	59.3	170.7	230.0
9	2		N OF 1 & 797 N OF LANGDON	W OF 21 E OF BEISEKER	64.868	2090	2599	78.2	3.9	0.9	7.9	9.1	17.9	49.6	25.6	72.7	197.1	269.8
9	4	4	E OF 21 E OF BEISEKER	W OF 836 S OF SHARPLES	11.420	1810	2400	81.3	4.5	0.8	4.8	8.6	14.2	7.6	4.2	36.7	165.1	201.8
9	4	8	E OF 836 S OF SHARPLES	W OF 840 N OF ROSEBUD	7.624	1890	2510	82.4	7.6	0.8	3.1	6.1	10.0	5.3	2.9	25.8	119.5	145.3
9	4	12	E OF 840 N OF ROSEBUD	W OF 841 W OF DRUMHELLER	16.734	2030	2700	85.9	2.2	1.0	5.3	5.6	11.9	12.4	6.9	47.4	117.8	165.2
9	4	16	E OF 841 W OF DRUMHELLER	DRUMHELLER W.C.L.	7.398	2460	3270	86.4	1.0	0.9	6.6	5.1	12.6	6.7	3.7	71.5	130.0	201.5
9	4		E OF 21 E OF BEISEKER	DRUMHELLER W.C.L.	43.176	2021	2666	84.4	3.4	0.9	5.0	6.3	12.2	31.9	17.7	44.5	132.0	176.5
9	6	8	N.C.L. OF DRUMHELLER	S OF MUNSON ACCESS	11.370	2920	3510	86.3	2.6	0.5	4.4	4.2	9.1	12.2	6.1	56.6	127.1	183.7
9	6	12	N OF MUNSON ACCESS	S OF 27 & 56 SE OF MORRIN	8.863	2390	2670	86.6	4.7	0.3	3.9	4.3	8.5	7.6	3.9	41.1	106.5	147.6
9	6		N.C.L. OF DRUMHELLER	S OF 27 & 56 SE OF MORRIN	20.233	2688	3230	87.6	3.4	0.4	4.2	4.2	8.8	19.9	10.0	49.7	117.0	166.7
9	8	4	E OF 27 & 56 SE OF MORRIN	W OF 849 N OF MICHICHI	13.071	2210	2600	81.9	6.5	0.3	3.7	7.6	11.6	10.6	5.2	36.0	174.1	210.1
9	8	8	E OF 849 N OF MICHICHI	W OF 851 NW OF DELIA WJ	9.810	2160	2530	80.3	6.3	0.3	4.6	8.5	13.4	7.8	3.8	43.8	190.3	234.1
9	8	12	E OF 851 NW OF DELIA WJ	W OF DELIA ACCESS	1.105	2250	2650	79.6	5.6	0.4	5.6	8.6	14.6	0.9	0.4	55.5	200.6	256.1
9	8	16	E OF DELIA ACCESS	W OF 855 W OF HANNA	20.145	2270	2670	81.0	9.8	0.4	2.6	6.2	9.2	16.7	8.2	26.0	145.9	171.9
9	8	20	E OF 855 W OF HANNA	W OF 862 W OF HANNA	1.631	2370	2790	80.8	2.8	0.4	5.1	10.9	16.4	1.4	0.7	53.2	267.8	321.0
9	8	24	E OF 862 W OF HANNA	W OF 36 E OF HANNA WJ	15.249	2620	3090	81.5	1.1	0.9	6.4	10.1	17.4	14.6	7.2	73.9	274.3	348.2
9	8		E OF 27 & 56 SE OF MORRIN	W OF 36 E OF HANNA WJ	61.011	2329	2740	81.2	5.9	0.5	4.3	8.1	12.9	52.0	25.6	44.1	195.5	239.6
9	10	4	E OF 36 E OF HANNA WJ	W OF 36 E OF HANNA EJ	2.591	2300	2700	80.3	5.3	0.9	2.8	10.7	14.4	2.2	1.1	28.4	255.1	283.5
9	10	8	E OF 36 E OF HANNA EJ	W OF 871 SE OF RICHDALE	14.924	1770	2100	78.8	7.9	0.5	3.2	9.6	13.3	9.7	4.8	24.9	176.1	201.0
9	10	12	E OF 871 SE OF RICHDALE	YOUNGSTOWN	29.126	1700	2030	78.8	7.5	0.4	2.7	10.6	13.7	18.1	9.0	20.2	186.8	207.0
9	10		E OF 36 E OF HANNA WJ	YOUNGSTOWN	46.641	1758	2090	78.8	7.5	0.5	2.9	10.3	13.7	30.0	14.9	22.4	187.5	209.9
9	12	2	YOUNGSTOWN	W OF 884 AT YOUNGSTOWN WJ	0.615	1700	2030	79.8	9.2	0.6	2.0	8.4	11.0	0.5	0.3	15.0	148.0	163.0
9	12	4	E OF 884 AT YOUNGSTOWN WJ	W OF 884 SE OF YOUNGSTOWN EJ	6.969	1500	1790	80.2	8.2	0.3	2.9	8.4	11.6	3.6	1.9	19.2	130.6	149.8
9	12	8	E OF 884 SE OF YOUNGSTOWN EJ	W OF 886 S OF CEREAL	23.534	1530	1830	77.0	7.7	0.5	5.2	9.6	15.3	13.2	6.6	35.0	152.2	187.2
9	12	12	E OF 886 S OF CEREAL	W OF 41 N OF OYEN	23.683	1590	1900	76.4	6.9	0.9	5.1	10.7	16.7	13.8	6.9	35.7	176.3	212.0
9	12		YOUNGSTOWN	W OF 41 N OF OYEN	55.021	1555	1858	77.2	7.4	0.7	4.8	9.9	15.4	31.3	15.6	32.9	159.6	192.5
9	14	4	E OF 41 N OF OYEN	W OF 899 W OF SIBBALD WJ	16.178	1500	1800	83.1	1.0	0.9	5.6	9.4	15.9	10.0	5.0	37.0	146.1	183.1
9	14	8	E OF 899 W OF SIBBALD WJ	W OF 899 W OF SIBBALD EJ	1.707	1530	1840	79.4	4.7	0.8	6.5	8.8	15.9	1.0	0.5	43.6	139.6	183.4
9	14	12	E OF 899 W OF SIBBALD EJ	SASKATCHEWAN BORDER	13.133	1420	1710	76.9	8.3	0.5	7.1	7.2	14.8	6.8	3.4	44.4	106.0	150.4
9	14		E OF 41 N OF OYEN	SASKATCHEWAN BORDER	33.018	1470	1768	80.6	4.0	0.7	6.2	8.5	15.4	17.8	6.9	40.1	129.5	169.6

Figure 5.1 Example of AT&U Traffic Report

6 ECONOMIC EVALUATION

6.1 Life Cycle Costs

At the project level there often exists a need to maximize economy and compare different pavement structure options. Life Cycle Costs (LCC) is a term coined to reference the economic analyses undertaken to compare either new construction or rehabilitation alternatives. LCC refer to all costs related to a highway over the life cycle of the pavement structure. These cost components include capital costs, maintenance costs, rehabilitation including overlay costs, as well as residual value, and user costs. User costs are generally more difficult to quantify compared to the other input costs, and for the purposes of this Manual, user costs are not generally included in the analysis.

Although the economic analysis will provide a basis for decision making, several additional factors need to be considered and together with LCC will allow a rational decision making process. These factors include, but are not limited to, geometrics, drainage, safety, climate, budgets, maintenance levels, materials availability, past experience with similar pavements, and good engineering judgement.

LLC analysis requires the following inputs:

<u>Initial Capital Costs</u>

Initial capital costs are the total sum of the investments to design and construct a highway or a highway improvement. The most significant items include grading, base course, surfacing, bridge structures, right-of-way, engineering, signing, and signals. These costs can be determined from current AT&U published unit price reports and modified for local conditions.

<u>Rehabilitation Costs</u>

Rehabilitation costs are associated with upgrading or overlaying a pavement when the riding quality or serviceability decreases to a minimum level of acceptability. Items include overlays, recycling, seal coats, and reconstruction. Rehabilitation cost inputs are similarly available from AT&U unit price reports.

Maintenance Costs

Maintenance costs are those costs that are essential to maintain a pavement investment at a specified level of service or at a specified rate of deterioration. For LCC analysis, items directly affecting pavement surface performance include crack filling, crack repair, grinding, and patching. Until a larger data base of cost information is developed, this input will have to be estimated by the pavement designer.

Residual Value

The residual value is the value of the investment or capital outlay remaining at the end of the analysis period. For a valid LCC analysis, the residual value must be included. Current AT&U policy for determination of residual value involves a simple calculation using a linear relationship for the value remaining in the last overlay. Therefore if an overlay is made at year 20, the residual value at the end of a 30 year analysis period (assuming a service life of 15 years) will be 5/15 x overlay cost. This cost is then discounted to year 0.

Analysis Period

The analysis period is the time period over which the economic analysis is conducted. For Alberta highways, an analysis period of 30 years is used to compare alternatives.

Design Period

The design period is the time period for which the pavement structure is designed to carry the anticipated traffic loadings. As a matter of policy, 20 years is used by AT&U and this would generally apply to the design of new construction, final stage and structural overlay pavements.

Service Life

The service life is the estimate of the time period for which the pavement will provide adequate structural and/or ride quality performance before rehabilitation is necessary. The estimated service lives of alternative strategies are critical inputs into LCC analysis.

Discount Rate

The discount rate is the rate of interest used to adjust future values to present values, normally taken as the difference between the prime interest rate and the rate of inflation. Current AT&U policy is to use 4% [AT&U 91].

6.2 Method of Economic Evaluation

There are several economic models applicable to the evaluation and comparison of alternative pavement design/rehabilitation strategies, all of which incorporate to varying degrees, future costs and/or benefits. A very basic method utilized by many transportation agencies, and selected for Alberta, is the present worth method, using costs alone.

This method discounts all future sums to the present using an appropriate discount rate. The empirical relationship to determine the present worth of some future expenditure is solved by the following equation:

$$PW = \frac{F}{(1 + I)^n}$$

where PW = present worth

F = future value

I = discount rate = 4%

n = number of years in the future that the amount is to be received.

It is essential that comparisons only be made for analysis periods of equal length. Caution must also be exercised such that the economic analyses data is coupled with sound engineering judgement in order to select and finalize a pavement design alternative.

6.3 Summary

The process of carrying out a life cycle cost analysis to allow potential design strategies to be evaluated on an economic basis is:

1. Estimate unit and per km construction costs of each alternative.
2. Estimate yearly maintenance costs if there are significant differences between alternatives.
3. Estimate the service life for each alternative.
4. Determine the unit and per km construction costs, maintenance costs and future rehabilitation costs for each alternative for a 30 year analysis period.
5. Estimate the residual value of the final rehabilitation at the end of the analysis period.
6. Using a 4% discount rate, calculate the present worth of initial construction, rehabilitation, and maintenance costs and residual value for each alternative.

7 '3R/4R' GEOMETRIC DESIGN

Projects that generally include resurfacing, restoration or rehabilitation of existing paved roads are termed 3R projects. Projects that include some reconstruction of existing paved roads, which generally takes place in conjunction with resurfacing, restoration or rehabilitation of the existing pavement are termed 4R projects.

Pavements are designed with an intended life of 20 years and therefore, the first and subsequent rehabilitations will generally occur in 15 - 25 year cycles. This pattern establishes a logical timetable for the review of geometric design standards on existing paved roads. If geometric improvements are required it is generally most cost-effective to construct these improvements at the time of rehabilitation.

3R/4R Geometric Design Guidelines have been developed by AT&U [AT&U 96]. These guidelines focus on the most safety-cost effective improvements and also encourage the use of low-cost opportunities to improve safety where major reconstruction is not cost-effective. The guidelines contained in this document are general in nature and are not a substitute for engineering judgement. The guidelines should be used when the terms of reference on rehabilitation projects requires a geometric assessment to be carried out.

8 NEW CONSTRUCTION

8.1 Introduction

The design of flexible pavements involves a study of soils and paving materials, their behaviour under load and the design of the pavement structure to carry that load under all climatic conditions [Yoder 75]. Additionally, in the cases of new construction, the thickness of the designed pavement structure is a necessary input into the geometric design of the roadway. The subgrade must be designed to a sufficient width to accommodate the projected width of the pavement structure and still achieve the design roadway width after final paving.

The methods outlined in this section address the design of pavement structures for new construction, reconstruction and widening.

The use of granular base pavements as a standard pavement structure type has been adopted by AT&U based upon long experience with their performance across the province. The availability of good quality aggregates and the benefits related to permeability and drainage, which may be more critical with the fine grained soils typically used for subgrade construction in Alberta, are supporting advantages.

Cement stabilized base courses (soil cement) in more recent years have not been widely used, either on the grounds of performance or economy, except where there are pressing problems with gravel supply and associated higher costs related to haul. For these reasons, very few cement stabilized base course pavement structures are designed.

AT&U experience with full depth asphalt concrete pavements has been mixed. Performance problems with projects constructed in the 1970's along with the increased costs of asphalt cement materials associated with this pavement structure type, resulted in the discontinuance of the use of full depth pavements in the early 1980s.

Typical pavement structures used in Alberta are presented in Figure 8.1.

Typical granular base pavement structures used in the past, as reported by AT&U for a range of design ESALs from 3×10^5 to 5×10^6 and for two typical subgrade soil types, are presented in Table 8.1. Using AASHTO procedures will provide similar pavement thicknesses for low and medium traffic levels and increased pavement structures for high traffic levels.

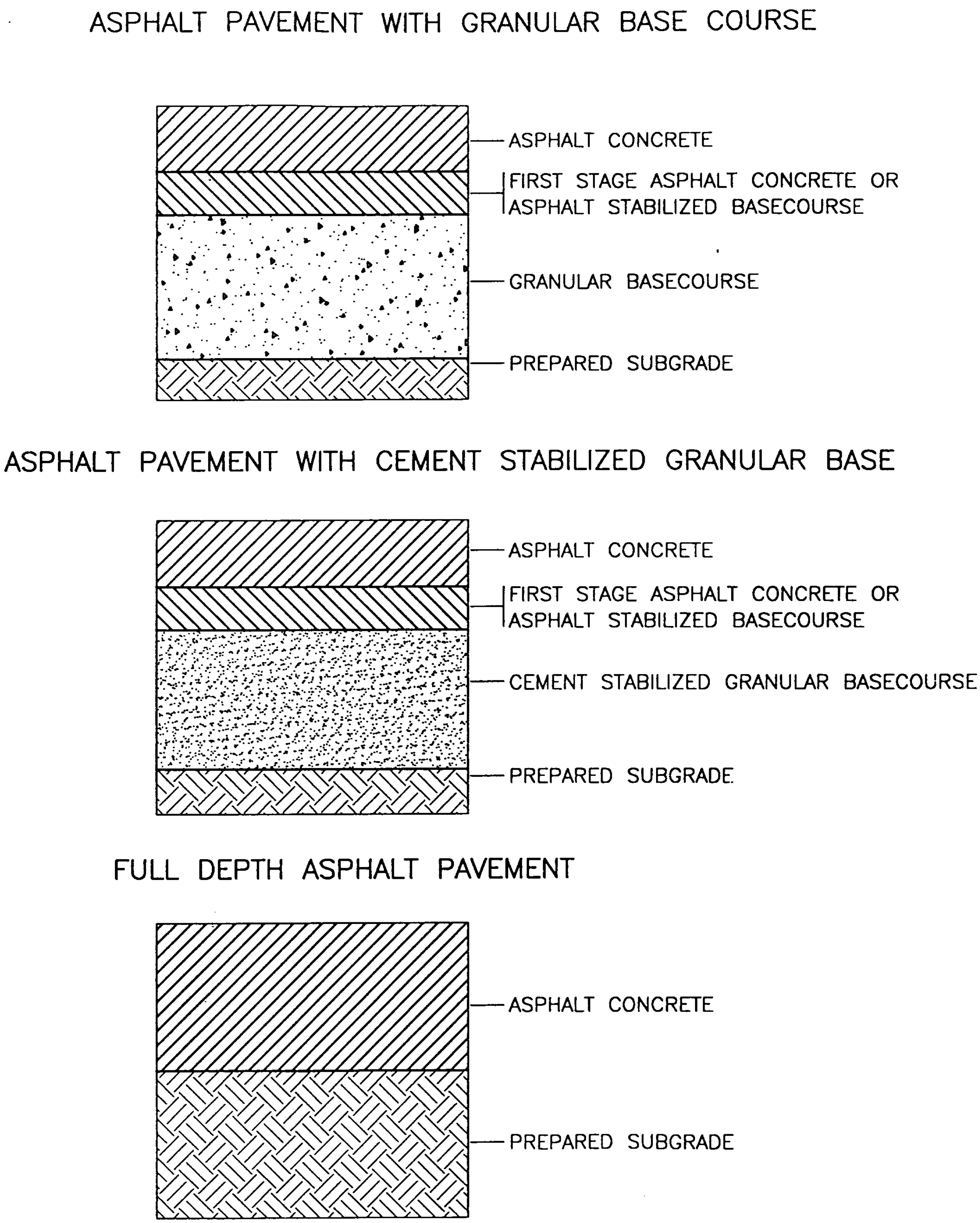

Figure 8.1 Typical Pavement Structures

The provision of relatively thick granular bases has historically provided acceptable pavement performance. The pavement structures derived from the methods outlined in this Manual should follow a similar design philosophy.

TABLE 8.1 TYPICAL AT&U PAVEMENT STRUCTURES FOR SELECTED SUBGRADE AND TRAFFIC LOADING

Subgrade Type	Conventional Pavement Structure Course	Design ESALs				
		0.3×10^6	0.5×10^6	1×10^6	3×10^6	5×10^6
APoor@ (CBR . 3) Unified Soil Class. - CH-CI	ACP (Final Paving) ACP (1st Stage Paving) GBC	80[1] 60 250	80 60 280	90 60 300	110 60 330	110 60 350
AGood@ (CBR . 4) Unified Soil Class - CL, SC	ACP (Final Paving) ACP (1st Stage Paving) GBC	80 60 200	80 60 230	80 60 250	100 60 280	100 60 300

[1] all thicknesses in mm.

8.2 Design Strategies

Staged design and construction of new pavement structures has been adopted as a design policy by AT&U. The primary advantage related to staged design and construction is reduced life cycle costs and more effective use of funds as compared to unstaged construction .

Staged design and construction methodology as defined by AT&U can be outlined as follows:

- The full pavement structure is designed for a 20 year design period.
- The full base structure and a component of the asphalt concrete layer is constructed under one contract as the first stage.
- After 1 or more years of service, the final asphalt concrete pavement stage is designed and constructed.

For high traffic primary highways, the final asphalt concrete stage is normally programmed to follow the first stage pavement by one or two years. For lower traffic primary and secondary highways, the programming of the final asphalt concrete stage is based upon pavement evaluation testing and monitoring of the first stage pavement and can be delayed by 3 to 5 years.

The average pavement service lives for staged and unstaged pavement structures, as measured from the time of the original first stage base and paving to the first overlay, have been determined by AT&U to be:

Pavement Base Type	Average Service Life to First Overlay (yrs.)	
	Staged	Unstaged
Granular Base	22	18
Cement Stabilized Base	18	14

These service lives are average values for the entire primary highway network. The actual service lives for highly trafficked highways may be less. However the difference of about 4 years between staged and unstaged pavement structures would still be expected.

The staged design and construction approach allows for the stabilization of the subgrade and base course and initial differential consolidation and settlement of the subgrade to occur prior to the application of the final stage asphalt concrete. The staging approach also allows for fine tuning of the thickness of the final stage pavement based upon the analysis of FWD test results to account for varying subgrade and pavement conditions throughout the length of the project.

Unstaged pavement design and construction would only be considered on widening projects, selective reconstruction, where remobilization construction costs may be high or in other special cases.

However, in order to achieve the lower life cycle costs related to staged design and construction, it is critical that funding for the final stage asphalt concrete be programmed and that timely performance testing and monitoring programs of the first stage pavement structures are in place.

8.3 Economic Analysis

Except for the special cases as noted, the staged design and construction approach is to be adopted on all new construction projects. In general, life cycle cost analysis to justify staged design and construction will not be required.

8.3.1 Cement Stabilized Base Course

The design of new construction projects located in areas short of high quality gravel which would result in very long gravel hauls may require that pavement structure alternatives other than conventional granular base be considered. In these circumstances, it will be necessary to carry out an economic analyses, as outlined in Section 6, to assess the relative life cycle costs of both granular base course and cement stabilized base course design alternatives.

The recommended estimated service lives to be used in this analysis are presented in Table 8.2. These estimated service lives are based upon average service life values over all ranges of traffic for the whole province as reported by AT&U. These service lives may be based upon limited performance data especially for very high traffic levels and may need to be modified based upon local experience and engineering judgement.

TABLE 8.2 RECOMMENDED ESTIMATED SERVICE LIVES FOR GBC AND CSBC PAVEMENT STRUCTURES

PAVEMENT BASE TYPE	YEAR OF CONSTRUCTION			
	FIRST STAGE	FINAL STAGE	FIRST OVERLAY	SECOND OVERLAY
GBC	0	2 - 4	21 - 23	35 -37
CSBC	0	2 – 3	17 - 19	31 -33

The unit costs used in the economic analysis would be based on Unit Price Reports prepared by AT&U adjusted to reflect actual haul costs of both gravel and sand materials and estimated design Portland cement requirements for the geographic area the project is located in. Maintenance costs over the design period for both the alternatives would be estimated based upon experience and other inputs.

8.3.2 Granular Subbase

The use of granular subbase courses (GSBC) as part of a new construction structural design is a viable alternative. However their inclusion must be supported by an economic analysis to demonstrate potential cost savings. There may be other situations where the inclusion of granular subbase courses can provide cost effective engineering solutions with respect to frost protection, drainage, aggregates management, etc..

8.4 Design Methodology for New Construction

8.4.1 Background to AASHTO Flexible Pavement Design

The procedures developed in the Guide [AASHTO 86, 93] for new construction or reconstruction are basically an extension of the algorithms originally developed from the AASHO Road Test and provide the designer with the opportunity to use state-of-the-art design techniques. Major modifications to previous practices are described in the 1993 Guide include:

1) The introduction of the resilient modulus to provide a rational testing procedure that can be used by an agency to define material properties.

2) The layer coefficients for the various materials are defined in terms of resilient modulus as well as standard methods such as CBR.

3) The environmental factors of moisture and temperature are objectively included to replace the subjective regional factor term previously used.

4) Reliability is introduced to permit the designer to use the concept of risk analysis for various classes of roadways.

Material properties for structural design are based on characterization of an elastic or resilient modulus. For roadbed materials, AASHTO recommends that laboratory resilient modulus tests (now AASHTO T294-92) be performed on representative samples in stress and moisture conditions simulating those of the primary moisture seasons. Procedures are described in the Guide to allow the determination of an effective roadbed soil resilient modulus which is equivalent to the combined effect of all the seasonal modulus values. Alternatively, the resilient modulus values may be determined by correlations with other measured soil strength properties. The purpose of identifying seasonal moduli is to quantify the relative damage a pavement is subjected to during each season of the year and treat it as part of the overall design.

Pavement layer coefficients may be based on those traditionally developed and used in the original AASHTO procedure or, more preferably, derived from test roads or satellite sections. Charts are also available in the Guide for estimating structural layer coefficients from various base strength parameters as well as resilient modulus test values.

The effectiveness of the ability of various drainage methods to remove moisture from the pavement is not described with detailed criteria, but is recognized through the use of modified layer coefficients. The factor for modifying the layer coefficient is referred to as an m_i value and has been integrated into the structural number (SN) equation along with the layer coefficient (a_i) and thickness (D_i); thus:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

Recommended m_i values, as a function of the quality of drainage and percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation, are also tabulated in the Guide.

Determination of the required structural number (SN) involves the use of a nomograph which solves a specific design equation and parameters required for specific conditions, including:

1) the estimated future traffic for the performance period
2) the reliability, R, which assumes all inputs reflect an average value,
3) the overall standard deviation, S_o
4) the effective resilient modulus of roadbed material, M_R
5) the design serviceability loss, Î PSI = p_o - p_t

The DARWin 3.0 computer program can be used to analyze the FWD deflection data from representative prototype pavements and determine the backcalculated subgrade modulus. This program can also be used to determine the required structural number (SN) and provide pavement design alternatives for the design inputs.

An example given in Appendix H of the Guide [AASHTO 93] may be referred to in order to familiarize the designer with the AASHTO flexible pavement design procedure. For purposes of this Manual, design examples applicable to typical projects in Alberta are presented in subsequent sections.

Since the AASHTO Guide utilizes Imperial units, conversions to the metric system are required to be made manually. Metric Conversion Factors are given in Appendix B of this Manual. The metric version of the Guide is expected to be published in 1997. The computerized design system DARWin 3.0 performs on-the-fly conversions between the Imperial and metric systems.

8.4.2 Agency Practices

A Synthesis of agency pavement structural design practice was conducted in 1990 and 1991 and reported in NCHRP Synthesis 189 [NCHRP 93]. For flexible pavement, the information revealed that 51 agencies employed empirical procedures, predominately those in the AASHTO Guide for Design of Pavements [AASHTO 86]. The AASHTO 1972 procedures were reportedly used by some 22 agencies. AASHTO 1986 procedures were mentioned as used in full or partially by 20 agencies, with 10 others anticipating changing to this procedure in the future.

There has been concern expressed that despite the major efforts towards advancing pavement technology, much of this research has not found its way into the daily design procedures used by practicing engineers. It has been reported that only about half of the State Highway Agencies have formally incorporated or use the 1986/93 AASHTO Guide for flexible pavement design [ERES 95]. Approximately 28 percent still use the 1972 AASHTO Interim Guide, although more design capabilities are provided in the newer Guides. In view of this seemingly low percentage of implementation, a number of reasons were presented. The increased size, compared to the relatively small and simple 1972 Interim Guide, may have led to a perceived increase in complexity for the new Guide. Another cause may be in the difficulty in obtaining and demonstrating the applicability of some of the new inputs, such as the resilient modulus of the subgrade soil, reliability and the drainage coefficients. Despite these and other difficulties, improvements in pavement design practice can be achieved with recently developed efforts.

As major research activity to enhance implementation and use of the 1993 AASHTO Design Guide has recently been performed in cooperation with the U.S. Department of Transportation, Federal Highway Administration, by Brent Rauhut Engineering Inc. [BRE 97]. This analysis was conducted on relating pavement performance to specific pavement layers utilizing data contained in the Long Term Pavement Program National Information Management System (LTPP). Specifically, the focus of this research activity was to identify differences that exist between laboratory measured and backcalculated resilient moduli; determine the applicability of the C-Values, drainage coefficients, and relative damage factors that are included in the Design Guide; and to provide procedures to adequately consider the seasonal variation of material properties as related to flexible pavement designs. Based on these results, design pamphlets have been prepared in support of the AASHTO Design Guide. Detailed review of these reports may be considered of interest to the users of this Manual.

8.4.3 Design Inputs

The design inputs required by the AASHTO method and the DARWin 3.0 program and the recommended values for the design of new construction are as follows:

Design Traffic

The Design ESALs/lane for the design period is determined as outlined in Section 5.

Serviceability Loss

In Alberta the Ride Comfort Index (RCI) measures the riding quality provided by the pavement. The RCI scale ranges from 10 (excellent) to 0 (very poor). From the experience in Alberta, the minimum desirable RCI levels for highways is 5.5.

In the AASHTO system the roughness scale for ride quality ranges from 5 to 0. For design it is necessary to select both an initial and terminal serviceability index. An initial serviceability index of 4.2 is suggested to reflect a newly constructed pavement. A terminal serviceability index of 2.5 is suggested to be used in the design of major highways. The design serviceability loss, Î PSI, is the difference between the newly constructed pavement serviceability and that tolerated before rehabilitation.

For the Design Charts developed for this Manual, the recommended Î PSI is 1.7, based on $p_o = 4.2$ (initial serviceability) and $p_t = 2.5$ (terminal serviceability). This corresponds to a relative difference in RCI values of 3.4. These values for initial and final serviceability have also been adopted within this Manual for final stage pavement and rehabilitation design.

Reliability

Reliability concepts are given in detail in the AASHTO Guide, and are intended to account largely for chance variations in traffic prediction and performance prediction, and therefore provides a predetermined level of assurance (R) that pavement sections will survive the period for which they were designed. Suggested levels of Reliability for various Functional Classifications are given in Table 2.2 of the AASHTO Guide and range from 50 to 99.9% for local roads to Interstate and other Freeways, respectively.

It is recommended that Reliability Levels shown in Table 8.3 be used for design of new construction.

TABLE 8.3 RECOMMENDED LEVELS OF RELIABILITY FOR NEW CONSTRUCTION

Design ESALS ($x10^6$)	Reliability %
<0.1	75
0.1 to 5.0	85
5.0 to 10.0	90
>10.0	95

Overall Standard Deviation

The value for Overall Standard Deviation as developed at the AASHO Road Test for flexible pavements was 0.45. This value is recommended for use in this Manual.

Resilient Modulus Correction Factor

Resilient modulus values are used to characterize subgrade strength values for use in design. Where the subgrade modulus is backcalculated from the results of FWD deflection test data, the recommended combined factor, to correct the backcalculated subgrade modulus to make it consistent with the value used to represent the AASHO Road Test subgrade and to adjust it for seasonal variations in subgrade strength is 0.36. Further background to determination of subgrade moduli values for design purposes was presented in Section 2.2.3.

Design Charts

In order to simplify the application of the AASHTO design method in this Manual, a series of design charts have been developed for a suitable range of traffic and effective roadbed resilient modulus values that would be encountered in Alberta. These design charts are intended to be used for initial preliminary designs, rather than the nomograph shown as Figure 3.1 in the AASHTO Guide. They are based on parametric solutions provided initially by the DARWin 2.0 (non-metric) design program. The SN values have been converted to metric units and have been verified with DARWin 3.0.

These charts provide a range of Structural Numbers (SN) from 25 to 200 mm, for Design ESAL values from 2×10^4 to 3×10^7. Ranges of Reliability levels are plotted for various traffic values. An overall standard deviation value S_o of 0.45 was used.

Seven individual charts (Figures 8.2 - 8.8) are provided for Effective Roadbed Resilient Modulus values of 20, 25, 30, 35, 40, 50, and 70 MPa, corresponding to approximate CBR values of 2 through 7 respectively. If modulus values other than those provided in the charts are considered necessary, interpolation of the SN required from adjacent chart values may be used.

Alternatively, DARWin 3.0 can be used directly to provide the design outputs.

<u>Structural Layer Coefficients</u>

In the AASHTO system for structural design, layer coefficients (a_i values) are assigned to each layer material in the pavement structure in order to convert actual layer thicknesses into a structural number (SN). This layer coefficient expresses the empirical relationship between SN and thickness and is a measure of the relative ability of the material to function as a structural component of the pavement.

In lieu of actual laboratory resilient modulus values for each material, it is recommended that the following coefficients be used for local materials:

TABLE 8.4 RECOMMENDED LAYER COEFFICIENTS

Material	Layer Coefficient (a_i)
Asphalt Concrete (ACP)	0.40
Asphalt Stabilized Base Course (ASBC)	0.23
Cement Stabilized Base Course (CSBC)	0.23
Granular Base Course (GBC)	0.14
Granular Subbase Course (GSBC)	0.10

Guidelines for selection of other values for layer coefficients are given in the AASHTO Guide and earlier documentation. [NCHRP 72].

Drainage Coefficients

Section 2.4 of the AASHTO Guide provides general definitions corresponding to different drainage levels in the pavement structure. Recommended m-values for untreated base and subbase materials are given in Table 2.4 of the Guide. Local experience would determine the most appropriate drainage coefficient to be used, however it is expected that the range of 1.20 to 0.80 would be suitable for Ahigh and dry@ subgrades to Alow and wet@ subgrades, respectively. Normally an m-value of 1.00 would be expected.

Other Considerations

It should be recognized that changes in surfacing structural design cross-sections may have implications on the geometric sections for grade design. Drainage provided for the pavement structure may also have profound effects on the drainage coefficients used for design.

Improved, or other changes in material properties and availability may also have significant changes on the layer coefficients used for design.

It was recognized that the design input parameters and values will need to be validated and updated based upon pavement performance and experience as designers become more familiar and experienced with the new design methodology.

8.4.4 Staged Design

Minimum ACP thicknesses for the first stage pavement structure are required to ensure that the structural and fatigue capacity of the relatively thin initial ACP layer is not exceeded. Recommended first stage minimum ACP thicknesses as a function of Design ESALs are given in Table 8.5. These minimum thicknesses have been established to allow for minimum two lift final stage pavement design. A minimum two lift final stage pavement design is a requirement to ensure that grade, cross-section and smoothness can be restored.

TABLE 8.5 RECOMMENDED MINIMUM THICKNESS OF FIRST STAGE ACP

Design ESALs ($x10^6$)	Minimum Thickness of 1st Stage ACP
< 2.0	60 mm
2.0 to 5.0	80 mm
5.0 to 10.0	100 mm
> 10.0	120 mm

8.4.5 Unstaged Design

The structural design of unstaged pavements follows the same methodology as for staged pavements except that the full asphalt concrete pavement structure is designed and constructed at one time.

For unstaged design and construction of widening projects, it is necessary to consider the following factors when designing the new full unstaged pavement section:

$ the base type of the existing pavement structure
$ pavement layer thicknesses of the existing pavement structure
$ the location of the transition between the existing pavement structure and the new widened structure with respect to wheelpath locations and travel lanes.

It is critical to ensure the continuity of lateral drainage of the subbase and base course layers through the existing and adjoining new pavement structure.

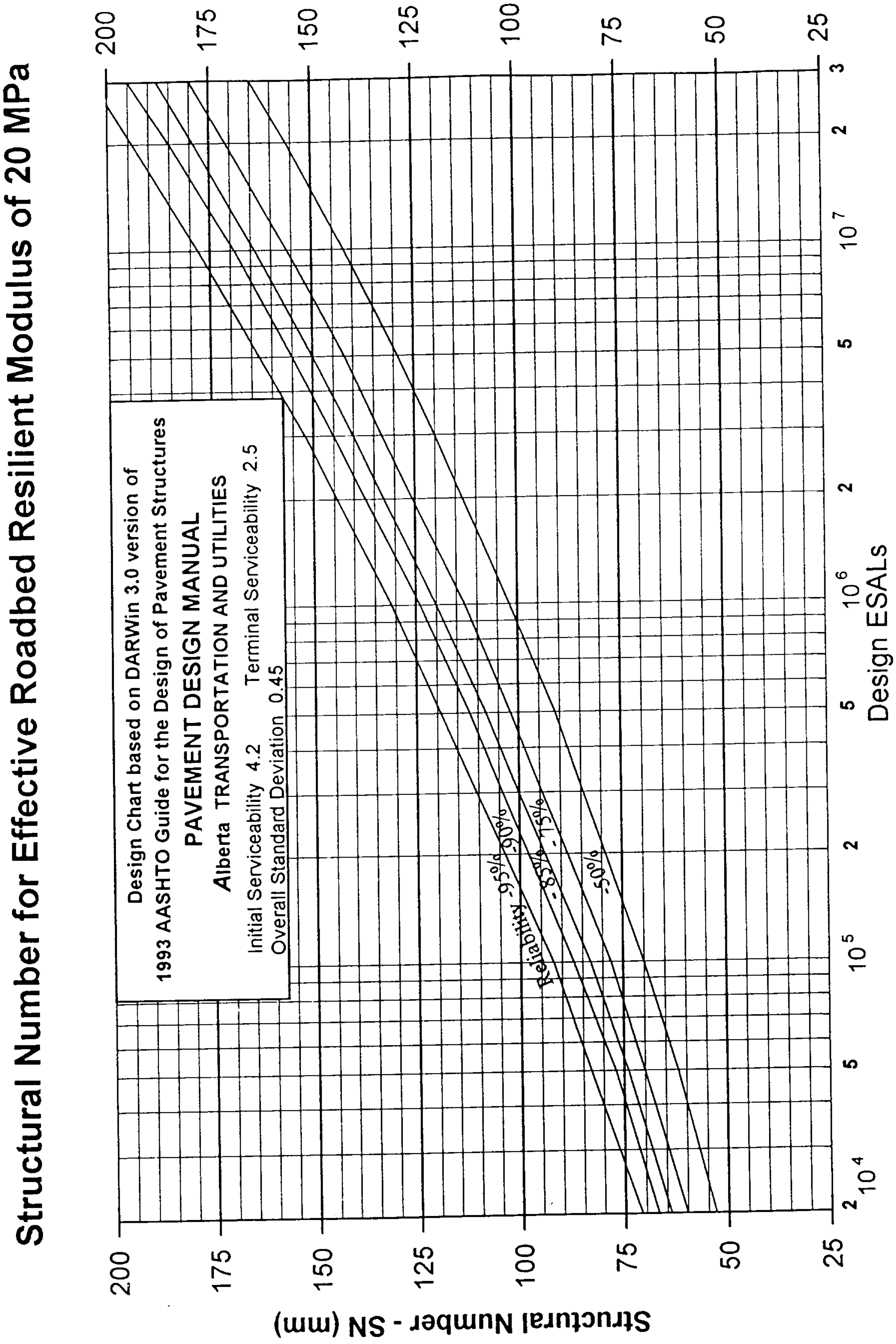

Figure 8.2 Design Chart for M_R = 20 MPa

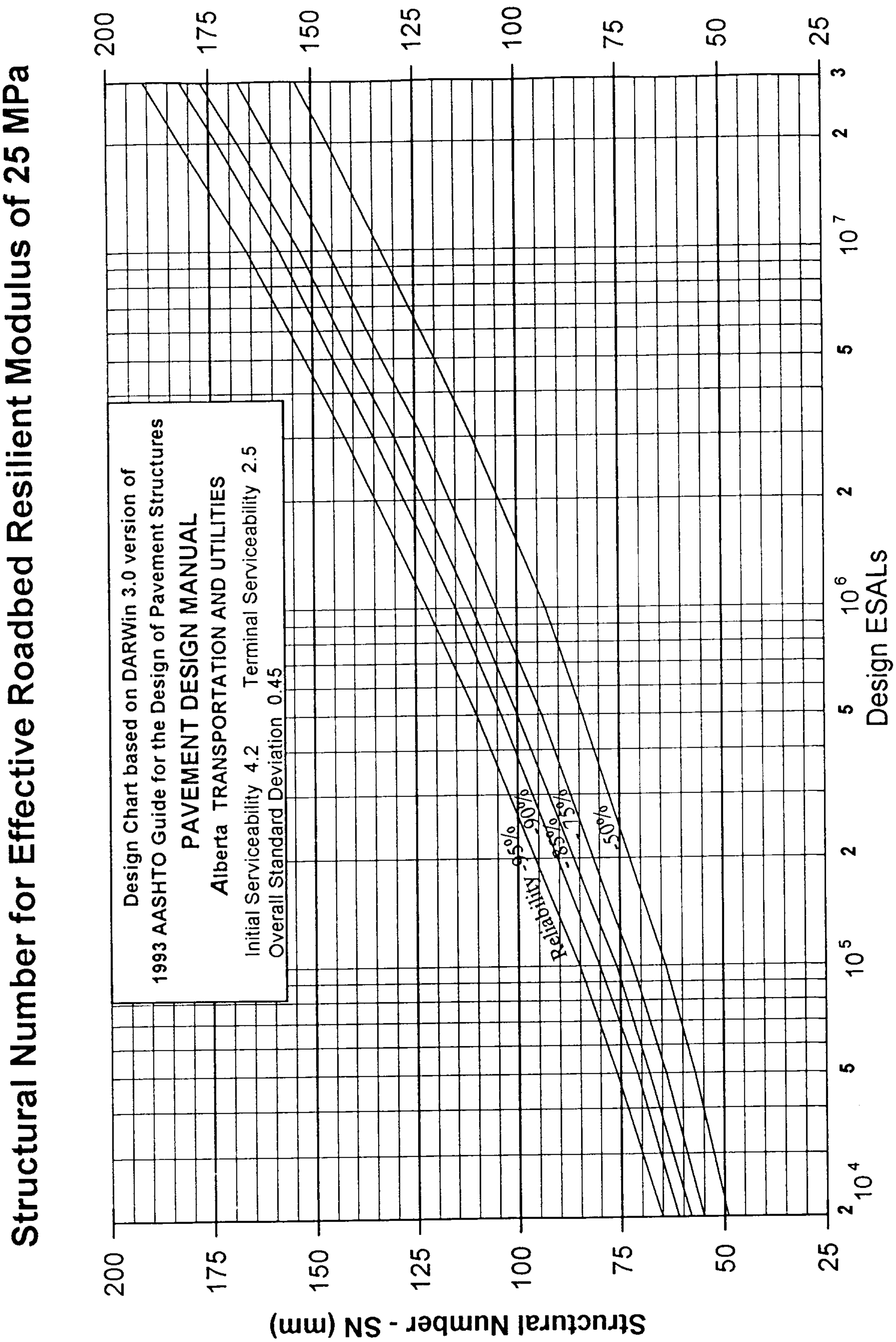

Figure 8.3 Design Chart for M_R = 25 MPa

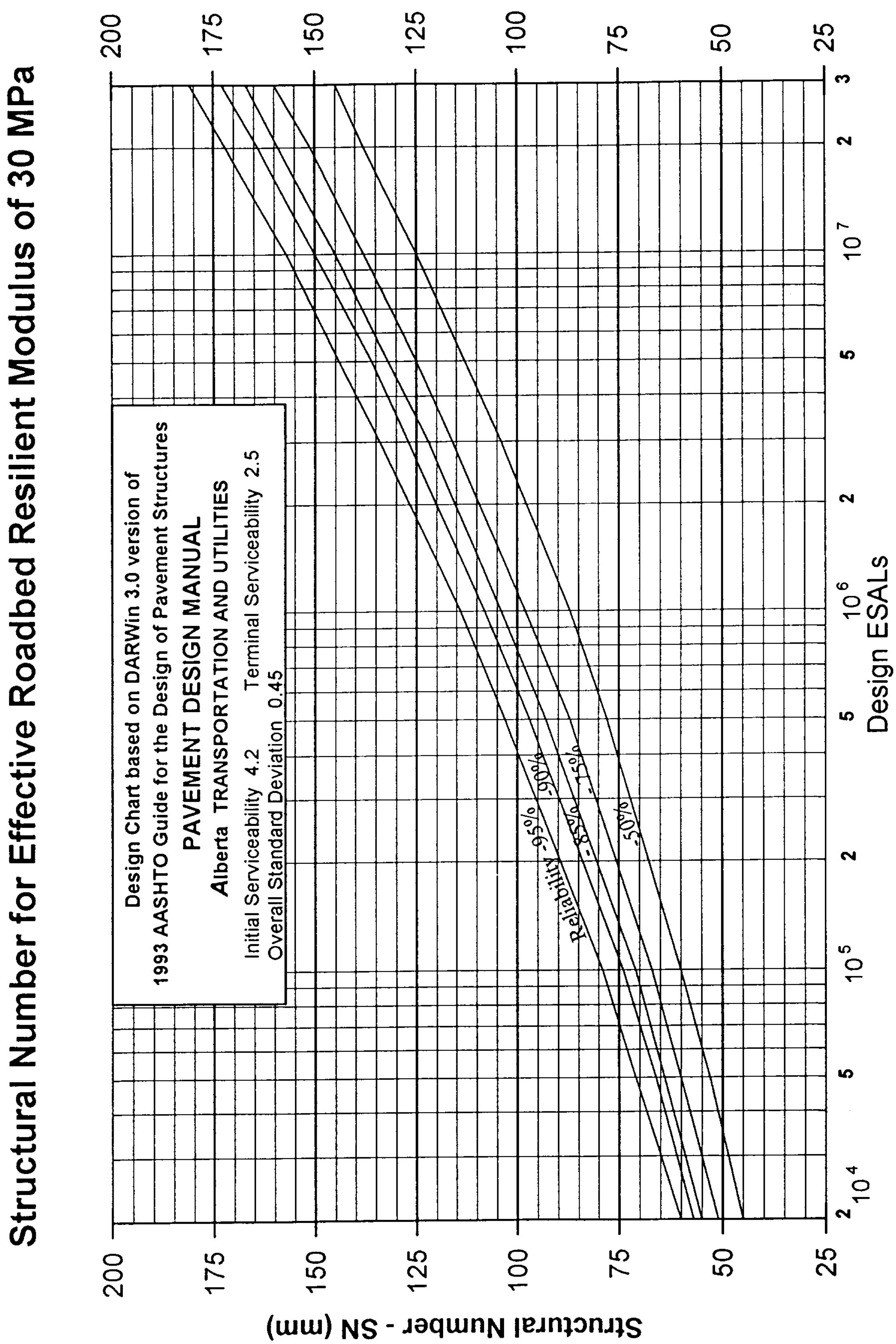

Figure 8.4 Design Chart for M_R = 30 MPa

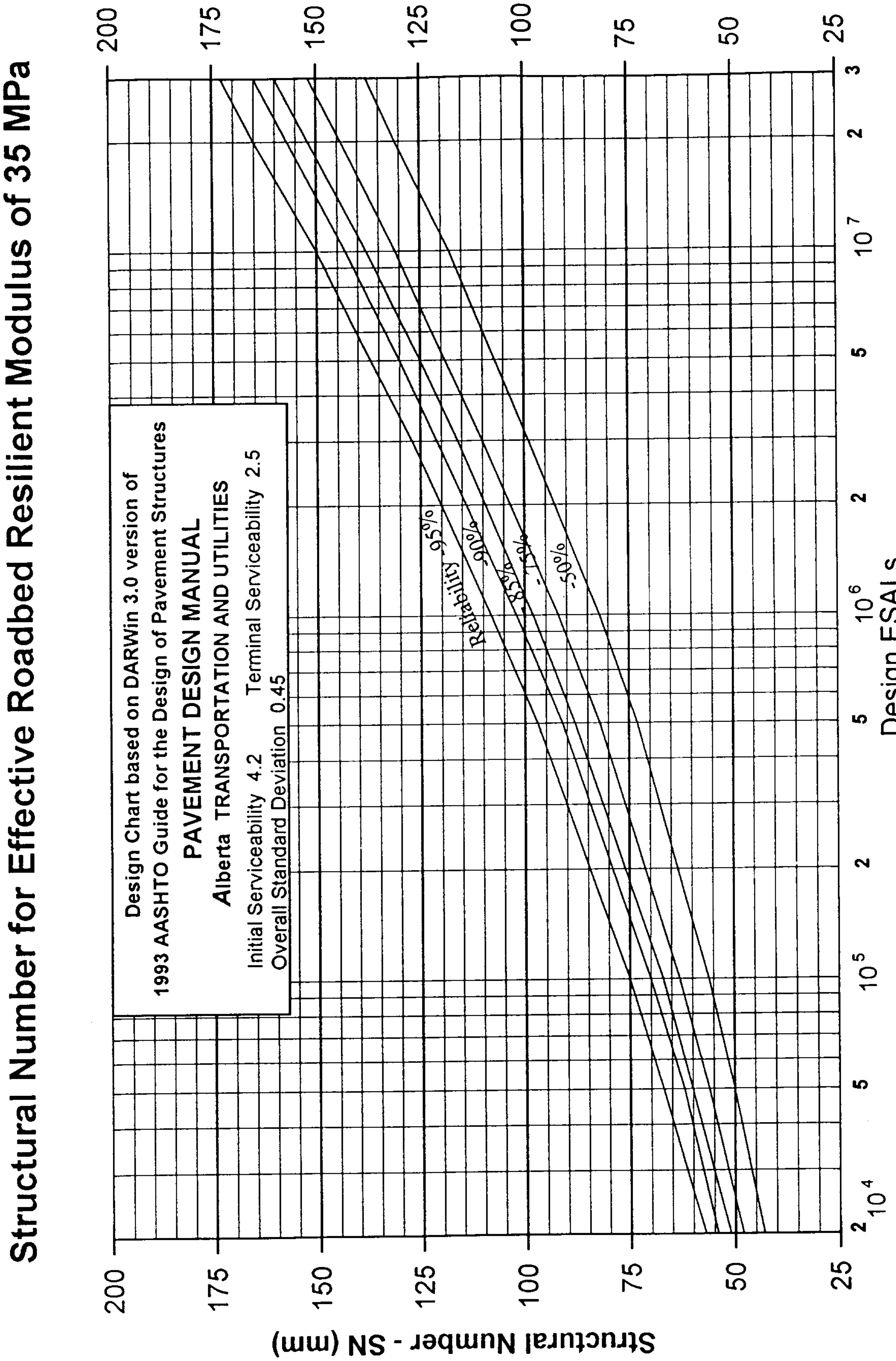

Figure 8.5 Design Chart for M_R = 35 MPa

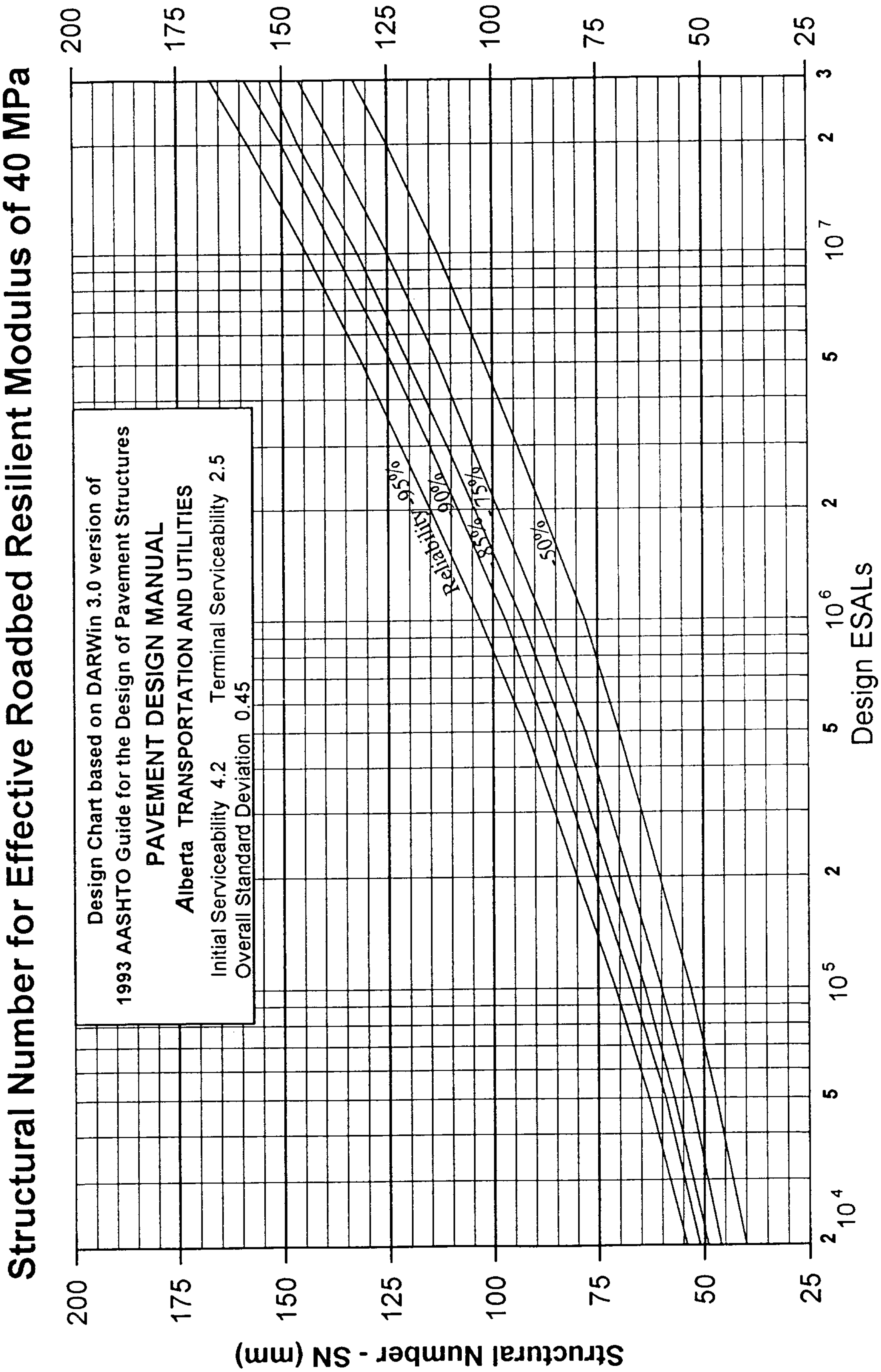

Figure 8.6 Design Chart for M_R = 40 MPa

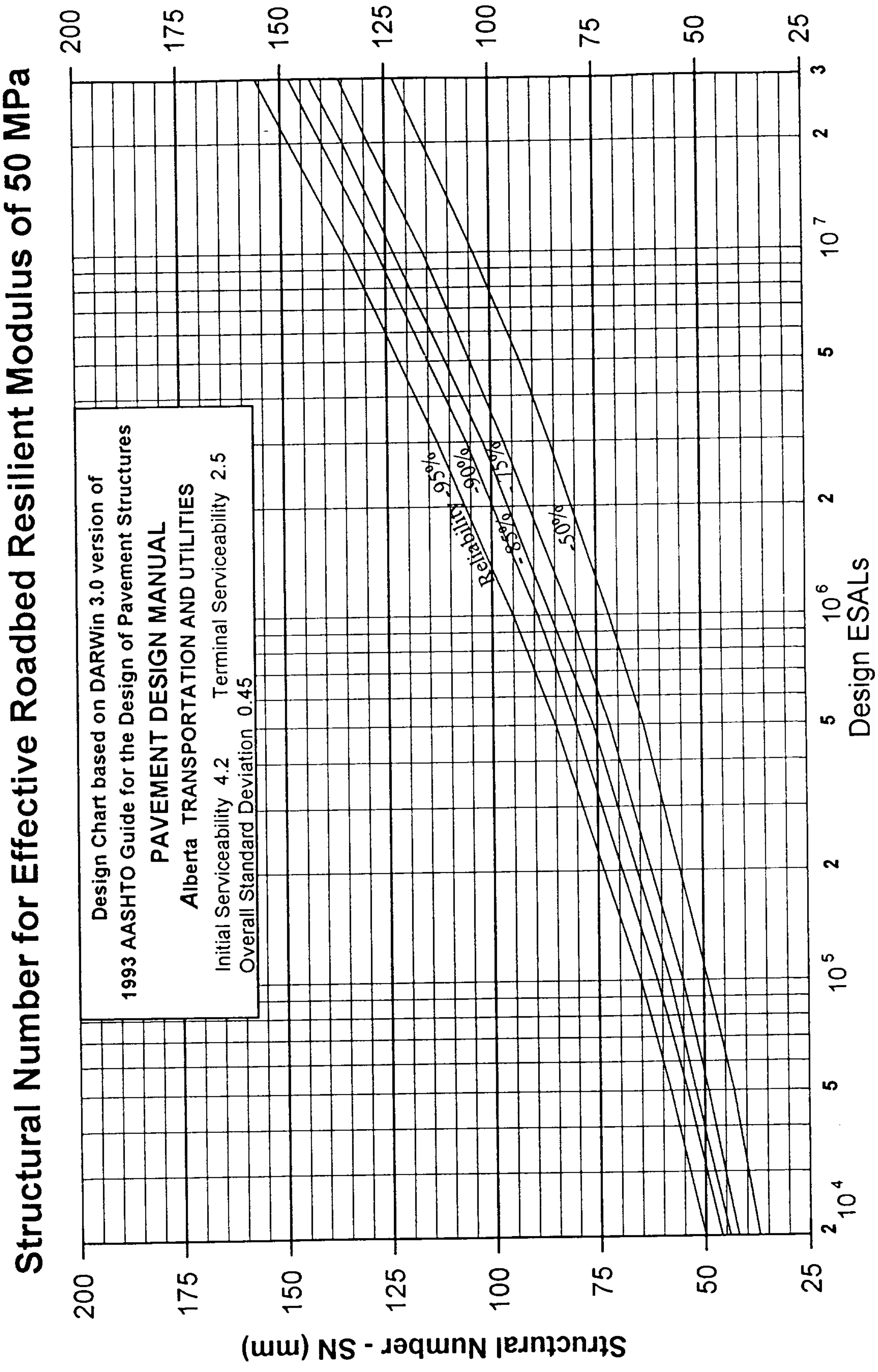

Figure 8.7 Design Chart for M_R = 50 MPa

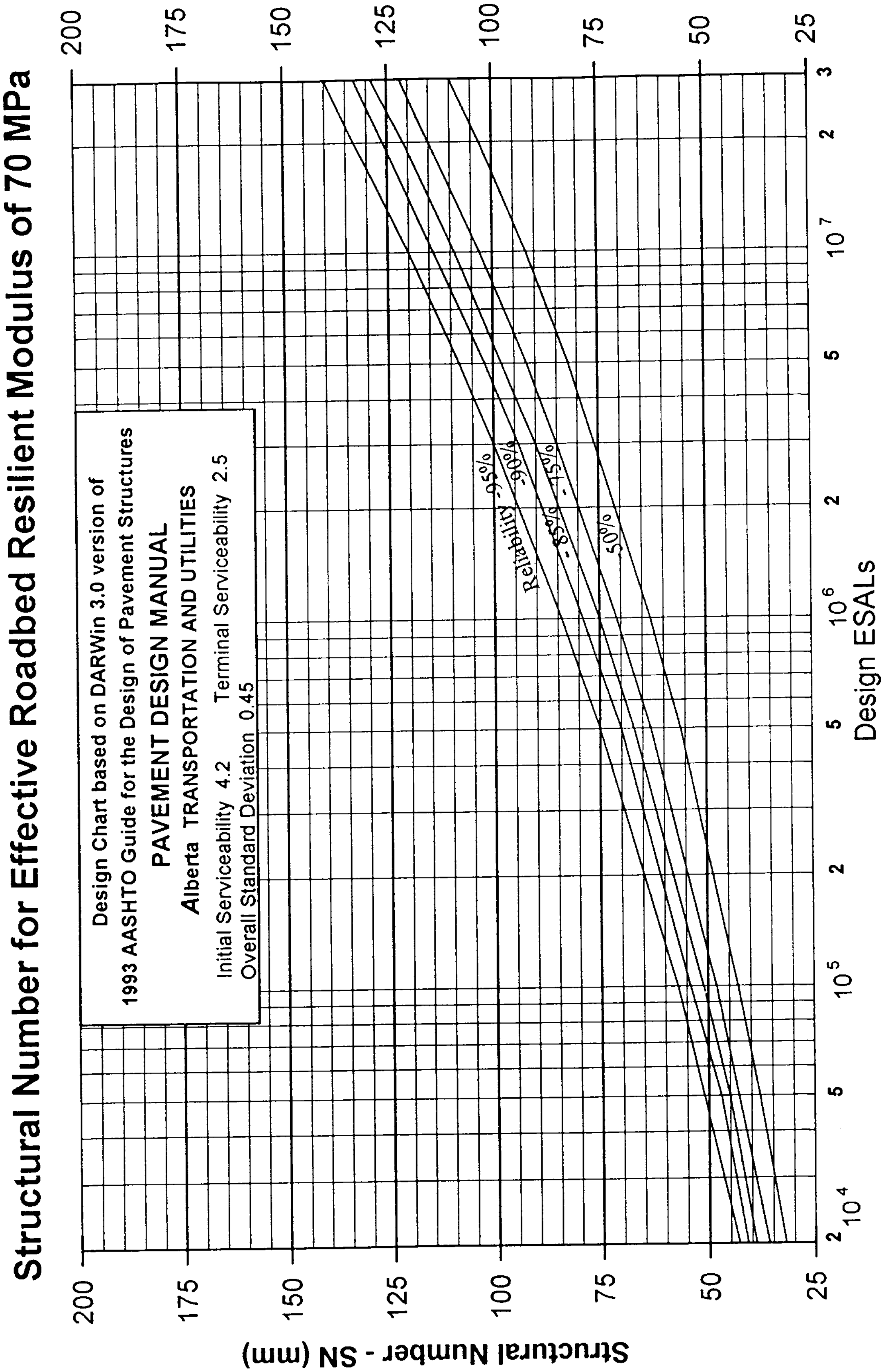

Figure 8.8 Design Chart for M_R = 70 MPa

8.5 Design Examples - New Construction

8.5.1 Project 1

Project 1 is the design of new westbound lanes of a four lane divided highway located west of Edmonton.

Establish Design Inputs

Design ESALs

ESALs/day/direction = 888 (1995 Traffic Volumes Report)
Lane distribution = 85% outside lane: 15% inside lane
Seasonal distribution = uniform
Design period = 20 years
Traffic growth factor = 3% (for twinning projects)
Design ESALs = 888 x .85 x 365 x 26.87
= 7.4×10^6 (in design lane)

Serviceability

Initial Serviceability = 4.2
Terminal Serviceability = 2.5

Reliability

R = 90% (for 7.4×10^6 Design ESALs) in accordance with Table 8.3.

Overall Standard Deviation

Overall Standard Deviation $S_O = 0.45$

Environmental Impacts

Specific attention to potential heaving due to frost and the possibility of swelling soils should be considered, however loss of serviceability due to these conditions is very difficult to quantify. The climate in Alberta corresponds to Region VI in the United States, i.e., dry, hard freeze, spring thaw (Fig.4.1 AASHTO Guide 93). The subgrade soil is classified as a CI-CL in the Unified System, having a Plasticity Index of approximately 12%. As such, the soil is not considered to be a highly swelling soil. It does have potential for frost heaving in the presence of moisture, especially in the cut sections. The fill sections vary from 2 to 3 m in height from surrounding terrain.

Recognition of these environmental impacts is not directly designed for, however consideration is given to these conditions in the selecting of effective roadbed resilient modulus values and drainage coefficients.

Effective Roadbed Resilient Modulus

A nearby section of the existing east bound where soil types and drainage conditions were considered to be representative of the new west bound lanes was selected as a prototype. The FWD deflection testing of the prototype was carried out in June. The raw data was converted into a format recognizable by DARWin 3.0. The average backcalculated subgrade modulus for the prototype section was 82 MPa when processed with the DARWin 3.0 program. Using the recommended combined factor of 0.36 to correct the backcalculated subgrade modulus and to adjust it for seasonal variations in subgrade strength resulted in an Effective Roadbed Resilient Modulus of 30 MPa.

Structural Layer Coefficients

The structural layer coefficients were selected in accordance with Table 8.4.

Drainage Coefficients

A drainage coefficient of 1.0 was selected for both the base and subbase courses.

Design of Pavement Structure

Selection Structural Number

Since the Effective Roadbed M_R value was 30 MPa, the chart for 30 MPa was selected for design purposes. Using this chart and the Design ESALs of 7.4×10^6, a Structural Number SN of 144 mm was determined for a Reliability of 90%. This is presented in Figure 8.9.

Selection of Layer Thickness

The following basic equation provides the basis for converting SN into actual thicknesses of surfacing, base and subbase:

$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$

where

a_1, a_2, and a_3 = layer coefficients representative of various courses,

D_1, D_2 and D_3 = actual thicknesses in mm of various courses, and

m_2, m_3 = drainage coefficients for base and subbase courses.

Trial No. 1- Surface and Granular Base Course:

An initial trial thickness of 180 mm of ACP was selected, in consideration of local experience and traffic.

SN_1 = (180 mm x 0.40) = 72 mm

The resulting thickness of granular base course is determined:

Required SN_2 = (144-72) = 72

Calculated D_2 = (72/0.14) = 514 mm; rounded off to 520 mm.

Calculated SN_2 = (520 x 0.14 x1.0) = 73

Total SN provided = (72 +73) = 145

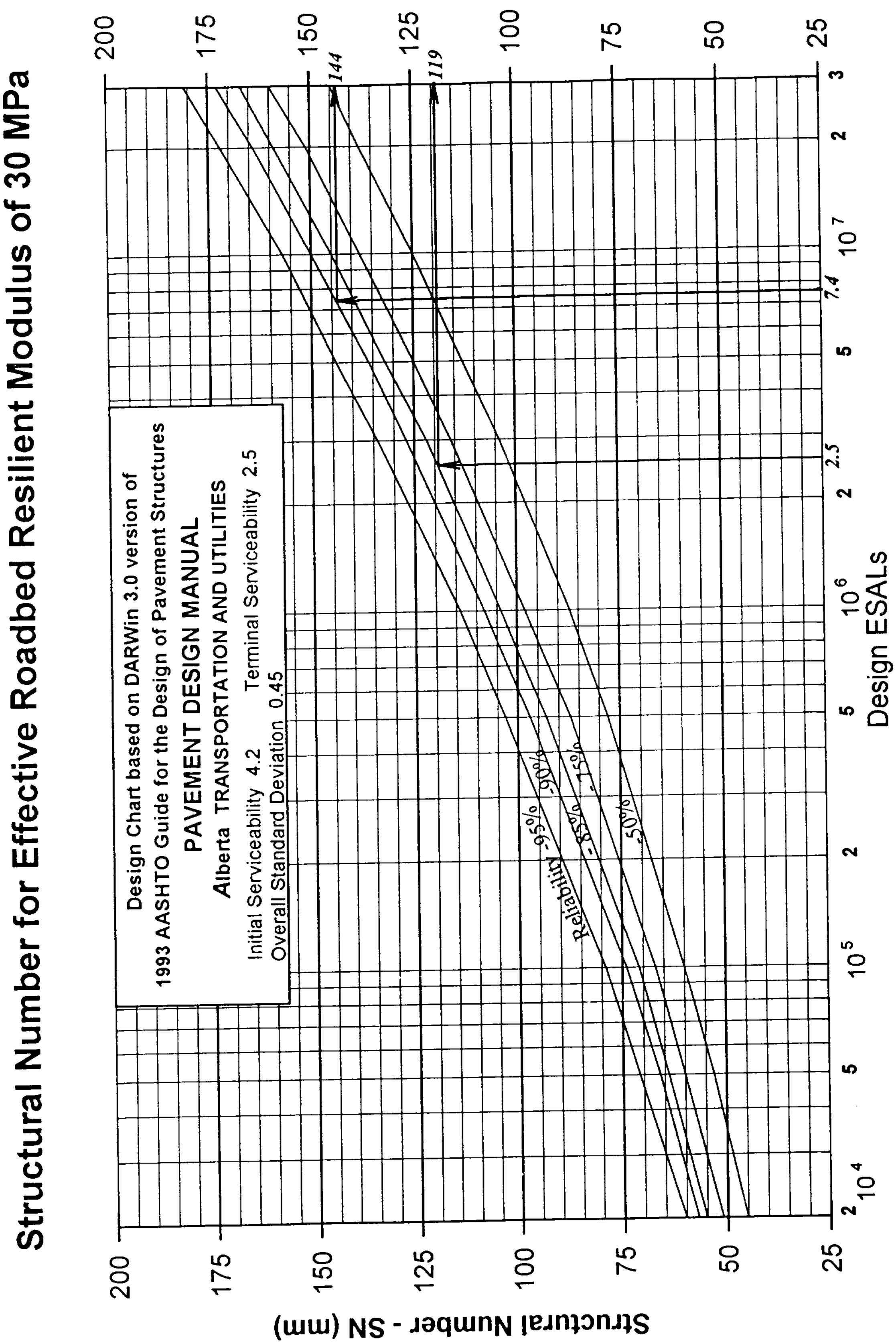

Figure 8.9 Design Examples 1 and 2

Trial No. 2 - Subbase Course in addition to Surface and Granular Base Course

The initial thickness of 180 mm of ACP and 350 mm of granular base course is selected.

SN_1 = (180 mm x 0.40) = 72 mm
Calculated SN_2 = (350 x 0.14 x1.0) = 49
Subtotal SN = (72 + 49) =121

Required SN_3 = (144 - 121) = 23

The resulting thickness of subbase is determined:

D_3 = (23/0.10 x 1.0) = 230 mm of subbase

Total SN provided = (72 + 49 +23) = 145

These, and other trial sections shown in Table 8.6 were calculated with the above general equations to satisfy a SN of 144 mm, using various combinations of structural layer type, thickness and layer coefficients.

TABLE 8.6 TRIAL SECTIONS FOR SN= 144 mm

Trial No.	Structural Layer	ai	Di (mm)	mi	SNi	sum SNi	SN Total
1	ACP	0.40	180	1.0	72	72	
	GBC	0.14	520	1.0	73	145	145
2	ACP	0.40	180	1.0	72	72	
	GBC	0.14	350	1.0	49	121	
	GSBC	0.10	230	1.0	23	144	144
3	ACP	0.40	220	1.0	88	88	
	GBC	0.14	400	1.0	56	144	144
4	ACP	0.40	250	1.0	100	100	
	CSBC	0.23	195	1.0	45	145	144

Four acceptable pavement sections that satisfy the design criteria are:

Structural Layer	Trial 1	Trial 2	Trial 3	Trial 4
ACP	180 mm	180 mm	220 mm	250 mm
GBC	520 mm	350 mm	400 mm	-
GSBC	-	230 mm	-	-
CSBC	-	-	-	195 mm

Other combinations of thicknesses could be used to satisfy the SN of 144, depending upon availability and cost of the individual components in place.

Based upon knowledge that large deposits of high quality aggregates are located near the project, the recommended pavement structure is:

120 mm ACP (thickness to be confirmed at the time of the final stage design)
100 mm ACP first stage
400 mm GBC

The computerized design system DARWin 3.0 was also used to carry out the pavement design. The output of the Flexible Structural Design Module run in metric mode for the recommended pavement structure is presented in Table 8.7.

Separate analysis and structural design should be carried out for the inside lane. Because design traffic ESALs are significantly less, a more economic final design utilizing a thinner pavement structure in the inside lane should be considered.

Material Types

Granular Base Course

Based on the knowledge of the aggregate in the pit, Designation 2 Class 25 is selected for the project since the Designation 2 Class 40 does not meet the crush count criteria.

Asphalt Concrete

The project is located in Zone D. Based upon the Design ESALs of 7.4×10^6, Asphalt Mix Type 1 is selected from Table 2.2.

TABLE 8.7 DARWin 3.0 OUTPUT FOR RECOMMENDED PAVEMENT STRUCTURE

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

AGRA Earth & Environmental
4810 -93 St
Edmonton, AB
Canada

Flexible Structural Design Module

Project 1

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	7,400,000
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	90 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	30,000 kPa
Stage Construction	1
Calculated Design Structural Number	144 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	acp	0.4	1	220	-	88
2	gbc	0.14	1	400	-	56
Total	-	-	-	620	-	144

8.5.2 Project 2

Project 2 is the design of a new two lane highway located north-west of Edmonton. All design inputs used in this example are the same as for Project 1 except for the design traffic and reliability.

Establish Design Inputs

Design ESALs

ESALs/day/direction = 207 (1995 Traffic Volumes Report)
Seasonal distribution = uniform
Design period = 20 years
Traffic growth factor = 5% (for new construction)
Design ESALs = 207 x 365 x 33.06
= 2.5×10^6 (in design lane)

Serviceability

Initial Serviceability = 4.2
Terminal Serviceability = 2.5

Reliability

R = 85% (for 2.5×10^6 Design ESALs) in accordance with Table 8.3.

Overall Standard Deviation

Overall Standard Deviation $S_O = 0.45$

Environmental Impacts

The same comments provided in Design Example 1 would apply.

Effective Roadbed Resilient Modulus

The same comments provided in Design Example 1 would apply.

Structural Layer Coefficients

The structural layer coefficients were selected in accordance with Table 8.4.

Drainage Coefficients

A drainage coefficient of 1.0 was selected for both the base and subbase courses.

Design of Pavement Structure

Selection of Structural Number

Since the Effective Roadbed M_R value was 30 MPa, the chart for 30 MPa was selected for design purposes. Using this chart and the Design ESALs of 2.5×10^6, a Structural Number SN of 119 mm was determined for a Reliability of 85%. This is presented in Figure 8.9.

Selection of Layer Thickness

The approach presented in Design Example 1 was followed using various combinations of layer types, thicknesses and coefficients. These trial sections are presented in Table 8.8.

TABLE 8.8 TRIAL SECTIONS FOR SN= 119 mm

Trial No.	Structural Layer	ai	Di (mm)	mi	SNi	sum SNi	SN Total
1	ACP	0.40	180	1.0	72	72	
	GBC	0.14	350	1.0	49	121	121
2	ACP	0.40	180	1.0	72	72	
	GBC	0.14	200	1.0	28	100	
	GSBC	0.10	200	1.0	20	120	120
3	ACP	0.40	250	1.0	100	100	
	GBC	0.14	150	1.0	21	121	121
4	ACP	0.40	180	1.0	72	72	
	CSBC	0.23	210	1.0	48	120	120

Four acceptable pavement sections that satisfy the design criteria are:

Structural Layer	Trial 1	Trial 2	Trial 3	Trial 4
ACP	180 mm	180 mm	250 mm	180 mm
GBC	350 mm	200 mm	150 mm	-
GSBC	-	200 mm	-	-
CSBC	-	-	-	210 mm

Other combinations of thicknesses could be used to satisfy the SN of 119, depending upon availability and cost of the individual components in place.

Based upon knowledge that large deposits of high quality aggregates are located near the project, the recommended pavement structure is:

100 mm ACP	(thickness to be confirmed at the time of the final stage design)
80 mm ACP	first stage
350 mm GBC	

The computerized design system design system DARWin 3.0 was also used to carry out the pavement design. The output of the Flexible Structural Design Module run in metric mode for the recommended pavement structure is presented in Table 8.9.

Material Types

Granular Base Course

Based on the knowlede of the aggregate in the pit, Designation 2 Class 40 is selected for the project since this meets the crush count criteria.

Asphalt Concrete Mix Type

The project is located in Zone D. Based upon the design ESALs of 2.5×10^6, Asphalt Mix Type 2 is selected from Table 2.2.

Numerous other designs with different parameters can be readily and quickly performed in a similar manner with the DARWin Pavement Design and Analysis System.

TABLE 8.9 DARWIN 3.0 OUTPUT FOR RECOMMENDED PAVEMENT STRUCTURE

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
AGRA Earth & Environmental
4810 -93 St
Edmonton, AB
Canada

Flexible Structural Design Module

Project 2

Flexible Structural Design

80-kN ESALs Over Initial Performance Period	2,500,000
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	85 %
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	30,000 kPa
Stage Construction	1
Calculated Design Structural Number	119 mm

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	acp	0.4	1	180	-	72
2	gbc	0.14	1	350	-	49
Total	-	-	-	530	-	121

8.5.3 Staged Design

The pavement structures provided in the design examples would apply directly to unstaged construction. For staged design and construction, which is the usual case for new construction, the first stage design would include the total base and subbase thickness and a portion of the total design ACP. The recommended minimum thickness of first stage ACP would be selected based on the values presented in Table 8.5 in consideration of other project specific conditions.

8.6 Low Volume Roads

Historically in Alberta, the hard surfacing of low volume roads has been accomplished through the design and construction of "Sealed Bases" and conventional pavement structures.

Sealed bases consist of layers of conventional granular base course, with a final running surface of two applications of a high float seal coat. Sealed bases are known to perform well, provided traffic volumes are light and the surface is properly maintained. It is AT&U experience that sealed bases, when properly designed and constructed, can carry relatively small numbers of fully loaded trucks, without failure, for a similar number of years to conventionally designed pavement structures. However, the sealed base structure is fragile and if truck traffic increases, an asphalt concrete pavement surface should be designed and constructed.

Conventional pavement structures generally consist of layers of granular base with a temporary running surface of asphalt stabilized base course and a final pavement layer of asphalt concrete, applied later. Conventional pavements are very smooth and are known to perform well.

The design charts provided in the Manual would apply to the design of conventional pavement structures, which would include an ASBC or ACP wearing surface, for design ESALs greater than 2 x 10^4 ESALs. The recommended levels of reliability to be used are presented in Table 8.3.

The design charts are not considered appropriate for the structural design of sealed base structures.

9 FINAL STAGE PAVEMENTS

9.1 Introduction

Most new pavement structures in Alberta are built using the staged design and construction concept. According to Alberta's past practice, the first stage pavement consists of the subbase (if any) and granular base course, and a minimum of 60 mm (based upon Table 8.5 and the project design ESALs) of the first stage asphalt concrete layer. (Where special conditions warrant, a cement stabilized base course may be designed.) The final asphalt concrete pavement stage is deferred and is designed and constructed one or more years following original construction.

The structural evaluation of the existing first stage pavement structure considered for final paving is standard practice in Alberta. Until 1991, AT&U used the Benkelman beam to measure pavement deflections. Based on the deflections the required final stage pavement thickness was determined. The design method which was used by AT&U was developed by Alberta Research Council (ARC) and was an empirical procedure.

In 1989, with the acquisition by the Department of its first FWD, work commenced to replace the existing empirical design method with a mechanistic-empirical approach. It was decided at that time to adopt the ELMOD computer program as the design methodology to interpret the FWD deflection data. ELMOD is a computer program which incorporates a simplified mechanistic-empirical design procedure.

AT&U practice was to design the final stage pavement thickness based upon several inputs: ELMOD analysis, new construction design principles, past experience and engineering judgement.

More recently, the AASHTO method for the design of pavement overlays, based upon nondestructive deflection testing of the existing pavement with the FWD, has been adopted by AT&U. A computer program called DARWin (Design, Analysis and Rehabilitation for WINdows) is used to analyze the FWD deflection data and determine final stage pavement thickness requirements using AASHTO design procedures for rehabilitation of existing pavements and design input values developed for Alberta conditions.

Pavements are tested using the FWD, usually one or two years following construction of the first stage based upon the roadway classification and design traffic. The collection of FWD test data and other pavement evaluation testing is coordinated by AT&U. The raw FWD test results will be provided, along with any available pavement evaluation test results, to the consultant retained to carry out the pavement design. These are discussed in more detail in Section 4.

The design of final stage pavements is therefore very similar to the design of structural overlays. The main difference between the two design approaches is that in the design of structural rehabilitation, there may be several cost-effective rehabilitation strategies available to the designer to remedy both functional and structural pavement distresses.

The design of a final pavement structure in most cases is governed by structural needs because the first stage pavement is usually not old enough to exhibit distresses related to traffic loading and the environment. Therefore the design strategy is generally to only increase the structural capacity of the pavement structure through the addition of an ACP layer.

9.2 AASHTO Design Procedures

9.2.1 Background

The AASHTO methodology and design procedures for rehabilitation of existing pavements is presented in PART III of the Guide [AASHTO 93] and would apply to the design of final pavement structures. The design steps to determine the required final stage pavement thickness as outlined in the Guide are:

1. Assessment of existing pavement design and construction

2. Traffic analysis

3. Condition survey

4. Deflection testing

5. Coring and materials testing

6. Determination of required structural number for future traffic (SN_f). The effective design subgrade modulus is determined by backcalculation from deflection data.

7. Determination of effective structural number (SN_{eff}) of the existing pavement. The determination of SN_{eff} is based upon an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The effective modulus of the pavement layers is backcalculated from FWD deflection data.

8. Determination of overlay thickness. The thickness of the AC overlay is computed as follows:

$$D_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{(SN_f - SN_{eff})}{a_{ol}}$$

where

SN_{ol}	=	Required overlay Structural Number
a_{ol}	=	Structural layer coefficient for the AC overlay
D_{ol}	=	Required overlay thickness
SN_f	=	Structural Number for future traffic
SN_{eff}	=	Effective Structural Number of the existing pavement

DARWin 3.0 [DARWIN 97] is used to analyze the FWD deflection data to determine the backcalculated subgrade and pavement moduli. These values along with other required design inputs are used to determine pavement overlay requirements in accordance with the design methodology presented in the AASHTO Guide. Familiarity with the AASHTO Guide and knowledge of the requirements of this design manual, AASHTO design principles and FWD testing methods are required in order to use the DARWin program as an effective design and analysis tool.

9.2.2 Design Inputs

The adaptation of AASHTO design methodology required the calibration of AASHTO input design parameter values to past and present AT&U practice, experience and pavement performance. A number of projects were selected to evaluate and compare ELMOD and AASHTO design methodologies. Based upon the analysis of these designs, values for the required design inputs were determined for Alberta conditions. In general, the use of these design inputs and values for the design of final stage pavements will result in total pavement structures that fit AT&U typical pavement structures for Design ESAL levels less than about 1×10^6 and reasonable total pavement structures for Design ESAL levels greater than about 1×10^6. It was recognized that the design input parameters and values will need to be validated and updated based upon pavement performance and consultant experience as designers become more familiar and experienced with the new design method.

The design inputs required by the AASHTO method and the DARWin 3.0 program along with recommended values follow.

FWD Deflection Data

The FWD raw data provided to consultants by AT&U are in electronic files generated by the Dynatest Field Program Edition 25.03. These files include basic project identification information, pavement temperatures and uncorrected deflections in µm for all nine sensors and for the three loading drops. Text comments may also be included.

The format of the Dynatest 25.03 data file cannot be read by DARWin 3.0. The designer will be required to develop a program that will convert the Dynatest 25.03 data file into a format that DARWin can recognize. This conversion program must be able to:

- remove any comments
- convert load, deflections and pavement temperatures to Imperial units, eg. lbf, mils, F.
- select seven sensors to be used in the design.

DARWin 3.0 only processes seven of the nine deflections. In general, it is recommended that the following sensor arrangement be used:

Sensor No.	**Location**		**Used**	**Not Used**
No. 1	0 mm	(0")	T	
No. 2	200 mm	(8")		T
No. 3	300 mm	(12")	T	
No. 4	450 mm	(18")	T	
No. 5	600 mm	(24")	T	
No. 6	900 mm	(36")	T	
No. 7	1200 mm	(48")	T	
No. 8	1500 mm	(60")	T	
No. 9	1800 mm	(72")		T

The deflection from the No. 1 sensor is used to determine the backcalculated pavement stiffness modulus. One of the outer sensors is used to determine the backcalculated subgrade modulus. Experience to date has indicated that the sensors at 200 mm and 1800 mm are not normally required. The DARWin 3.0 program reports which sensor was used to backcalculate the subgrade modulus. The designer should be aware that it may be necessary to adjust which sensors should be used.

It may be useful to apply the ASTM Standard Guide [ASTM 94] or procedures developed as a product from the Strategic Highway Research Program [BRE 97] for analyzing deflection basin results.

Design Traffic

The Design ESALs/lane for the design period is determined as outlined in Section 5.

Serviceability Loss

The recommended values for Initial Serviceability (p_o) and Terminal Serviceability (p_t) are 4.2 and 2.5 respectively.

Reliability

The Reliability to be used for final stage pavement design is a function of the Design ESALs shown in Table 9.1:

TABLE 9.1 RECOMMENDED LEVELS OF RELIABILITY FOR FINAL STAGE PAVEMENTS

Design ESALs (x10^6)	**Reliability (%)**
<0.1	75
0.1 - 5.0	75
5.0 - 10.0	85
> 10.0	90

Overall Standard Deviation, S_O

The recommended value for Overall Standard Deviation is 0.45.

Resilient Modulus Correction Factor

The recommended combined factor to adjust the backcalculated subgrade modulus to make it consistent with the value used to represent the AASHO Road Test subgrade and to adjust it for seasonal variations in subgrade strength is 0.36. Further background to this adjustment factor is presented in Section 2.2.3.

Structural Coefficient

The recommended value for the structural layer coefficient of asphalt concrete is 0.40.

Other Design Inputs

Other design inputs include the Total Pavement Thickness (combined thickness of all pavement layers above the subgrade), the Existing AC Thickness (total thickness of all asphalt bound layers), Pavement Temperature at the time of FWD testing (this is included in the FWD data file), pavement base type (granular, cement treated, etc.), and Milling Thickness (any material that will be milled is not counted in the structural contribution of the existing pavement) which would normally be 0.

9.2.3 DARWin 3.0

DARWin 3.0 allows the designer to utilize FWD deflection data to determine the subgrade soil and pavement moduli through back calculation techniques built into the program. The original version of DARWin 3.0 was released in April 1997 and replaced DARWin 2.0. In addition to its capabilities to design both rigid and flexible new pavement structures and pavement overlays, the program also include other features such as life cycle cost analysis and the ability to convert metric/Imperial units.

For the design of pavement overlays, DARWin 3.0 will allow the designer to determine the average backcalculated subgrade and pavement moduli and required overlay thickness for any user specified section of roadway. The program will also provide profile plots of subgrade or pavement moduli over the length of roadway being analyzed. However, the current program will not provide profile plots of required final stage pavement thickness on a point-by-point basis. DARWin 3.0 also currently lacks flexibility with respect to exporting of analysis results. It will be necessary for consultants to develop required plots and other report formats by manually inputting DARWin 3.0 output into a spreadsheet (Excel or Lotus). Users are also cautioned that as with any new programs, future modification and enhancements will be required as the program is implemented and more widely tested.

9.3 Methodology - Final Stage Pavement Design

The recommended design steps to be followed by consultants using AASHTO methodology and DARWin 3.0 are:

1. Review Existing Information (Existing pavement structures, historical strength test results, historical QA test results, etc.)

2. Carry Out Preliminary Pavement Structural Design

- Establish Design Inputs (Design period, Design ESALs, Reliability, subgrade correction factors, etc.)
- Analyze Project
  - Divide project into preliminary sections based on design ESALs, existing pavement structure and pavement performance and condition.
  - Analyze preliminary sections in 0.5 km segments using DARWin 3.0 and review point-by-point FWD deflections and moduli values. Determine the backcalculated moduli and the final stage pavement thickness requirement for each segment.
- Develop preliminary structural pavement design
  - Subdivide project into uniform sections based on backcalculated moduli and final stage thickness requirements for each segment and determine average backcalculated moduli and preliminary final stage pavement thickness for each uniform section using DARWin 3.0.

3. Carry Out Field Evaluation

- Confirm/validate pavement performance
- Identify requirements for preliminary leveling, localized repairs, reprofiling, etc.
- Identify needs for field sampling and testing.

4. **<u>Carry Out Laboratory Testing And Evaluation</u>** (if required)

5. **<u>Finalize Design, Documentation And Report</u>**

- The final report should summarize all background information and test data referenced; document all design analysis, field observations, additional field testing, laboratory testing and evaluation, and all assumptions; and provide the recommended strategy and detailed design and ACP Mix Type selection.

Depending upon the complexity and other project specific requirements, it may not be necessary to carry out each step in detail. Alternatively, it may be necessary to broaden the design process in some cases. In general, it is expected that each step will be addressed by the consultant.

9.4 Design Example - Final Stage Pavement

Highway 3:10 Eastbound Lanes from Lethbridge to Junction Highway 36N, km 13.58 to km 21.30, was designed as two lane roadway to twin the existing roadway, with 3.75 m wide lanes and 2.5 m wide paved median shoulder and 3.0 m wide paved outside shoulder. The subgrade, granular base and first stage ACP layer was constructed in 1994. FWD testing of this facility was carried out in August 1995. The final stage pavement was programmed for construction in 1996.

This example is for the eastbound outer travel lane.

<u>1. Review Existing Information</u>

The Pavement Management System Status Summary report indicates the following structure for the outside eastbound lane:

<u>Section</u>	<u>Structure</u>
13.58 - 21.30	60mm ACP 1994 300mm GBC 1994 CL Subgrade

2. Carry Out Preliminary Pavement Structural Design

2.1 Establish Design Inputs

- FWD Deflection Data - The FWD testing was carried out in July 1995. The raw data was converted into a format recognizable by DARWin. 3.0. The recommended seven sensors were used.
- Design ESALs - ESALs/day/direction = 567 (1993 Traffic Volumes Report)
  Lane distribution = 85% outside lane: 15% inside lane
  Seasonal distribution = uniform
  Design period = 20 years
  Traffic growth factor = 3% (changed from 5% based on local experience)
  Design ESALs = 567 x .85 x 365 x 26.87
  = 4.7 x 10^6 (in design lane)
- Serviceability Initial Serviceability = 4.2
  Terminal Serviceability = 2.5
- Reliability R = 75% (for 4.7 x 10^6 Design ESALs) in accordance with Table 9.1
- Overall Standard Deviation S_O = 0.45
- Resilient Modulus Correction Factor = 0.36

2.2 Analyze Project

- The project was broken into preliminary sections based upon existing pavement structure. Each 0.5 km section was analyzed using DARWin 3.0 and the design input values. A review of the profile plots of the FWD central deflection (Figure 9.1) highlighted one single test result at km 16.7 which was considered an outlier and was not included in further analysis. Special note was made to take a close look at this location during the field evaluation. The DARWin 3.0 reported average subgrade modulus, pavement modulus and required final stage pavement thickness for each 0.5 km section was manually transferred into Excel. Excel profile plots generated are presented in Figure 9.2.

- Based upon a visual evaluation of the profile plots, the project was divided into the following uniform sections:

  km 13.5 - km 14.5
  km 14.5 - km 21.3

The actual individual deflection data and backcalculated moduli values for each tested location between km 13.5 to 14.5 were reviewed. This review indicated that the data and values were consistent and reflected the roadway structural condition.

2.3 Develop Preliminary Structural Pavement Design

The DARWin 3.0 output for the section from km 13.5 to 14.5 is presented in Figure 9.3. DARWin 3.0 was used to determine the average required final stage pavement thickness for each section:

Section	Preliminary Overlay Thickness (mm)
km 13.5 - km 14.5	155
km 14.5 - km 21.3	131

This was considered the preliminary structural pavement design.

3. Carry Out Field Evaluation

The field evaluation indicated that the existing first stage pavement appeared to be in an acceptable condition with no signs of structural distress. The pavement condition in the area of km 16.7 was evaluated in detail. This area was located near a median cross-over. No particular distress was observed during the evaluation. This location should be inspected again at the time of construction to determine if any deterioration is present which could require special remediation or repair.

Additional field testing or sampling was not identified as a need.

4. Carry Out Laboratory Testing and Evaluation

A laboratory testing and evaluation program was not identified as a need.

5. Finalize Design, Documentation and Report

The final recommended final stage pavement thickness design was:

Section	Final Stage Pavement Thickness (mm)
km 13.5 - km 14.5	150 (60mm final lift)
km 14.5 - km 21.3	130 (60 mm final lift)

This is presented in Figure 9.4.

The pavement condition in the area of km 16.7 should be inspected again at the time of construction to determine if any deterioration is present which could require special remediation or repair.

In this case the pavement design for the inside lane is the same as for the outside lane since no centreline shift was allowed for during the construction of the first stage ACP.

The project is located in Zone B. Based upon the Designs ESALs of 4.7 x 10^6, Asphalt Mix Type 1 was selected from Table 2.2.

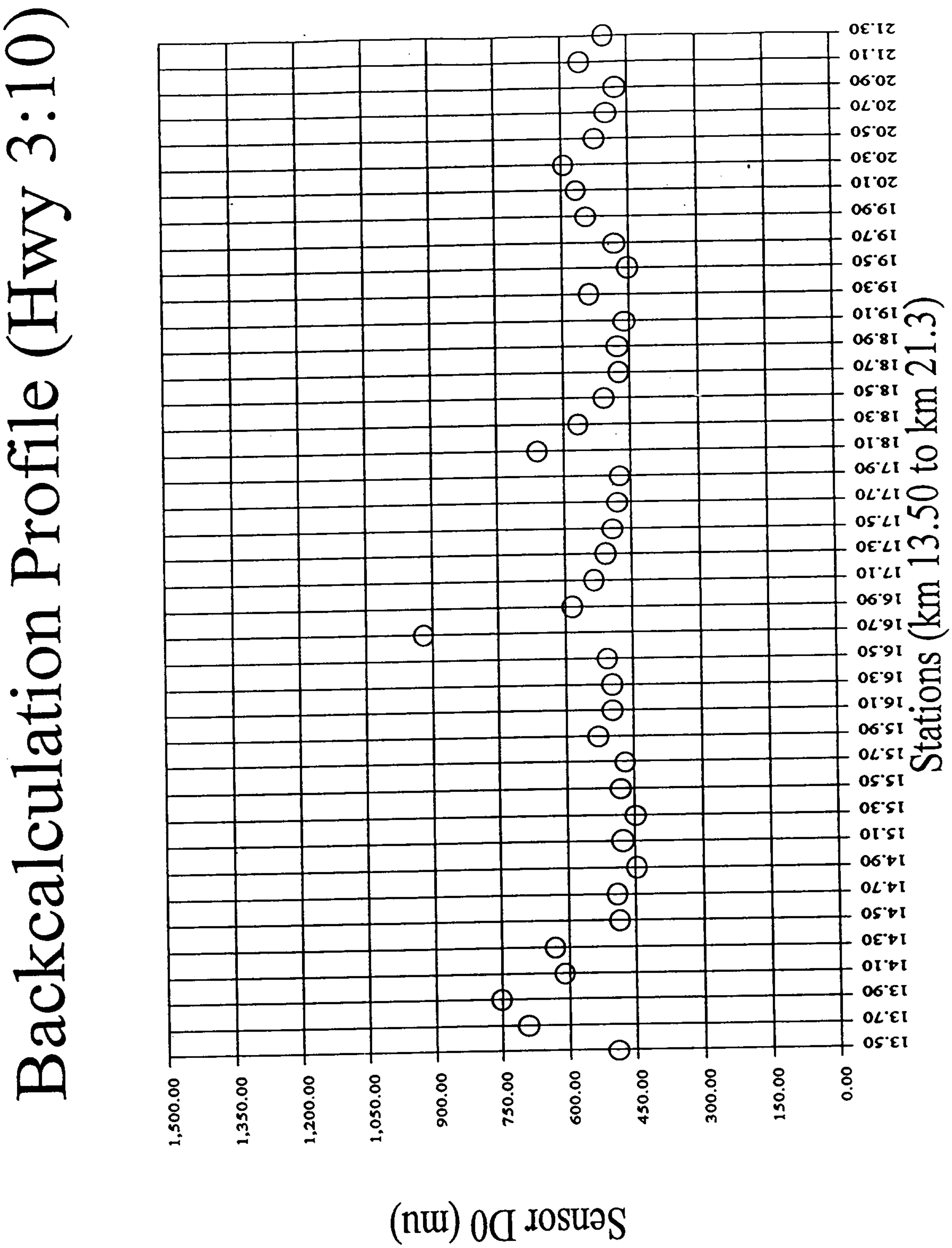

Figure 9.1 HW 3:10 - Sensor D_o Profile

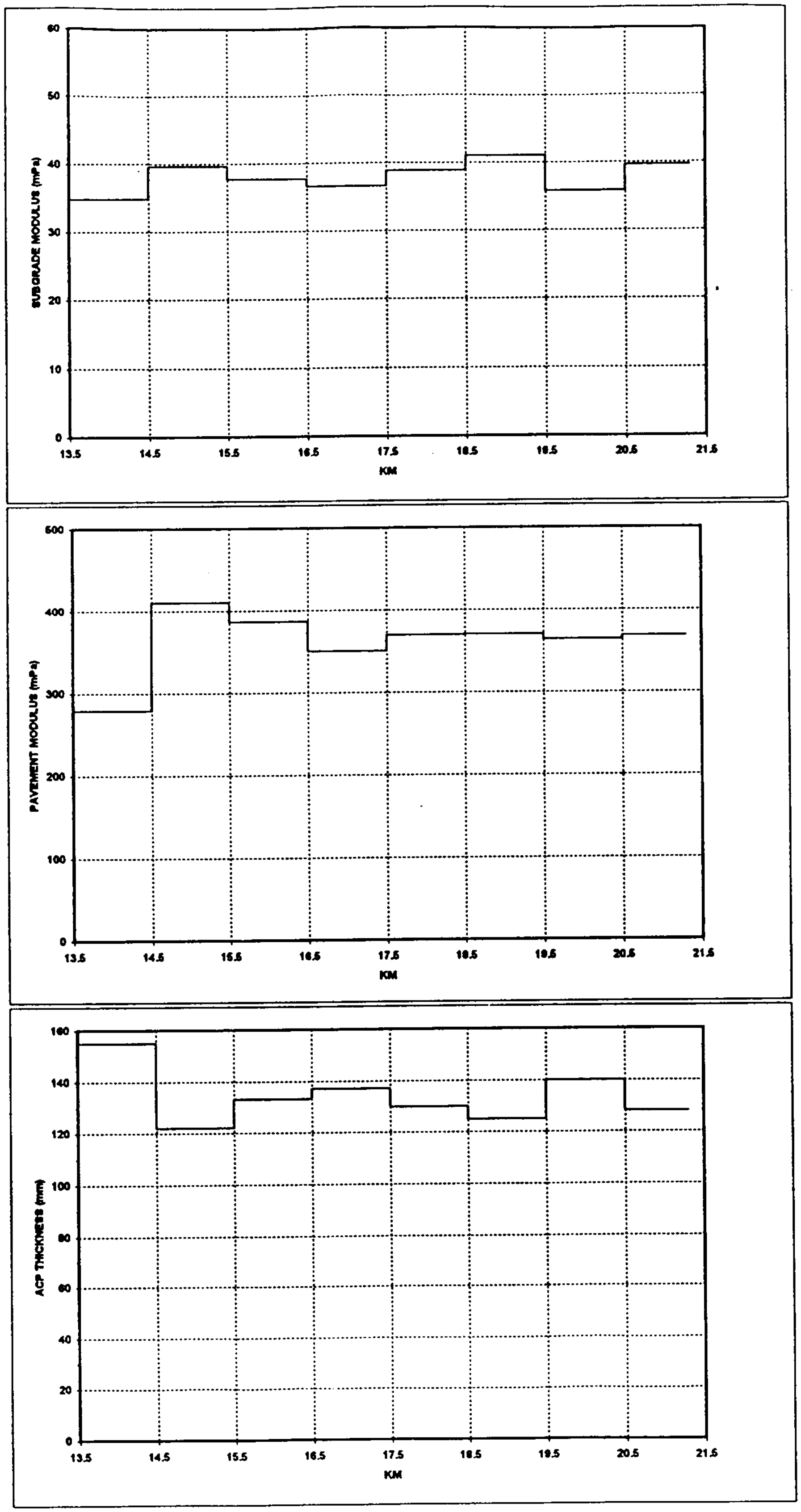

Figure 9.2 HW 3:10 - Profile Plots

1997 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
AGRA Earth & Environmental
4810 -93 St
Edmonton, AB
Canada

Overlay Design Module

H310, 13.500 - 14.500

AC Overlay of AC Pavement

Structural Number for Future Traffic 117.602 mm

Design Method	Effective Existing Structural Number (mm)	Overlay Structural Number (mm)
Component Analysis	-	-
Remaining Life	-	-
Non-Destructive Testing	55.626	62

Structural Number for Future Traffic

Future 18-kip ESALs Over Design Period	4,700,000
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	75 %
Overall Standard Deviation	0.45
Subgrade Resilient Modulus	34,873.69608 kPa
Calculated Structural Number for Future Traffic	118 mm

Effective Structural Number - Non-Destructive Testing

Total Pavement Thickness	360.68 mm
Backcalculated Effective Pavement Modulus	278,644.83064 kPa
Milling Thickness	0 mm
Effective Existing Pavement SN (SNEff)	56 mm

Backcalculation - M:\MAT\FWD\H310\H310_MET.FWD

Total Pavement Thickness	360.68 mm
Resilient Modulus Correction Factor, C	0.36
Existing AC Thickness	60.96 mm
Base Type	Granular
Data Evaluation Basis	Mean

Calculated Results

Subgrade Resilient Modulus (MR)	34,873 kPa
Effective Pavement Modulus (Ep)	278,643 kPa
Dynamic k-value	- kPa/mm

Effective Roadbed Soil Resilient Modulus

Period	Description	Roadbed Resilient Modulus (kPa)
Calculated Effective Modulus	- kPa*	

*Note: This value is not represented by the inputs or an error occurred in calculation.

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(mm)	Width (m)	Calculated SN (mm)
1	ACP	0.4	1	155	-	62
Total	-	-	-	155	-	62

Figure 9.3 Hwy 3:10 - DARWin 3.0 Output

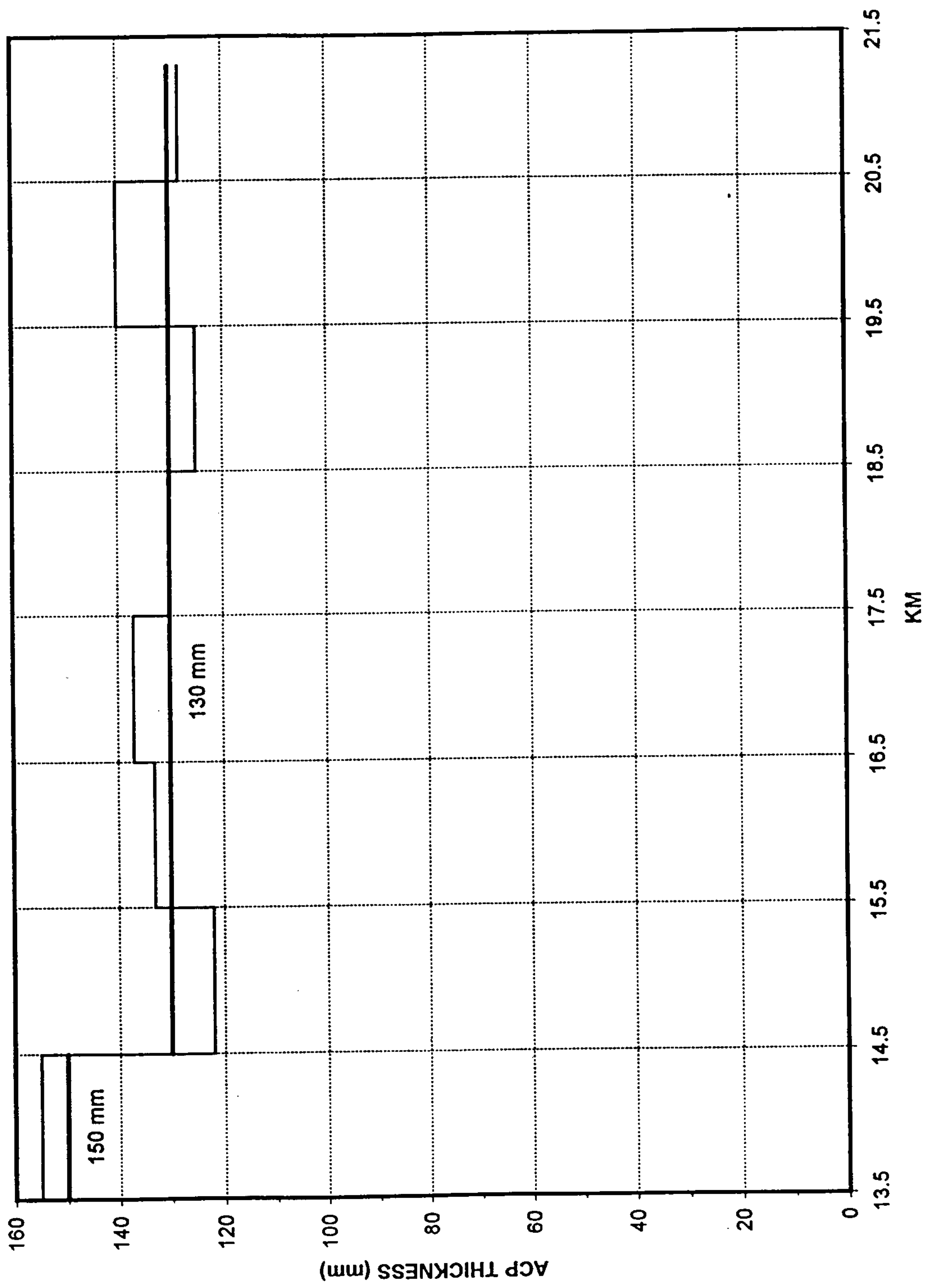

Figure 9.4 HW 3:10 Recommended Final Stage Pavement Design

10 REHABILITATION

10.1 Introduction

The roadway network in Alberta has gone through a rapid expansion in the 1950's, 1960's and 1970's. Many of the pavements are now reaching the end of their service lives. Rehabilitation treatments are required to further extend pavement life and provide acceptable serviceability.

As the expansion of the highway network is reduced and the existing network matures, a greater proportion of transportation budgets will be focused on maintenance and rehabilitation of paved roadways.

The reasons for rehabilitating a pavement include:

- unacceptable level of service in terms of riding comfort
- unacceptable level of distress
- inadequate structural capacity for expected traffic loads
- unacceptable level of safety
- unacceptable maintenance costs.

In the design of structural rehabilitation, factors such as traffic, environment, pavement materials, performance prediction and economic evaluation of alternative design strategies need to be considered.

The design of structural rehabilitation is therefore very similar to the design of final pavement structures. The main difference between the two design approaches is that in the design of structural rehabilitation, there may be several cost-effective rehabilitation strategies available to the designer to remedy both functional and structural pavement distresses. The design of a final pavement structure in most cases is governed by structural needs because the first stage pavement is usually not old enough to exhibit distresses related to traffic loading and the environment.

The structural evaluation of the existing pavement considered for rehabilitation is standard practice in Alberta. Until 1991, AT&U used the Benkelman beam to measure pavement deflections. Based on the deflections the required structural overlay thickness was determined. The design method which was used by AT&U was developed by Alberta Research Council (ARC) and was an empirical procedure.

In 1989, with the acquisition by the Department of its first FWD, work commenced to replace the existing empirical design method with a mechanistic-empirical approach. It was decided at that time that the ELMOD computer program be adopted as the design methodology to interpret the FWD deflection data. ELMOD is a computer program which incorporates a simplified mechanistic-empirical design procedure.

More recently, the AASHTO method for the design of pavement overlays based upon nondestructive deflection testing of the existing pavement with the FWD has been adopted by AT&U. A computer program called DARWin (Design, Analysis and Rehabilitation for WINdows) is used to analyze the FWD deflection data and determine pavement overlay requirements in accordance with the design methodology presented in the AASHTO Guide and using design input values developed for Alberta conditions.

Pavements are tested using the FWD, usually one or two years prior to a project being programmed for rehabilitation. The collection of FWD test data and other pavement evaluation testing is coordinated by AT&U. The raw FWD test results are provided, along with all other pavement evaluation test results, to the consultant retained to carry out the pavement design. These are reported on in more detail in Section 4.

10.2 AASHTO Rehabilitation Design Procedures

10.2.1Background

The AASHTO methodology and design procedures for rehabilitation of existing pavements is presented in PART III of the Guide [AASHTO 93]. The design steps to determine the required overlay thickness as outlined in the Guide are:

1. Assessment of existing pavement design and construction.
2. Traffic analysis
3. Condition survey
4. Deflection testing
5. Coring and materials testing
6. Determination of required structural number for future traffic (SN_f). The effective design subgrade modulus is determined by backcalculation from deflection data.
7. Determination of effective structural number (SN_{eff}) of the existing pavement. The determination of SN_{eff} is based upon an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The effective modulus of the pavement layers is backcalculated from FWD deflection data.

8. Determination of overlay thickness. The thickness of the AC overlay is computed as follows:

$$D_{ol} = \frac{SN_{ol}}{a_{ol}} = \frac{(SN_f - SN_{eff})}{a_{ol}}$$

where

SN_{ol}	=	Required overlay Structural Number
a_{ol}	=	Structural layer coefficient for the AC overlay
D_{ol}	=	Required overlay thickness
SN_f	=	Structural Number for future traffic
SN_{eff}	=	Effective Structural Number of the existing pavement

DARWin 3.0 [DARWIN 97] is used to analyze the FWD deflection data to determine the backcalculated subgrade and pavement moduli. These values along with other required design inputs are used to determine pavement overlay requirements in accordance with the design methodology presented in the AASHTO Guide. Familiarity with the AASHTO Guide and knowledge of the requirements of this design manual, AASHTO design principles and FWD testing methods are required in order to use the DARWin program as an effective design and analysis tool.

10.2.2Design Inputs

The adaptation of AASHTO design methodology required the calibration of AASHTO input design parameter values to past and present AT&U practice, experience and pavement performance. A number of projects were selected to evaluate and compare ELMOD and AASHTO overlay design methodologies. Based upon the analysis of these designs, values for the required design inputs were determined in order that similar overlay thicknesses would be provided by the AASHTO method as had been provided in the past for low and medium Design ESAL levels. The AASHTO method may provide thicker overlays for very high Design ESALs than provided for in the past.

It was recognized that the design input parameters and values will need to be validated and updated based upon pavement performance and consultant experience as designers become more familiar and experienced with the new design method.

The design inputs required by the AASHTO method and the DARWin 3.0 program along with recommended values follow.

FWD Deflection Data

The FWD raw data provided to consultants by AT&U are in electronic files generated by the Dynatest Field Program Edition 25.03. These files include basic project identification information, pavement temperatures and uncorrected deflections in µm for all nine sensors for the three loading drops. Text comments may also be included.

The format of the Dynatest 25.03 data file cannot be read by DARWin 3.0. The consultant will be required to develop a program that will convert the Dyntatest 25.03 data file into a format that DARWin can recognize. This conversion program must be able to:

- remove any comments
- convert the load, deflections and pavement temperatures to Imperial units, eg. lbf, mils, F.
- select seven sensors to be used in the design.

DARWin 3.0 only processes seven of the nine deflections. In general, it is recommended that the following sensor arrangement be used:

Sensor No.	Location		Used	Not Used
No. 1	0 mm	(0")	T	
No. 2	200 mm	(8")		T
No. 3	300 mm	(12")	T	
No. 4	450 mm	(18")	T	
No. 5	600 mm	(24")	T	
No. 6	900 mm	(36")	T	
No. 7	1200 mm	(48")	T	
No. 8	1500 mm	(60")	T	
No. 9	1800 mm	(72")		T

The deflection from the No. 1 sensor is used to determine the backcalculated pavement stiffness modulus. One of the outer sensors is used to determine the backcalculated subgrade modulus.

Experience to date has indicated that the sensors at 200 mm and 1800 mm are not normally required. The DARWin 3.0 program reports which sensor was used to backcalculate the subgrade modulus. The designer should be aware that it may be necessary to adjust which sensors should be used.

It may be useful to apply the ASTM Standard Guide [ASTM 94] or procedures developed as a product from the Strategic Highway Research Program [BRE 97] for analyzing deflection basin results.

Design Traffic

The Design ESALs/lane for the design period is determined as outlined in Section 5.

Serviceability Loss

The recommended values for Initial Serviceability (p_o) and Terminal Serviceability (p_t) are 4.2 and 2.5 respectively.

Reliability

The Reliability to be used for overlay design is a function of the Design ESALs as shown in Table 10.1

TABLE 10.1 RECOMMENDED LEVELS OF RELIABILITY FOR REHABILITATION OVERLAY DESIGN

Design ESALs ($x10^6$)	**Reliability (%)**
<0.1	50
0.1 - 5.0	50
5.0 - 10.0	75
> 10.0	85

Overall Standard Deviation, S_O

The recommended value for Overall Standard Deviation is 0.45.

Resilient Modulus Correction Factor

The recommended combined factor to adjust the backcalculated subgrade modulus to make it consistent with the value used to represent the AASHO Road Test subgrade and to adjust it for seasonal variations in subgrade strength is 0.36. Further background to this adjustment factor is presented in Section 2.2.3.

Structural Layer Coefficient

The recommended value for the structural layer coefficient of asphalt concrete is 0.40.

Other Design Inputs

Other design inputs include the Total Pavement Thickness (combined thickness of all pavement layers above the subgrade), the Existing AC Thickness (total thickness of all asphalt bound layers), Pavement Temperature at the time of FWD testing (this is included in the FWD data file), pavement base type (granular, cement treated, etc.), and Milling Thickness (any material that will be milled is not counted in the structural contribution of the existing pavement).

10.2.3 DARWin 3.0

DARWin 3.0 allows the designer to utilize FWD deflection data to determine the subgrade soil and pavement moduli through back calculation techniques built into the program. The original version of DARWin 3.0 was released in April 1997 and replaced DARWin 2.0. In addition to its capabilities to design both rigid and flexible new pavement structures and pavement overlays, the program also include other features such as life cycle cost analysis and the ability to convert metric/Imperial units.
For the design of pavement overlays, DARWin 3.0 will allow the designer to determine the average backcalculated subgrade and pavement moduli and required overlay thickness for any user specified section of roadway. The program will also provide profile plots of subgrade or pavement moduli over the length of roadway being analyzed. However, the current program will not provide profile plots of required overlay thickness on a point-by-

point basis. DARWin 3.0 also currently lacks flexibility with respect to exporting of analysis results. It will be necessary for consultants to develop required plots and other report formats by manually inputting DARWin 3.0 output into a spreadsheet (Excel or Lotus). Users are also cautioned that as with any new programs, future modification and enhancements will be required as the program is implemented and more widely tested.

10.3 Rehabilitation Design Strategies

Where rehabilitation is required only to strengthen the pavement structure, the design strategy for the project may be relatively straight forward and simple.

However in many cases, although rehabilitation may be governed by a structural deficiency and a pavement overlay is required to strengthen the existing pavement structure, the pavement may also exhibit VCI and RCI needs. In these cases the overlay design process will only identify the thickness of the overlay required to strengthen the pavement structure. This design thickness may need to be increased based upon project logistics and existing pavement condition to allow for a two lift overlay in order to restore the pavement cross-section, fill ruts, treat existing crackfiller etc.. Alternatively, there may be other more cost effective strategies that may be appropriate for the project.

Potential rehabilitation strategies include:

$ Thick two lift structural overlay ($ 80mm)
$ Two lift overlay (20 mm + 50mm min.)
$ One lift overlay (60 mm)
$ Reprofile by cold milling plus overlay (50 mm min.)
$ HIR (Hot In-place Recycling) plus overlay (50 mm min.)
$ Mill & inlay plus overlay (50 mm min.)
$ Preoverlay repairs (eg. crack pre-treatment) plus overlay.

Rehabilitation needs frequently are related to pavement roughness. Pavement roughness distress may be due to frost heaving, differential settlements, rutting, pavement cracks ('cupping' or 'dipping' resulting from the ingress of water into the pavement structure) or localized pavement failures.

There are cases where pavement rehabilitation is required only to restore the riding quality of the pavements. In many other cases, although pavement rehabilitation may be driven by roughness needs, there may be a requirement for some degree of pavement strengthening to achieve the design pavement life.

In cases where the need for rehabilitation is governed by roughness needs and no additional thickness of the pavement structure is required to strengthen the pavement for the design period, potential rehabilitation strategies include:

- $ HIR
- $ Mill & inlay
- $ Two lift overlay
- $ Preoverlay repairs (eg crack pre-treatment) plus overlay.

Economic analysis following the life cycle costing approach outlined in Section 6 must be carried out to evaluate potential strategies. Pavement performance experience with respect to actual service lives of all potential alternative strategies may be limited. It will be necessary for the design engineer to estimate the service lives and associated maintenance costs of the alternative strategies being evaluated based upon available information, experience and engineering judgement.

For primary highways, approximately 38% of the network has been overlaid once and approximately 12% of the network has been overlaid two or more times. A network analysis of primary highway pavement performance data by AT&U indicated the following service lives have been historically been achieved:

Pavement Base Type	Average Service Life of First Overlay (yrs.)
Granular Base	14 - 15
Cement Stabilized Base	12 - 14

There may be exceptional circumstances where reconstruction should be considered as a rehabilitation strategy. In these cases the original roadbed may be the cause of the reduced serviceability and excessive maintenance costs and rehabilitation of the pavement surface may only provide a very short term solution. A detailed geotechnical investigation would be required to identify causes of the poor pavement performance, eg frost heaving, swelling soils, poor drainage, organic materials etc. and necessary remedial measures. In these cases where reconstruction is required, consideration should be given to salvage of the existing asphalt concrete and granular base materials. The potential to recycle these materials on the project should be assessed through a laboratory testing and evaluation program as discussed Section 2. The pavement design for reconstruction sections would follow the methodology outlined in Section 8.

10.4 Methodology - Rehabilitation Design

The recommended design steps to be followed by consultants using AASHTO methodology and DARWin 3.0 are:

1. **<u>Review Existing Information</u>** (Existing pavement structures, RCI test results, historical strength test results, historical QA test results, etc.)

2. **<u>Carry Out Preliminary Pavement Structural Design</u>**

- Establish Design Inputs (Design period, Design ESALs, Reliability, subgrade correction factors, etc.)

- Analyze Project
  - Divide project into preliminary sections based on design ESALs, existing pavement structure and pavement performance and condition.
  - Analyze preliminary sections in 0.5 km segments using DARWin 3.0 and review point-by-point FWD deflections and moduli values. Determine the backcalculated moduli and overlay thickness requirement for each segment.

- Develop preliminary structural pavement design
  - Subdivide project into uniform sections based on backcalculated moduli and overlay thicknesses requirements for each segment and determine average backcalculated moduli and preliminary overlay thickness for each uniform section using DARWin 3.0.

3. **<u>Carry Out Field Evaluation</u>**

- Confirm/validate pavement performance
- Identify requirements for preliminary levelling, localized repairs, pre-overlay repairs, reprofiling, etc.
- Identify needs for field sampling and testing.

4. Carry Out Laboratory Testing And Evaluation (if required)

5. Finalize Alternative Strategies And Carry Out Life Cycle Cost Analysis

- Life cycle cost analysis will be required to assess alternatives where the serviceability of structurally adequate pavements needs to be restored. A detailed analysis may not be required in cases where multilift overlays are required to increase pavement structural capacity.

6. Finalize Design, Documentation And Report

- The final report should summarize all background information and test data referenced; document all design analysis, field observations, additional field testing, laboratory testing and evaluation, life cycle cost analysis and all assumptions; and provide the recommended strategy and detailed design and ACP Mix Type selection.

Depending upon the complexity and other project specific requirements, it may not be necessary to carry out each step in detail. Alternatively, it may be necessary to broaden the design process in some cases. In general, it is expected that each step will be addressed by the consultant.

10.5 Design Example - Structural Overlay

Highway 8:06, from the Junction Highway 22 to west of the Calgary City Limits, is a two lane roadway with 3.75 m wide lanes and 2.5 m wide paved shoulders. The base and pavement was designed and constructed in 1981 and can be considered 'unstaged'.

1. Review Existing Information

The Pavement Management System Status Summary report indicates the following;

Section	Structure	PQI	RCI	SAI	VCI	RUT (mm)
0.00 - 14.40	110mm ACP 1981 250mm GBC 1981 CI-CH Subgrade	4.4	5.1	3.9	5.6	6
14.40 - 15.80	160mm ACP 1981 250mm GBC 1981 CL Subgrade	5.1	5.2	6.3	5.9	5

These parameters indicate that the pavement is rough, with substantial surface distress, and minor rutting. The first section is substantially weaker than the second one. The RCI and Rut profiles are presented in Figures 10.1 and 10.2.

2. Carry Out Preliminary Pavement Structural Design

2.1 Establish Design Inputs

- FWD Deflection Data - The FWD testing was carried out in September 1996. The raw data was converted into a format recognizable by DARWin. 3.0. The recommended seven sensors were used.
- Design ESALs - ESALs/day/direction = 253 (1993 Traffic Volumes Report)
  Lane distribution = 50:50
  Seasonal distribution = uniform
  Design period = 20 years
  Traffic growth factor = 3%
  Design ESALs = 253 x 365 x 26.87
  $= 2.5 \times 10^6$ (in design lane)
- Serviceability Initial Serviceability = 4.2
  Terminal Serviceability = 2.5
- Reliability R = 50% (for 2.5×10^6 Design ESALs) in accordance with Table 10.1.
- Overall Standard Deviation $S_O = 0.45$
- Resilient Modulus Correction Factor = 0.36

2.2 Analyze Project

- The project was broken into preliminary sections based upon existing pavement structure. Each 0.5 km section was analyzed using DARWin 3.0 and the design input values. A review of the profile plots of the FWD central deflection (the plot for km 0.0 to 6.5 is presented in Figure 10.3) identified results at km 2.40 and km 5.00 that were considered outliers and were not included in any further analysis. Special note was make to take a closer look at these locations during the field evaluation. The DARWin 3.0 reported average subgrade modulus, pavement modulus and required overlay thickness for each 0.5 km section was manually transferred into Excel. Profile plots generated are presented in Figure 10.4.

- Based upon a visual evaluation of the profile plots, the project was divided into the following uniform sections:

  km 0.0 - km 6.5
  km 6.5 - km 10.5
  km 10.5 - km 13.0, km 13.5 - km 14.5
  km 13.0 - km 13.5
  km 14.5 - km 15.8

  The actual individual deflection data and backcalculated moduli values for each tested location between km 13.0 to 13.5 were reviewed. This review indicated that the data and values were consistent and reflected the roadway structural condition.

2.3 Develop Preliminary Structural Pavement Design

DARWIN 3.0 was used to determine the average required overlay thickness for each section:

Section	Preliminary Overlay Thickness (mm)
km 0.0 - km 6.5	133
km 6.5 - km 10.5	103
km 10.5 - km 13.0, km 13.5 - km 14.5	65
km 13.0 - km 13.5	110
km 14.5 - km 15.8	4

This was considered the preliminary structural pavement design.

3. Carry Out Field Evaluation

The field evaluation confirmed that varying degrees of structural distress as evidenced by intermittent fatigue cracking in outer wheel path locations existed from km 0.0 to about km 10.0 and km 13.0 to km 13.5. The pavement the area of km 2.40 and km 5.00 did not indicate any unusual distress or conditions. The section between km 10.0 to km 13.0, km 13.5 to km 14.5 and km 14.5 to 15.8 did not appear to exhibit any structural distress. Low temperature transverse cracking was evident between km 0.0 to 14.5; the crack locations exhibited a build up of crack filler. The section between km 14.5 to 15.8 exhibited only intermittent hairline transverse cracking; these cracks had not been filled. Additional field testing or sampling was not identified as a need.

4. Carry Out Laboratory Testing and Evaluation

A laboratory testing and evaluation program was not identified as a need.

5. Finalize Alternative Strategies and Carry Out Life Cycle Cost Analysis

Based upon a review of all available information, rehabilitation of this project was required to address pavement structural needs. In order to restore the pavement cross-section and minimize the risk of existing crackfiller affecting the construction of the pavement, a 20 mm nominal thickness levelling course was required for the sections between km 10.0 - km 13.0 and km 13.5 - km 14.5. The section between km 14.5 to km 15.8 required only a single lift overlay. A detailed life cycle cost analysis was not required.

6. Finalize Design, Documentation and Report

The final recommended overlay design was:

Section	Design Overlay Thickness (mm)
km 0.0 - km 6.5	130 (60mm final lift)
km 6.5 - km 10.5	100 (60 mm final lift)
km 10.5 - km 13.0, km 13.5 - km 14.5	80 (50 mm final lift)
km 13.0 - km 13.5	110 (60 mm final lift)
km 14.5 - km 15.8	50 (50 mm final lift)

This is presented in Figure 10.5.

The project is located in Zone D. Based upon the Design ESALs of 2.5 x 10^6, Asphalt Mix Type 2 was specified form Table 2.2.

10.6 Design Example - Non-Structural Rehabilitation

Highway 2:76, from km 26.62 to km 44.64, is located in northern Alberta near the British Columbia border. Highway 2:76 is a two lane roadway with 3.75 m wide lanes and 2.0 m wide paved shoulders.

1. Review Existing Information

The Pavement Management System Status Summary report indicated the following;

Section	Structure	PQI	RCI	SAI	VCI	RUT (mm)
26.62 - 44.64	125mm ACP 1975	4.3	4.6	5.5	4.7	7
	50mm ACP 1960					
	50mm ACP 1959					
	150mm CSBC 1959					
	Subgrade type unknown					

These parameters indicate that the pavement was very rough, with substantial surface distress, and minor rutting. The RCI and Rut profiles are presented in Figures 10.6 and 10.7 respectively.

2. Carry Out Preliminary Pavement Structural Design

2.1 Establish Design Inputs

- FWD Deflection Data - The FWD testing was carried out in October 1993. The raw data was converted into a format recognizable by DARWin. 3.0. The recommended seven sensors were used.
- Design ESALs - ESALs/day/direction = 170 (1993 Traffic Volumes Report)
  Lane distribution = 50:50
  Seasonal distribution = uniform
  Design period = 20 years
  Traffic growth factor = 3%
  Design ESALs = 170 x 365 x 26.87
  = 1.7×10^6 (in design lane)
- Serviceability Initial Serviceability = 4.2
  Terminal Serviceability = 2.5
- Reliability R = 50% (for 1.7×10^6 Design ESALs) in accordance with Table 10.1.
- Overall Standard Deviation $S_O = 0.45$
- Resilient Modulus Correction Factor = 0.36

2.2 Analyze Project

- The project had a uniform pavement structure and traffic conditions. Each 0.5 km section was analyzed using DARWin 3.0 and the design input values. A review of the profile plots of the FWD central deflection did not identify any obvious outliers which should be excluded from the analysis. The plot for the section from km 30.50 to km 35.00 is presented in Figure 10.8. The DARWin 3.0 reported average subgrade modulus, pavement modulus and required overlay thickness for each 0.5 km section was manually transferred into Excel. Profile plots generated are presented in Figure 10.9.

2.3 Develop Preliminary Structural Pavement Design

- Based upon a review of all available information, rehabilitation of this project was required to address pavement roughness needs and generally no additional thickness of pavement structure was required for strength purposes for a 20 year design period. The section from km 30.0 to km 30.5 indicated a need for a 3 mm structural overlay which was considered not to be significant enough to influence the choice of rehabilitation strategies.

3. Carry Out Field Evaluation

Field observations confirmed that the pavement was in good structural condition with no evidence of fatigue cracking or other structural-related distress. Significant transverse cracking was evident throughout the project which is characteristic of cement stabilized base pavements. The very low RCI resulted from roughness associated with the transverse cracking.

No additional field sampling or testing was identified.

4. Carry Out Laboratory Testing and Evaluation

If the HIR strategy was determined to be a cost effective alternative, a field coring and laboratory testing and evaluation program would be initiated in order to evaluate the potential for recycling the existing pavement.

5. Finalize Alternative Strategies and Carry Out Life Cycle Cost Analysis

The following alternative potential strategies were finalized and the service lives and original construction costs estimated. The estimated service lives were based upon engineering judgement and knowledge of local conditions and would apply to the conditions of this project only.

Strategy	Est. Service Life (yrs.)	Est. Capital Cost ($/km)
A. 20 + 50 mm overlay	13 - 15	57 000
B. HIR (50 mm)	7 - 11	25 000
C. Mill and Inlay (50 mm)	9 - 13	34 000
D. Reprofile + 50 mm overlay	11 - 13	47 000

It is assumed that a 60 mm overlay will be required at the end of the estimated service life for all three alternative strategies. This overlay is estimated to cost $49 000/km and have a 15 year service life.

It is assumed that maintenance costs are similar for the four alternatives over the analysis period.

Based upon the service life predictions made for the various strategies, the following proposed rehabilitation schedule has been developed:

Year	**Alternative Strategy**			
0.00	**20 + 50 mm overlay**	**HIR (50 mm)**	**Mill and Inlay (50 mm)**	**Reprofile + 50 mm overlay**
9		60 mm overlay		
11			60 mm overlay	
12				60 mm overlay
14	60 mm overlay			
24		60 mm overlay		
26			60 mm overlay	
27				60 mm overlay
29	60 mm overlay			

Maintenance costs have not been included because they have been assumed to be relatively equal for all alternative strategies.

Based upon the rehabilitation schedule, and the estimated costs of the various alternative strategies, and following the life cycle cost analysis methodology outlined in Section 6, the present worth of each alternative, discounted at 4% is summarized as follows:

	Alternative Strategy							
	20 + 50 mm overlay		HIR (50 mm)		Mill and Inlay (50 mm)		Reprofile + 50 mm Overlay	
Year	**Capital**	**Discounted to year 0**	**Capital**	**Discounted to year 0**	**Capital**	**Discounted to year 0**	**Capital**	**Discounted to year 0**
0.00	57,000	57,000	25,000	25,000	34,000	34,000	47,000	47,000
9			49,000	34,427				
11					49,000	31,829		
12							49,000	30,605
14	49,000	28,296						
24			49,000	19,116				
26					49,000	17,674		
27							49,000	16,994
29	49,000	15,712						
Credit for Residual Value at Year 30	45,733	14,100	29,400	9,065	35,900	11,069	39,200	12,086
Total Present Worth		86,908		69,478		72,435		82,513

The ranking of the three alternative rehabilitation strategies, based upon life cycle cost analysis was:

	Alternative Strategy	Total Present Worth Discounted to Year 0
B	HIR (50 mm)	$69 478
C	Mill and Inlay (50 mm)	$72 435
D	Reprofile + 50 mm overlay	$82 513
A	20 mm + 50 mm overlay	$86 908

6. Finalize Design, Documentation and Report

This preliminary economic analysis would indicate that HIR is the preferred alternative based upon lowest life cycle costs. In this case, this alternative also has the lowest initial construction cost. The final selection would be confirmed based upon a field coring and laboratory evaluation program to assess the potential to recycle the existing pavement materials.

There may be other factors that may need to be considered before an optimum strategy is selected. For example, there may be unquantifiable benefits associated with Alternative Strategies B (HIR) and C (Mill and Inlay) (eg. no reduction in pavement shoulder widths, no reduction in overpass clearances, a reduction in aggregate and asphalt material requirements) that need to be considered.

This example demonstrates that the calculated present worth of any alternative is very sensitive to the estimate of its service life.

Mix type selection should be carried out in accordance with AT&U criteria which are currently under review.

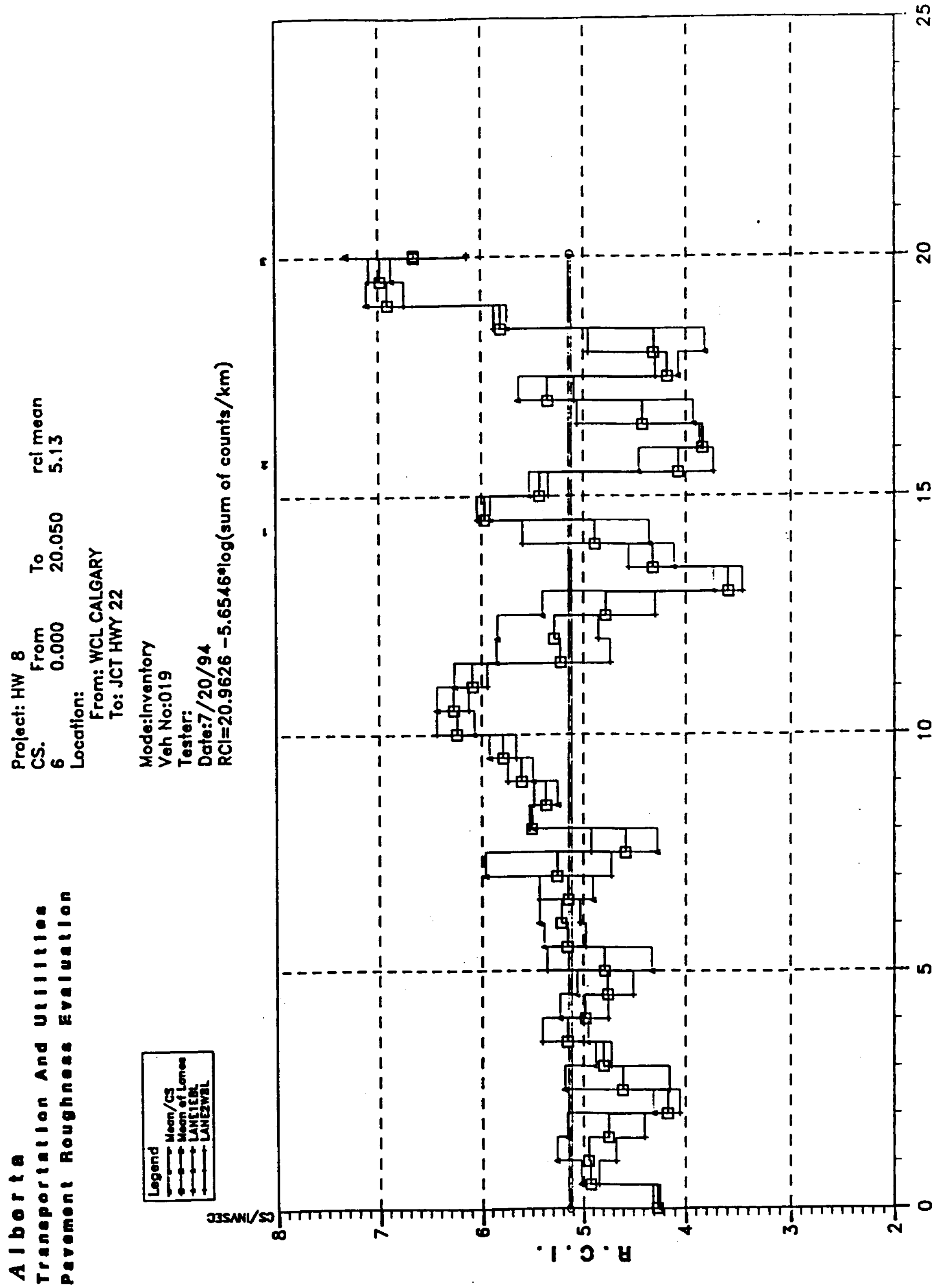

Figure 10.1 HW 8:06 - RCI Plot

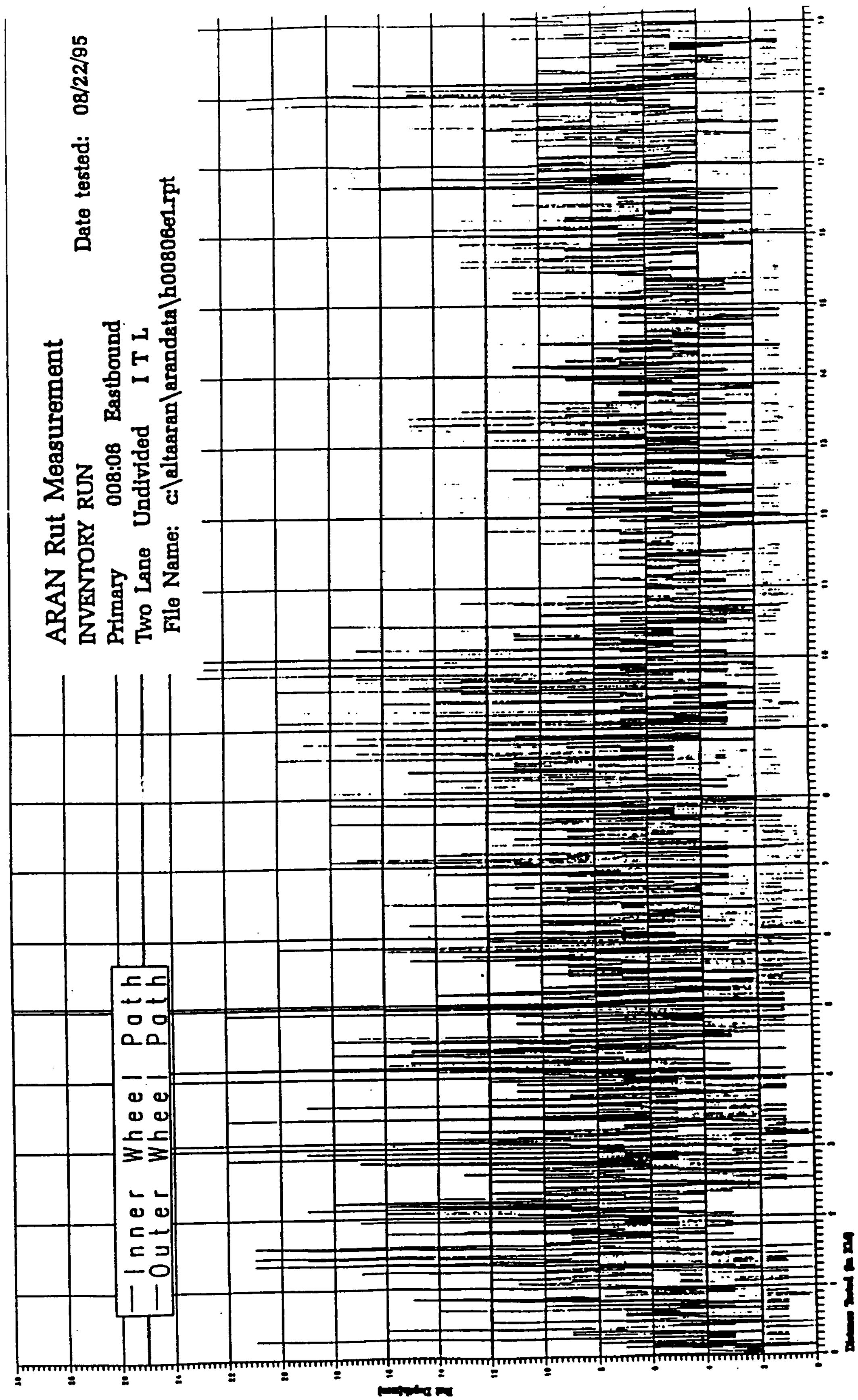

Figure 10.2 HW 8:06 - Rut Plot

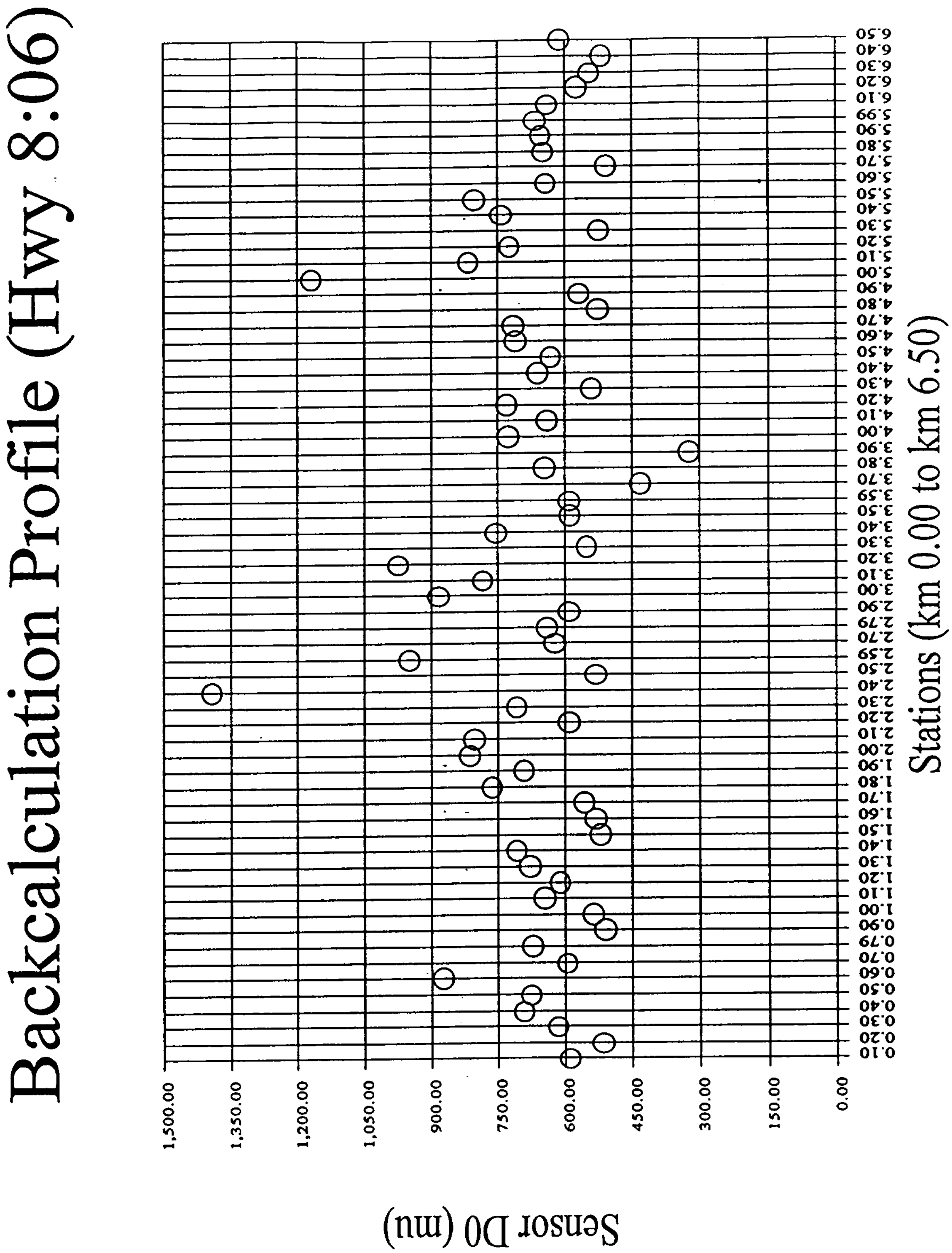

Figure 10.3 HW 8:06 - Sensor D_o Profile

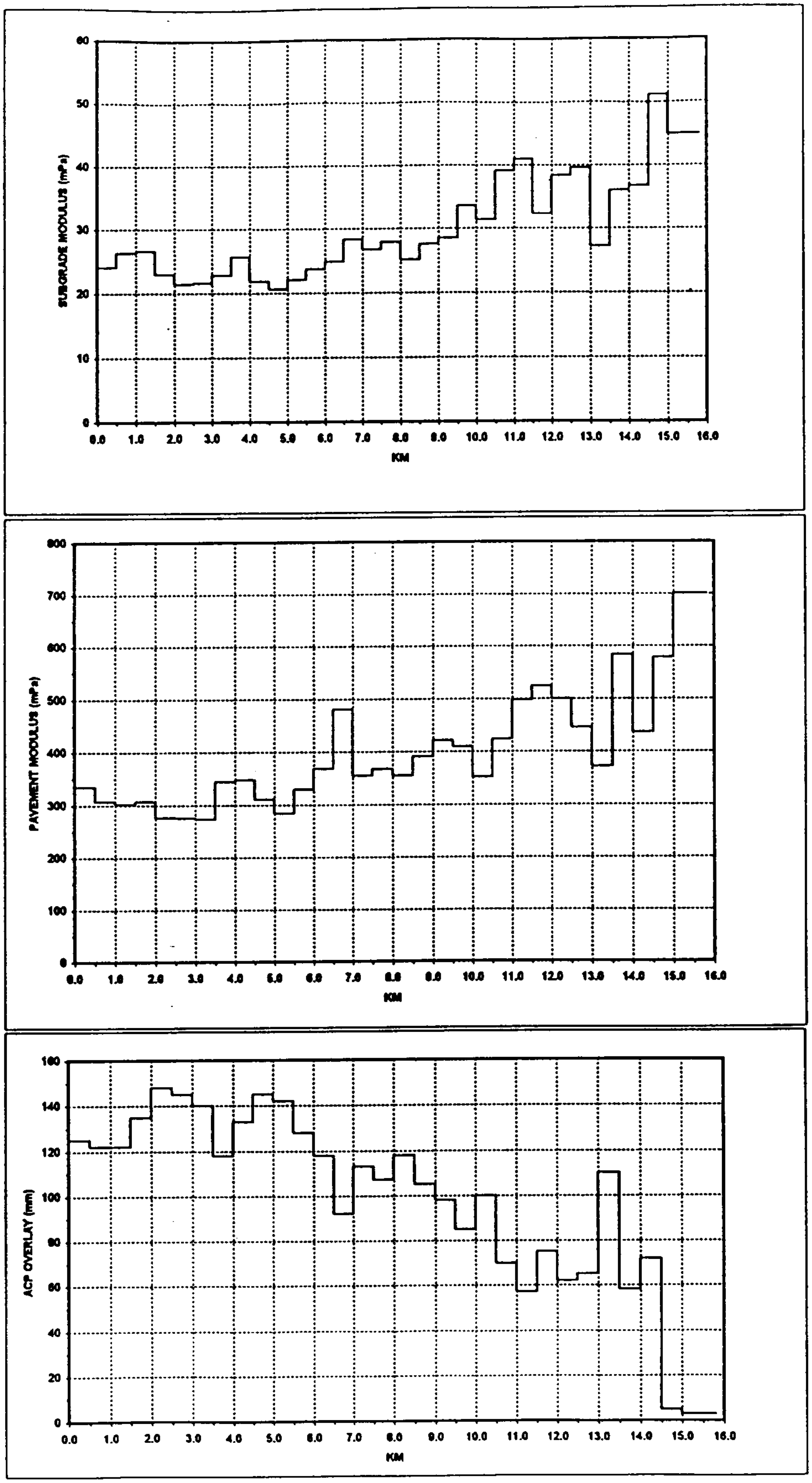

Figure 10.4 HW 8:06 - Profile Plots

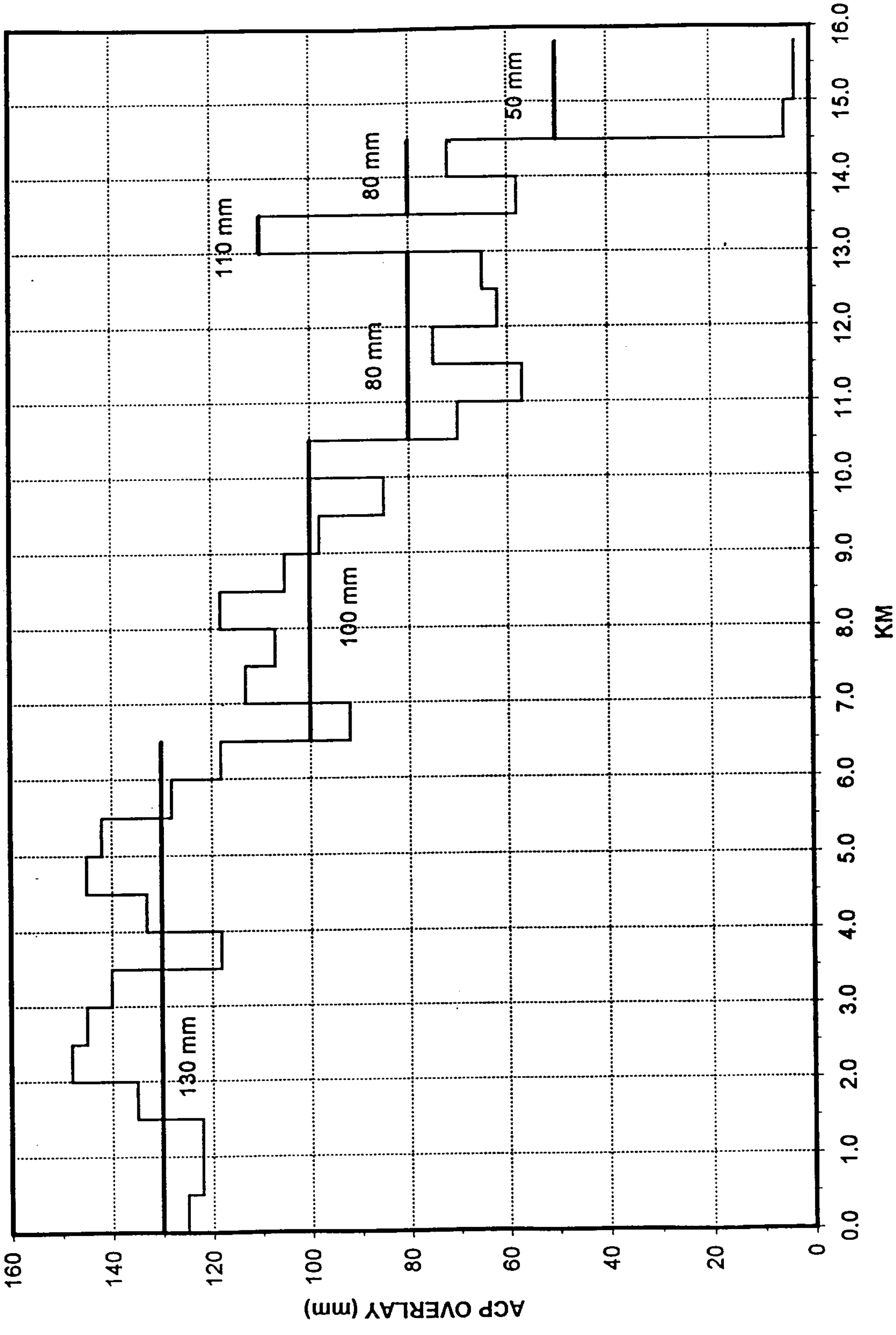

Figure 10.5 HW 8:06 - Recommended Structural Overlay Design

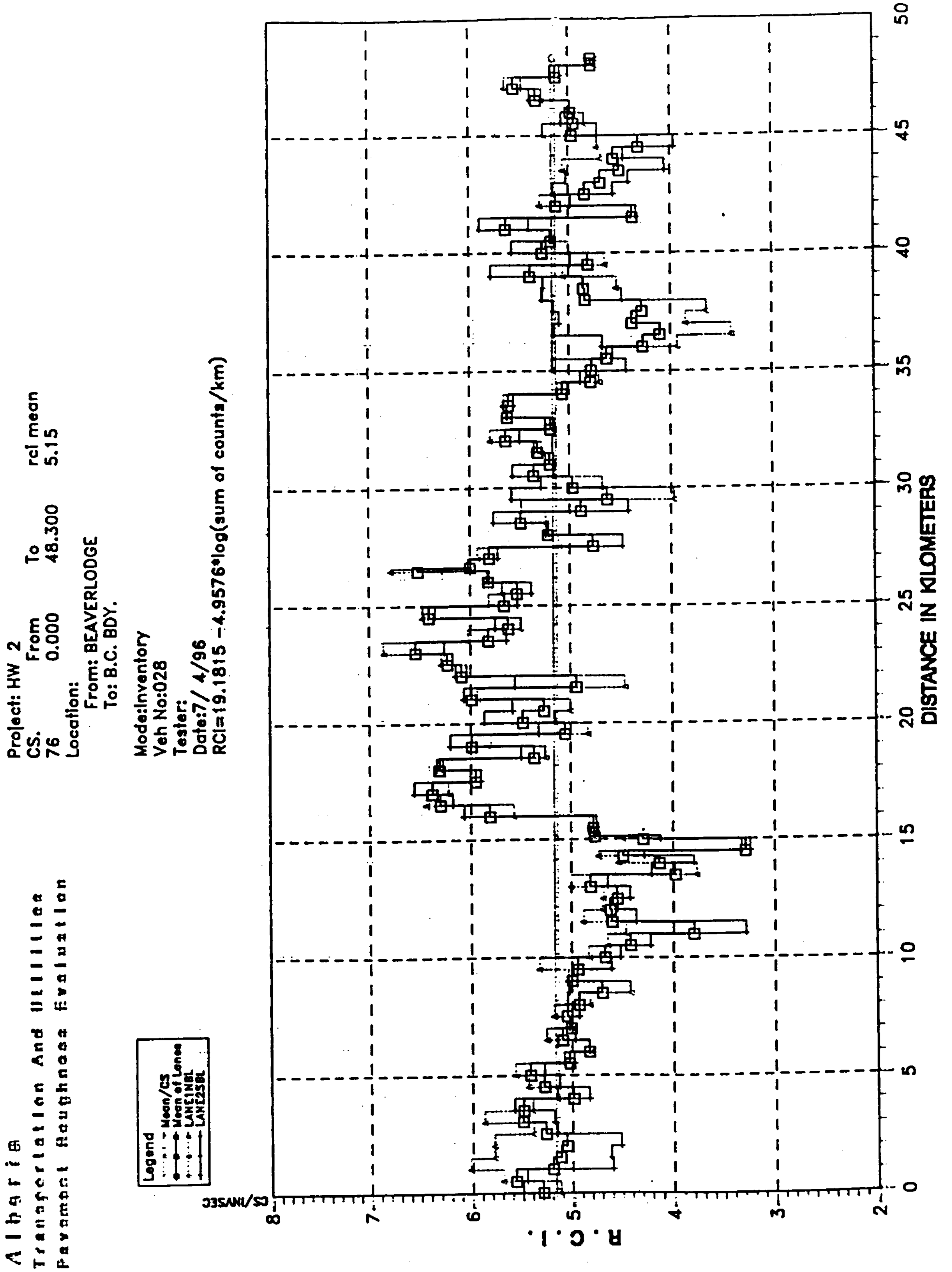

Figure 10.6 HW 2:76 - RCI Plot

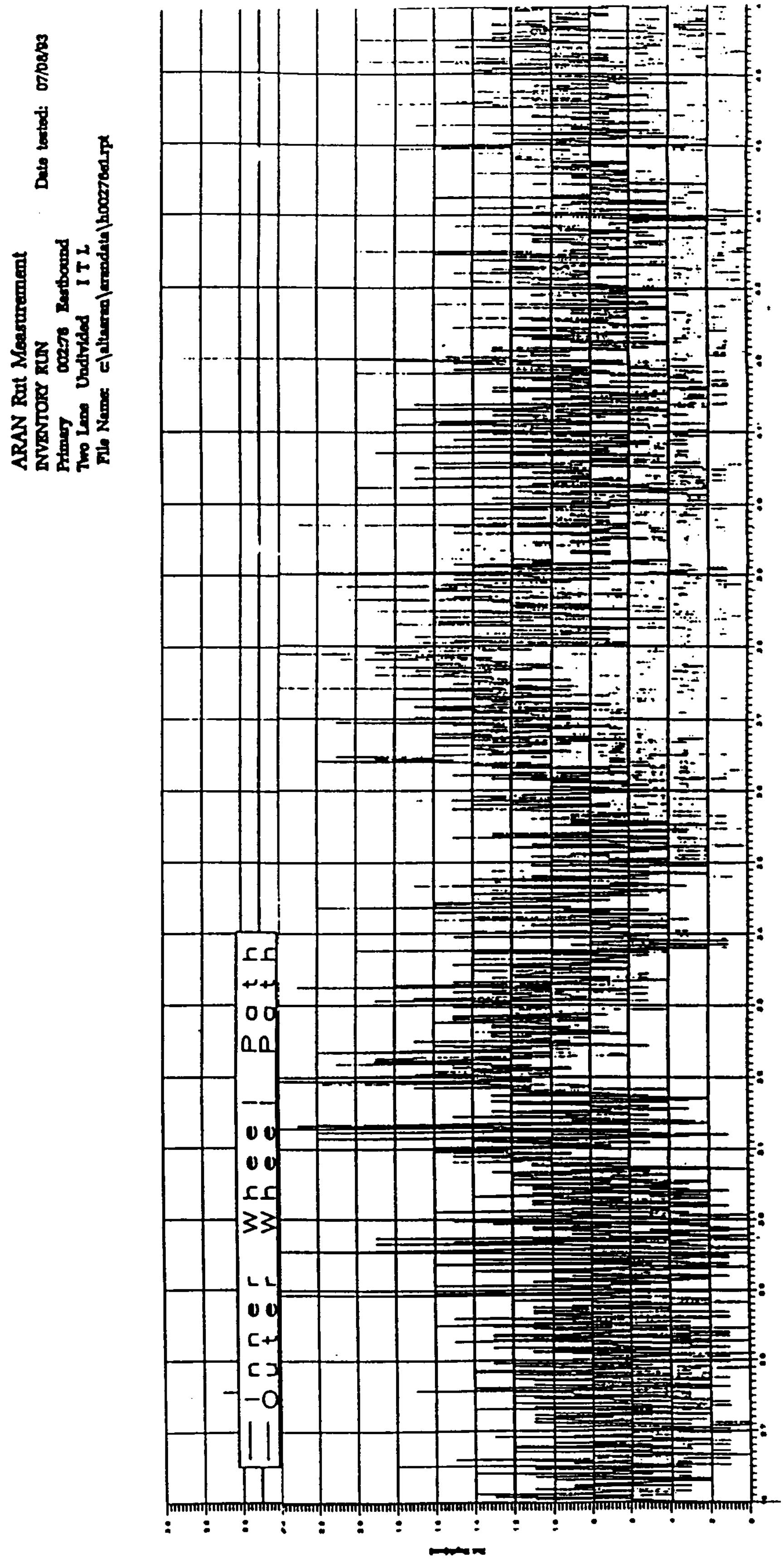

Figure 10.7 HW 2:76 - Rut Plot

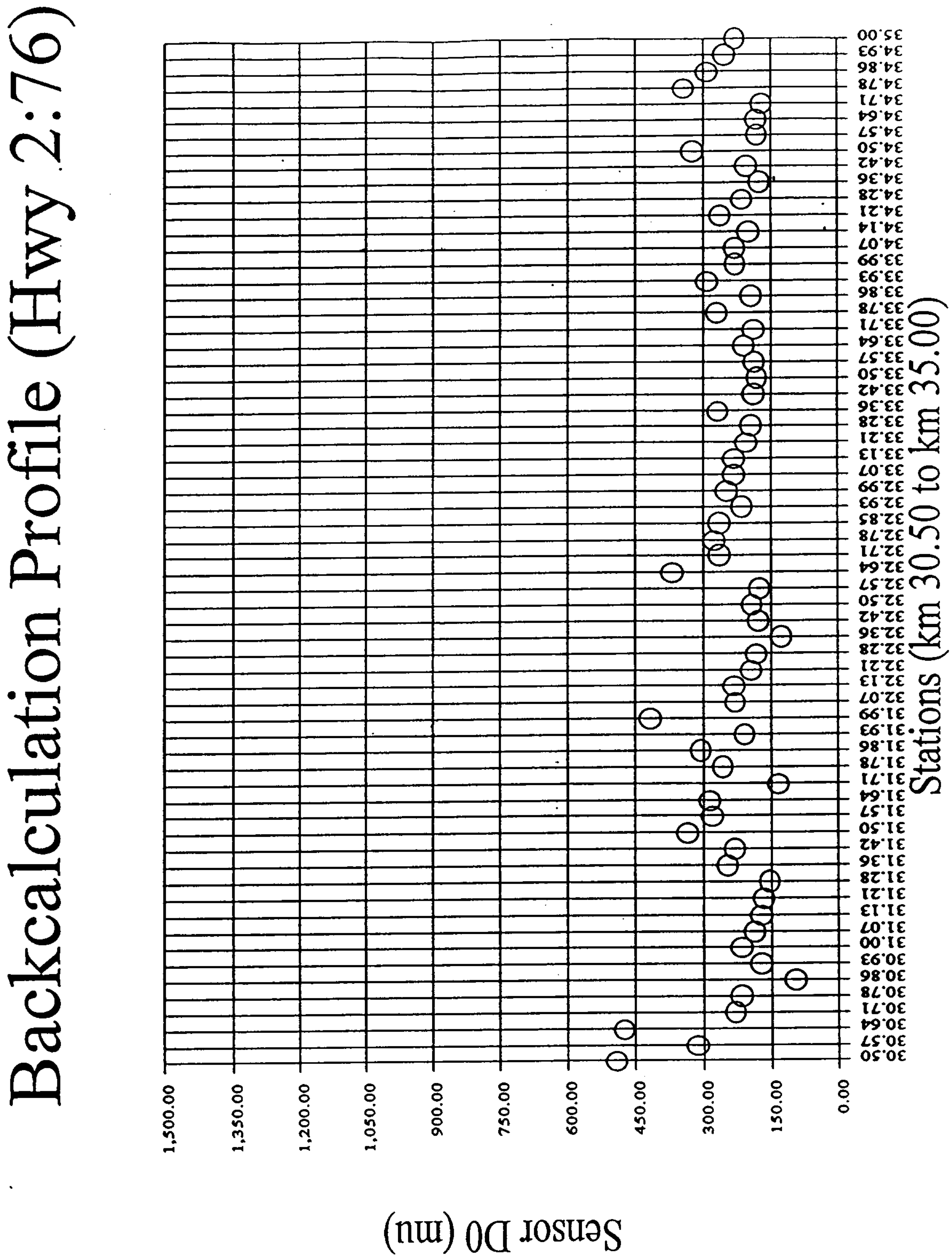

Figure 10.8 HW 2:76 - Sensor D_o Profile

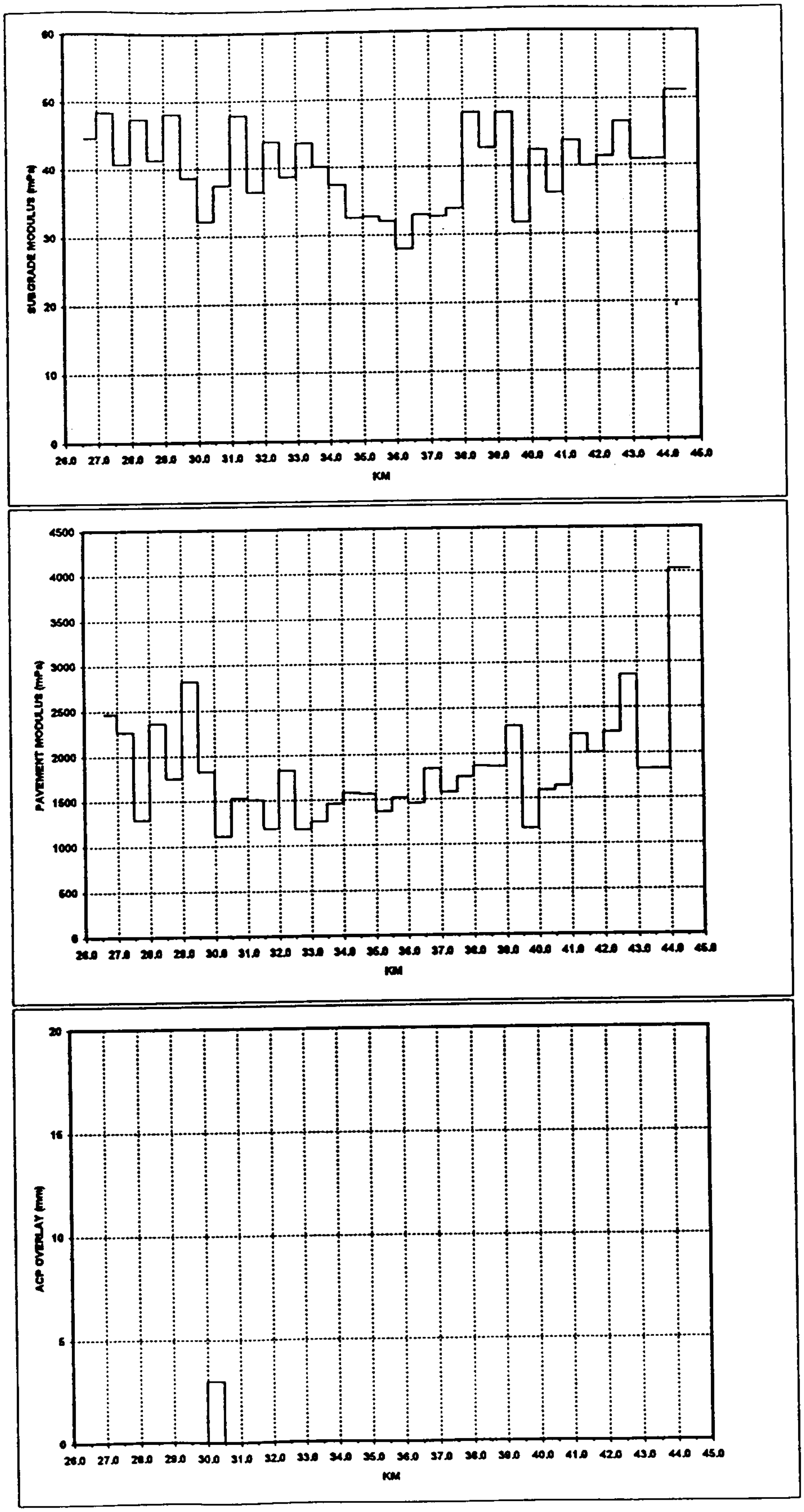

Figure 10.9 HW 2:76 - Profile Plots

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Note: AT&U references are updated periodically. The consultant shall ensure that the most recent version is used.

APPENDIX A

GLOSSARY OF ACRONYMS AND TERMS

GLOSSARY OF ACRONYMS AND TERMS

ªPSI	Design Serviceability Loss (8.4.1)
3R/4R	Resurfacing, Restoration, Rehabilitation/Resurfacing, Restoration, Rehabilitation, Reconstruction (Section 7)
AADT	Average Annual Daily Traffic (5.2.1)
AAPT	Association of Asphalt Paving Technologists
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACP	Asphalt Concrete Pavement (2.1)
AI	Asphalt Institute
a_i	Structural layer coefficients (8.4.3, 9.2.2, 10.2.2)
Analysis Period	Analysis Period (6.1)
ARAN	Automatic Road Analyzer (4.2.2)
ARC	Alberta Research Council
ASBC	Asphalt Stabilized Base Course (2.1)
Asphalt Mix Type	AT&U designations for asphalt mix types (2.4)
ASTM	American Society for Testing and Materials
C	Correction Factor applied to backcalculated subgrade Modulus (2.2.3)
C-SHRP	Canadian Strategic Highway Research Program
CBR	California Bearing Ratio
CSBC	Cement Stabilized Base Course (2.1)
CTAA	Canadian Technical Asphalt Association
DARWin	Design, Analysis and Rehabilitation for Windows (2.2.2, 4.2.1, 9.2.3, 10.2.3)
Design Period	Design Period (6.1)
Design ESALs	Cumulative ESALs in the design lane for the design period (5.1)
Design M_R	Same as Effective Roadbed Resilient Modulus (2.2.3)

Effective Roadbed Resilient Modulus	Same as Design M_R (2.2.3)
ELMOD	Evaluation of Layer Moduli and Overlay Design Computer Program (2.2.3)
ESAL	Equivalent Single Axle Load (5.1)
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
C_{REG}	Regional Correction Factor applied to backcalculated subgrade modulus (2.2.3)
GSBC	Granular Subbase Course (2.3)
HIR	Hot In-place Recycling
LCC	Life Cycle Costs (6.1)
m_i	Drainage coefficients (8.4.3)
MMS	AT&U Maintenance Management System (4.1)
M_R	Resilient Modulus (2.2.2)
NCHRP	National Cooperative Highway Research Program
Pavement	Pavement (1.1)
PI	Plasticity Index
PMS	AT&U Pavement Management System (4.1)
p_o	Initial serviceability (8.4.3, 9.2.2, 10.2.2)
PQI	Pavement Quality Index (4.1, 4.2.5)
p_t	Terminal serviceability (8.4.3, 9.2.2, 10.2.2)
PW	Present Worth 6.2)
QA	Quality Assurance
QC	Quality Control
R	Reliability (8.4.3, 9.2.2, 10.2.2)
RCI	Riding Comfort Index (4.2.2)
SAI	Structural Adequacy Index (4.2.5)
Service Life	Service Life (6.1)
SHRP	Strategic Highway Research Program

Skid Number	Skid Number (4.2.4)
SN	Design Structural Number for new construction (8.4.1)
SN_{eff}	Effective Structural Number (9.2.1, 10.2.1)
SN_f	Structural Number for Future Traffic (9.2.1, 10.2.1)
S_O	Overall Standard Deviation (8.4.3, 9.2.2, 10.2.2)
SUPERPAVE	Superior Performing Asphalt Pavements
SUT	Single Unit Truck (5.2.1)
TAC	Transportation Association of Canada
TCS	Traffic Control Section (5.2.1)
TGF	Traffic Growth Factor (5.2.3)
TRB	Transportation Research Board
TTC	Tractor Trailer Combination (5.2.1)
u_f	Relative damage factor (2.2.3)
VCI	Visual Condition Index (4.2.3)

APPENDIX B

CONVERSION FACTORS

CONVERSION FACTORS

To convert from	To	Multiply by
Fahrenheit (temperature)	Celsius	$t_c = (t_f - 32)/1.8$
foot	metre	0.30480
foot^2 (ft^2)	metre ^{2}	0.092903
inch	millimetre (mm)	25.4
mil	µm	25.4
mile	kilometre	1.609344
pound-force	newton (N)	4.448222
pound-force/inch^2	kilopascal (kPa)	6.894757
tonne (metric)	kilogram (kg)	1000
pound (mass)	kilogram	0.4535924
Celsius (temperature)	Fahrenheit	$t_f = (t_c * 1.8) + 32$
metre	foot	3.280840
metre^2 (m^2)	foot^2 (ft^2)	10.763915
millimetre (mm)	inch	0.03937
µm	mil	0.03937
kilometre	mile	0.6213711
Newton (N)	pound-force	0.2248089
kilopascal (kPa)	pound-force/inch^2	0.1450377
kilogram	tonne (metric)	0.001
kilogram(kg)	pound (mass)	2.2046

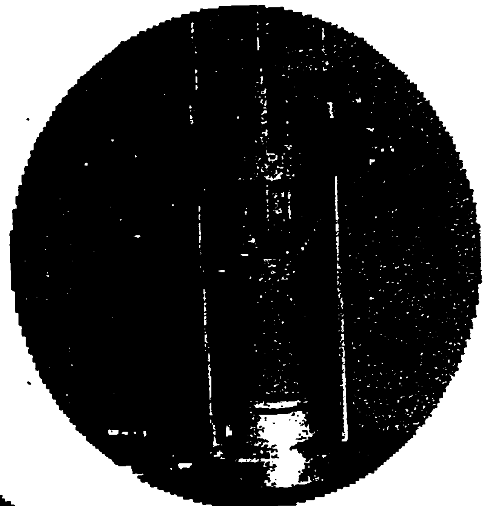
APPENDIX C

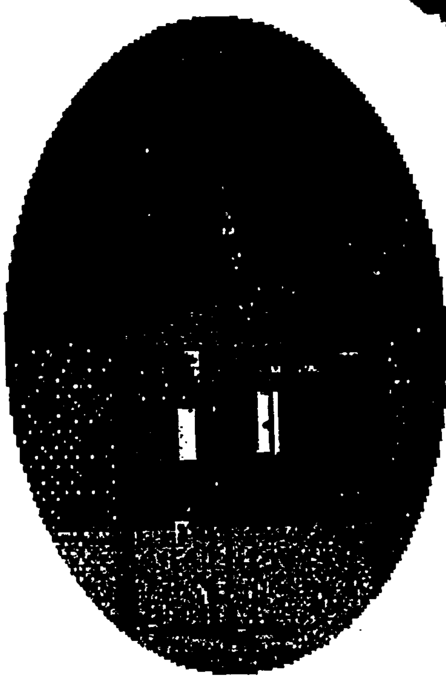
EXECUTIVE SUMMARY

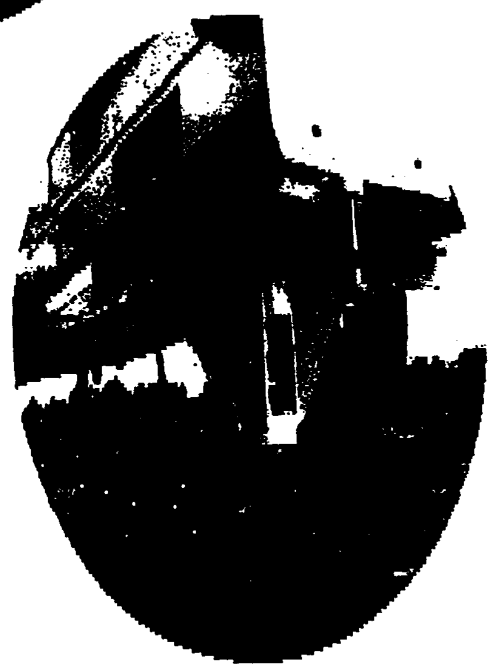
ABackcalculation of Pavement Layered Moduli In Support of
The 1993 AASHTO Guide For The Design of Pavement Structures@

U.S. Department
of Transportation
Federal Highway
Administration

Analyses Relating to Pavement Material Characterizations and Their Effects on Pavement Performance







DTFH61-95-C-00029 March 1997

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle **Analysis Relating to Pavement Material Characterizations and Their Effects on Pavement Performance**		5. Report Date March 1997 6. Performing Organization Code
7. Author(s) **Harold Von Quintus and Brian Killingsworth**		8. Performing Organization Report No. **BR95-01/J**
9. Performing Organization Name and Address **Brent Rauhut Engineering Inc. 8240 Mopac, Suite 220 Austin, Texas 78759**		10. Work Unit No. (TRAIS) 11. Contract or Grant No. **DTFH61-95-C-00029**
12. Sponsoring Agency Name and Address **U.S. Department of Transportation Federal Highway Administration 400 Seventh Street, SW Washington, DC 20590**		13. Type of Report and Period Covered Draft Report 4/95 - 8/96 14. Sponsoring Agency Code HCP30-C
15. Supplementary Notes **This study was performed in cooperation with the U.S. Department of Transportation, Federal Highway Administration.**		
16. Abstract		

This report presents the analysis conducted on relating pavement performance or response measures and design considerations to specific pavement layers utilizing data contained in the Long Term Pavement Performance Program National Information Management System. The goal of this research activity was to enhance implementation and use of the 1993 AASHTO Design Guide through improved materials characterizations Specifically, the focus of this research activity was to identify the differences that exist between laboratory measured and backcalculated resilient moduli; determine the applicability of the C-Values, drainage coefficients, and relative damage factors that are included in the Design Guide; and provide procedures to adequately consider the seasonal variation of material properties as related to flexible pavement designs. Based on these results, design pamphlets have been prepared in support of the AASHTO Design Guide. These design pamphlets are documented and included in other reports. The results reported here form the basis and background for those design pamphlets.

17. Key Words **Pavement Performance, Backcalculation, LTPP Database, Resilient Modulus, Deflection, AASHTO Guide, Drainage, Subgrade, Stabilization**		18. Distribution Statement	
19. Security Classif. (of this report) **Unclassified**	20. Security Classif. (of this page) **Unclassified**	21. Number of Pages	22. Price

DS-TL-1242 (Rev. 6/76) Facsimile

PAVEMENT MATERIAL CHARACTERIZATIONS AND PAVEMENT PERFORMANCE

EXECUTIVE SUMMARY

There have been major efforts in the last several decades towards advancing pavement technology in the areas of structural design and materials characterization. Unfortunately, much of this research has yet to find its way into routine use by practicing engineers. A classic example of this reluctance to use relatively new technology is the AASHTO Design Guide. Fewer than half of the State Highway Agencies (SHAs) have adopted or use the Guide for routine pavement design some 10 years after its initial publication in 1986.

One answer for this limited use and acceptance may be due to the increased complexity over the relatively small and simple 1972 AASHTO "Blue" book. Another answer may be related to the difficulty in using and understanding (or not having confidence in) some of these new inputs, such as resilient modulus, reliability and drainage coefficients. For example, resilient modulus testing for pavement design was available and being used more than 10 years before publication of the 1986 AASHTO Design Guide. However, most SHAs still do not actually use the resilient modulus test to determine the design modulus of the roadbed soil, but rather estimate this value using correlations that are simple, but highly inaccurate.

Another major research effort in the pavement performance area was initiated in 1987 through the creation of the Strategic Highway Research Program (SHRP), and was entitled the Long Term Pavement Performance (LTPP) program. This program set up hundreds of experimental test sites across the U.S. and initiated the data collection effort for each site. One of the goals of the LTPP program was to create an extensive, but well-structured, database that would help confirm and validate these new technologies and design procedures, but more importantly, build confidence in their use. This LTPP database, referred to as the National Information Management System (NIMS), was a key product of SHRP in which all of the data are being stored and updated on a continual basis for use by the pavement industry. The FHWA has assumed responsibility for managing this database and to continue with the data collection and monitoring effort to ensure that there are sufficient data to support the continued development and implementation of new technologies.

To begin capitalizing on this massive data collection effort, FHWA initiated several data analysis contracts, one of which was in the materials characterization area for pavement design. Specifically, the overall goal of this contract, entitled "Analyses Relating to Pavement Material Characterizations and Their Effects on Pavement Performance" (Contract No. DTFH61-95-C-00029) was to use the LTPP database to enhance implementation of the 1993 AASHTO Design Guide through improved material characterization. This contract has resulted in four reports and three design pamphlets in support of the 1993 AASHTO Design Guide. The reports and design pamphlets are listed below:

Reports:

1. "Analysis Relating to Pavement Material Characterizations and Their Effects on Pavement Performance".
2. "Backcalculation of Layer Moduli of SHRP-LTPP General Pavement Study Sites".
3. "Evaluation of IRI Decreases with Time in the LTPP Southern Region".
4. "LTPP FWD Deflection-Time Data for Characterizing Pavement Structures and Pavement Response".

Design Pamphlets:

1. "Backcalculation of Pavement Layer Moduli in Support of the 1993 AASHTO Guide for the Design of Pavement Structures".
2. "Determination of Design Subgrade Moduli in Support of the 1993 AASHTO Guide for Design of Pavement Structures".
3. "Determination of Layered Elastic Moduli in Support of the 1993 AASHTO Guide for the Design of Pavement Structures".

In summary, the reports noted above provide the background and a discussion of the work conducted, while the design pamphlets are intended to support the determination of selected design inputs that are required by the AASHTO Design Guide. Key findings from the overall study are listed below:

Backcalculated Layer Moduli for Structural Design:

1. Backcalculation of layer moduli using elastic layer theory can be used to determine the resilient modulus of different pavement layers. However, the insitu moduli must be adjusted to represent or equal the laboratory measured values for those design procedures developed with laboratory measured moduli (which includes the AASHTO Design Guide). Layer moduli backcalculated with different programs should not be used interchangeably, because of the differences found between the various backcalculation programs. The adjustments converting field calculated moduli to laboratory measured values (as reported in this study) are only applicable to the "MODULUS" and "WESDEF" programs. Both of these programs use a linear elastic layered response model to calculate a deflection basin.

2. Elastic layer theory is not applicable to all types of measured deflection basins. Some deflection basins are considered or identified as "problem" basins, because they do not fit the "standard" deflection basin profile calculated with elastic layer theory. Although layer moduli can be determined from problem deflection

basins, the elastic moduli may not be representative of the actual insitu material.

3. Backcalculated layer moduli are almost always greater than the laboratory measured values at comparable stress states and/or temperatures.

4. There is no unique solution for a specific deflection basin. The error term should be as low as possible, but less than a value of 2½ percent error per sensor when using the backcalculated moduli for design.

Subgrade Characterization for Structural Design:

1. Determination of the design subgrade modulus utilizing the relative damage factors based on the AASHTO serviceability criteria, tends to be greater than the design subgrade modulus calculated using damage factors based on minimizing the subgrade vertical compressive strain at the top of the subgrade. All pavement designs generated with the AASHTO Design Guide should be checked using the response criteria of minimizing subgrade vertical compressive strains, especially for lower volume roadways.

2. Correlations should not be used to estimate the design resilient modulus for pavement structural design for high volume roadways. The design resilient modulus should be determined from laboratory resilient modulus tests, or backcalculated from deflection basins. The possibility of large errors is simply too high when using gross correlations between physical properties or strength values (such as CBR) and resilient modulus.

Drainage Considerations:

1. The AASHTO drainage coefficients are not recommended for use in structural design. Instead, the design process should account for a reduction in the resilient modulus to account for saturated conditions through the calculation of a design modulus using relative damage factors for all unbound-moisture sensitive materials.

2. The use of positive drainage features in both asphalt concrete and portland cement concrete surfaced pavements was not qualified through the use of the LTPP database. Some of the problems in identifying the potential benefit of subsurface drainage features may be related to the assumption that the positive drainage system is

functioning properly. As such, it is recommended that those sites with positive drainage features (i.e., edge drain systems) be inspected by video inspection techniques to confirm that these drainage features are, in fact, functioning.

Determination of Design Layer Moduli:

1. Seasonal variations of layer moduli (estimated through moisture and/or temperature differences between the seasons) must be considered in determining the design modulus of different materials so that the structural layer coefficients can be determined for use with the AASHTO Design Guide. The design modulus can be determined using a damage concept similar to that used in determination of an effective resilient modulus of the roadbed soil. More specifically, structural designs based on a serviceability criteria should be checked using other pavement response criteria (i.e., asphalt concrete tensile strains, subgrade vertical compressive strains, layer modulus ratios, etc.).

Specific discussion on each of these key findings are given in the reports listed above. Application and use of these findings are expected to provide improved designs and a more realistic estimate of pavement behavior and performance. In addition, implementation of these studies should provide a more consistent use of the design parameters. These studies also attempt to merge and compare designs based on new technology using pavement response criteria that are required for mechanistic-empirical procedures and those using the serviceability concept.