THE UNIVERSITY OF ALBERTA

A STUDY OF PERMANENT DEFORMATION CHARACTERISTICS OF ASPHALT CONCRETE PAVEMENTS IN ALBERTA

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled A STUDY OF PERMANENT DEFORMATION CHARACTERISTICS OF ASPHALT CONCRETE PAVEMENTS IN ALBERTA submitted by CHARLES THOMAS MCMILLAN in partial fulfilment of the requirements for the degree of MASTER OF SCIENCE.

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DEDICATION

This work is dedicated

to L. Curtis for his love and innocence,

to Corinne for her love, understanding and encouragement,

and to my parents for their love and faith.

ABSTRACT

The occurrence of permanent deformation in the wheelpaths of asphalt concrete pavements reduces the overall rideability of the roadway and can create unsafe conditions for the motoring public. This permanent deformation, or rutting, is becoming a greater concern to highway agencies as truck tire loadings and traffic volumes increase and the occurrence of rutting becomes more of a widespread problem. This increased concern over the occurrence of pavement rutting has lead to the need to quantify the extent that the binder affects the permanent deformation characteristics of an asphalt concrete.

The objective of this study is to examine the role of the asphalt binder properties in terms of the permanent deformation of asphalt concrete. Binders investigated include conventional petroleum asphalt grades specific to Alberta, recycled asphalt and two polymer modified asphalts.

This study was divided into two main portions, each considered as a separate stage. One stage looked at in-place field performance of asphalt concrete and the other involved a laboratory testing and evaluation program. Both stages of the study examined the permanent deformation of the asphalt concretes and developed models to describe the observed behaviour in terms of the material properties of the mix, specifically of the binder.

The main finding of this study is that the difference in permanent deformation of an asphalt concrete pavement constructed with 150-200A versus 200-300A asphalt grade can be expected to be up to 30%. The use of polymer modified asphalt

binders were found to reduce the observed permanent deformation significantly, in the order of 50% when tested at 25°C and greater differences when tested at higher temperatures. As well the level of rutting of the Alberta highway system was found to be serious only on the higher traffic corridors such as Highways 1, 2 and 16.

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CHAPTER I
INTRODUCTION

1.0 BACKGROUND

An asphalt pavement distress mode which is gaining increasing attention by highway agencies world wide is permanent deformation in the wheel paths. This wheelpath channelization, or rutting, is becoming increasing noticeable in western Canada as traffic volumes and loadings both increase.

The occurrence of pavement rutting reduces the overall rideability of the pavement. The problem is amplified during wet weather when water depths in the wheelpaths can be sufficient to cause hydroplaning, especially at highway speeds.

Past research work has been relatively extensive, examining theoretical properties in order to predict field performance, correlating theoretical work with field observations and investigating field performance relative to conventional mix properties. Past works have examined the role of each of the pavement structure's constituent materials and physical properties to give an insight into the deformation mechanism.

Rutting is the result of the accumulation of small deformations in either the base or subgrade materials or by the instability of the asphalt concrete mix itself or a combination of the two. The problem of base deformation results from an underdesigned pavement transferring excessive loading to the support materials and can be considered a structural design problem. Rutting due to the instability of the asphalt concrete is a problem that must be addressed at the mix design stage, by considering inherent material properties, the physical properties of the mix itself and the magnitude and number of loadings to which the pavement is to be subjected.

It is important to understand the influence of the various mix components and of the physical properties of the mix so that proper engineering design can be carried out to minimize the risk of premature rehabilitation needs, due to excessive rutting. The problem of rutting is further highlighted when an otherwise sound pavement structure requires rehabilitation because of the permanent deformation caused by the inability of the asphalt concrete to resist the applied loading.

Within Alberta Transportation and Utilities, mixture design follows the Marshall method of mix design as described by the Asphalt Institute's Mix Design for Asphalt Concrete and Other Hot Mix types, Manual series 2. While these procedures are accepted industry practice, and the Alberta Transportation and Utilities Laboratory has demonstrated competence in performing the design procedure, recent studies have shown that there is no apparent relationship between Marshall stability and the ultimate resistance of the mix to permanent deformation[1].

It is because of this inability of standard test procedures or material characterization methods to provide sufficient guidelines that further research is required in order to provide highway engineers with the tools for improving pavement performance in this area.

1.1 Research Objectives and Scope

The objective of this research is to examine the role of the asphalt binder properties within the overall mechanism of permanent deformation of asphalt concrete pavements. Binders investigated include conventional petroleum asphalts, polymer modified asphalts, and recycled asphalt binders.

The main focus of the research is the binder influence on deformation mechanics; however all contributing factors have been analyzed. These factors include aggregate characteristics, in-place mix characteristics, mix design characteristics and traffic and climatic conditions.

Specifically stated, the objectives of this work are as follows:

1. To determine the influence of the binder rheology on the permanent deformation characteristics of asphalt concrete mixes.
2. To determine the physical characteristics of recycled asphalt mixes in terms of their resistance to permanent deformation compared to virgin asphalt mixes with comparable binder rheology.
3. To determine the effect of polymer modification to asphalt in terms of permanent deformation characteristics of asphalt concrete pavements.
4. To determine the influence of mix design and as-built parameters on the permanent deformation of in-place asphalt pavements.
5. To examine the influence of temperature on deformation behavior, in terms of asphalt stiffness.

The scope of this project includes a literature review, the review of forty selected sites within the Alberta Transportation and Utilities roadway network and an intensive laboratory testing program aimed at meeting the outlined objectives. The projects examined represent a complete cross section of structure and material types in various climatic settings throughout the Province. The study was limited for the most part to those pavement structures which had never been overlayed in order to reduce the variables involved.

1.2 Research Approach

The research conducted for this work was divided into two distinct stages. The first examined in-place performance and compared it to known pavement characteristics; the second related measured deformations from laboratory conducted repeated load triaxial testing to the material characteristics of the asphalt concrete mixes.

A pragmatic approach was taken to the work to facilitate the implementation of the findings. As a part of the first stage of the study pavement slabs were cut from a section of rutted pavement to determine where the rutting occurred within the pavement structure. The examination of of this project showed rutting to be predominantly in the asphalt concrete layers. Appendix A presents the information for this initial work.

Following the work done with the pavement slab sections, forty project sites were selected and historical design data and construction information collected. The projects were inspected and measurements of rut depths obtained. As well, core samples of the existing pavement were obtained at twenty eight of the sites in order to determine current mix characteristics. Statistical analyses were conducted on the assembled data in an attempt to determine those factors which make a significant contribution to the observed rutting.

The second stage of the investigation, the repeated load triaxial testing, involved the testing of over 200 individual samples to evaluate the influence of the binder and mix properties on the deformation characteristics of the mixes. These samples represented twenty four mixes. This work included samples formed in the Laboratory and in the field. This allowed the comparison of laboratory and field prepared mixes on the behaviour of the asphalt concrete.

1.3 Thesis Organization

This thesis is divided into eight chapters and five appendices.

The first chapter presents the background to this research and the scope and objectives of this work. As well a brief overview of the research approach is presented. Appendix A presents the information found from the pavement slabs taken as a preliminary part of the investigation.

Chapter II discusses those factors affecting rutting as determined through a review of the literature.

Chapter III describes the field work conducted for stage I of the investigation. The project selection and collection of field data and historical data is presented in this chapter.

Chapter IV presents the results and analysis for stage I. This includes the statistical analysis and model development for the field sites. Appendix B contains the field site profiles and complete materials information for eacn site.

Chapter V deals with the design of the laboratory repeated load testing program for stage II of the study. The materials characteristics and mix design information for the laboratory samples are presented in this chapter. As well, the sample preparation is discussed.

Chapter VI presents the results of the laboratory testing and the subsequent analysis of the results. Appendix C presents the laboratory testing procedure for the repeated load triaxial testing. Appendix D presents the results of the field formed test specimens and presents the analysis of the laboratory testing of these specimens.

Chapter VII discusses the results from the two stages of the study and examines the complementary nature of the results. This chapter also presents design guidelines for consideration when selecting an asphalt cement grade to minimize rutting.

In Chapter VIII, conclusions are presented and recommendations dealing with future work and implementation of the findings are made.

Appendix E presents a brief overview of the FOCUS data base program used for the data manipulation and statistical calculations.

CHAPTER II

SOME APPROACHES TO PREDICTING PERMANENT DEFORMATION

2.0 INTRODUCTION

The problem of permanent deformation in flexible asphalt concrete pavements has been examined from numerous perspectives over the years. The recognition of the loss of rideability and safety which accompanies rutting has prompted concern and the need to incorporate such considerations in all design methodologies.

Rutting reduces the serviceability of the road and can result in the need for rehabilitation. As well, the occurrence of rutting presents safety concerns because of the collection of water during wet weather which can cause hydroplaning and because of roughness during lane changes. Lister[2] has suggested that rutting in excess of 20mm constitutes the failure of the asphalt concrete pavement.

Some of the first publications addressing the rutting problem discuss the limiting strain approach, which remains as an inherent part of current structural design practice. This approach considers the vertical strains imposed on the subgrade and limits them by the structural design[3]. The purpose of this type of limiting strain design is to prevent the development of rutting due to consolidation of base materials. Rutting of this type is not a focus of this research, instead only the actual deformations within the asphalt concrete are being considered.

Approaches to predict permanent deformation within the asphalt concrete can be based on theoretical behaviour, empirical data, or a combination of both. While this research has taken a more empirical approach, a review of both theoretical and empirical methods is discussed briefly in this section in order to examine such considerations.

2.1 Material and Mix Parameters

The two materials which make up the asphalt concrete mix are the aggregate and the asphalt binder. Aggregate gradation, texture and angularity and the binder stiffness are the material properties that would be expected to have the greatest effects on the resistance to permanent deformation. The stiffness of the asphalt mix is a measure which gives an indication of the resistance to permanent deformation and accounts for the the combined behaviour of the binder aggregate matrix; the stiffness value is dependant upon temperature and the duration of the loading because of the visco-elastic behaviour of the asphalt binder.

Stiffness of an asphalt concrete can be measured directly, as for most engineering materials. An estimate of the mix stiffness can also be made by considering the stiffness of the asphalt binder in conjunction with specific mix properties. Van der Poel presented nomographs for determining binder stiffness based on rheological properties of the binder. Modifications to these nomographs by McLeod[4] allow their use knowing only the asphalt's penetration at 25°C and either the absolute viscosity at 60°C. or the kinematic viscosity at 135°C. Because an asphalt's stiffness is dependant on time of loading, this time factor must be considered as an input variable into the nomographs. For this investigation a loading frequency of 1/s was used as the actual loading in the laboratory testing and for consistency was also used for calculating stiffnesses for the field portion of the study.

In addition to the material characteristics, the mix also has certain characteristics which affect its rutting behavior. A recent study by Huber suggests that voids in the mineral aggregate (VMA), air voids, asphalt content, voids filled and Hveem stability all are significant mix characteristics affecting the pavement's resistance to permanent deformation. The influence of these various mix characteristics on rutting has been examined for the projects investigated as a part of this study and the results are reported in Chapter IV.

A study undertaken by EBA Engineering for the City of Lethbridge, Alberta also concludes the significance of the mix characteristics, specifically the design and compaction parameters[5]. In the Lethbridge study, use was made of Hveem stability testing, creep testing, and repeated load triaxial testing to help characterize the various mixes investigated. The results of this testing led the consultant to conclude that manufactured fines and larger topsize aggregates contribute to increased rutting resistance. Additionally it was concluded that recycled mixes and polymer modified asphalt mixes also improve the permanent deformation characteristics. The data presented also showed a reduction in permanent deformation under repeated load testing of approximately 20% between a 150-200A grade asphalt and an 85-100 grade. The harder asphalt grade was not considered viable for use because of concerns for low temperature cracking.

2.2 Loadings

When examining the permanent deformation of asphalt concrete, the problem becomes one of a materials response to an applied load. In the case of a highway pavement, this loading comes as a moving wheel load which imparts a load for a short duration. Varying vehicle sizes, axle configurations, and varying speeds all contribute to increasing the complexity of the analysis[6-8]

As with many civil engineering problems, the loading condition is often the most difficult parameter to determine. As a result, simplifying assumptions are used to allow the engineer to perform the required analysis and evaluate the pavement structure. In pavement design, the concept of the Equivalent Single Axle Load (ESAL) is used as a standard loading which represents a 80 Kn absolute load on a single axle with dual wheels. Additionally the tire contact area is usually considered circular and equal to the normal wheel load divided by the tire pressure. The contact pressure is therefore commonly assumed to be uniform and equal to the tire inflation pressure.

As the magnitude and extent of rutting appears to be on the increase, researchers have begun to re-examine these basic, simplifying assumptions which have been used in the past. The thrust of such research has looked at axle loads, tire pressures, and tire/pavement contact pressures.

Axle loadings have increased significantly over the past four decades and as a result tire designs have changed to accommodate the increased loadings. An Ontario study[9] suggests that in the order of 80% of trucks now use radial tires as opposed to bias ply tires. Additionally "super-single" tires which are less expensive and give fuel savings[10] are currently being introduced into the North American market. These newer tire types often use tire pressures in the order of 50% higher than previous conventional truck tire pressures, (ie: 825 vs 550 KPa)[11].

Work by Marshek et al[12] shows that contact pressures vary considerably throughout the contact area, dependant on tire inflation pressure and tread design. The tire pavement contact area has areas of discontinuities as a result of gaps in the tire tread which results in pressure concentrations at border areas.

Research has shown that higher pressures and higher absolute loadings result in increased pavement deterioration. Marshek[13] also incorporated the tire pressure discontinuities within the circular loading area used in the BISAR program to examine the distribution of stresses and strains through the pavement. The work suggests that tensile and shear strains increase with pressure, though the distribution of the strains also varies for different tire inflation pressures. Likewise, the tread influences the strain distribution with more uniform distributions resulting from bald tires. The compressive strains are influenced more by the tire load than the pressures.

Rutting which occurs on a highway pavement is a combination of cumulative compressive strains summed through the asphalt layer and of lateral displacement of the mix caused by shear stresses. As a result, the combined influence of the tire tread, pressure and total load all influence the rutting magnitude.

While the actual loadings may be impractical to determine for a specific pavement site, it is important to understand the influence of loading on the resulting deformation. Theoretically based calculations of permanent deformations using either elastic or viscoelastic theories can be used to illustrate the effect of variations in load and pressure.

2.3 Theoretical Methods

A pavement structure can be modelled as a layered elastic system for the purpose of determining the stress state and resulting strains[14]. The permanent deformation may be estimated based on the applied stress and the materials properties. Because the deformation will vary with depth, a number of layers must be selected for this type of analysis, and the strains for each layer calculated. The summation of each layer's strain yields the total deformation exhibited by the pavement structure. In order to determine material properties, testing such as repeated load triaxial tests or creep tests are required.

Calculations of permanent deformation can also be examined by considering the structure as a viscoelastic material. The theory of viscoelasticity accounts for the time dependant components and thus allows the direct calculation of permanent deformation.

2.4 Empirical Methods

Empirical methods using laboratory tests to help illustrate the rutting potential of various mixes have been reported by numerous researchers[15-18]. Morris[16], used repeated load triaxial testing to model actual in-place rutting of a test road with good success. The work undertaken in this study uses this type of testing in order to both quantify rutting potential of different binder grades and to examine rutting behaviour of different mix types.

The prediction of permanent deformation of asphalt concrete pavements is really a problem of determining loading conditions and material characteristics. As theoretical approaches are unable to account for all variations encountered in a field situation, it is not realistic to expect accurate predictions from these methods. In this study the approach followed resulted in the development of relationships to explain the relative magnitude of the observed deformations without any assumptions of material properties or deformation laws.

CHAPTER III
STAGE I - FIELD TEST SITES

3.0 INTRODUCTION

The field portion of this research, referred to as stage I, involved inspections of forty highway sites within the Provincial highways network. The field work was undertaken in an attempt to determine the contributions of the various characteristics of the asphalt concrete pavement to the permanent deformation of actual in-service pavements. The objectives for this stage of the investigation can be specifically stated as follows:

1. To examine actual cross-sections of in-place pavements to observe the deformation at the road surface.
2. To determine the location of the rutting within the pavement structure.
3. To determine the extent of permanent deformation distress on the Alberta Highway system.
4. To examine as-built materials characteristics, in conjunction with traffic and climate, and to relate these to in-service behaviour.
5. To examine in-place material properties, in conjunction with traffic and climate, and to relate these to in-service behaviour.

3.1 Project Selection

The approach taken to address these stated objectives attempted to satisfy statistical sampling requirements in order to allow meaningful analyses of the data. Therefore, the project selection was based on obtaining representative sites which covered a sufficient range for for each of the variables examined. Sites were selected for high and low traffic volumes, thin and thick pavement structures, hard and soft asphalt binders, and for all geographic locations representing the range of climates in the Province.

It was felt that it would be desirable to omit overlayed pavements because of the added variables/unknowns introduced by dealing with more than one pavement structure, each with different characteristics and a different loading history. Alberta Transportation and Utilities "Pavement Management System" (PMS) database was used to make the initial project selection. A computer program was written to extract those highway sections which had never been overlayed. From this initial list, projects were selected to obtain a range of material properties, pavement structures, traffic volumes and geographical areas. Once this tentative list of projects had been compiled, the historical construction data was obtained and summarized. Additional projects were omitted following a review of this historical data if it was felt that too m[illegible]ional variables were introduced, such as different aggregate sources, change[illegible]phalt supplier, or insufficient data for a given project.

The selection of projects which covered a sufficient range of the variables selected was necessary in order to allow meaningful analysis to be conducted. Forty project sites were inspected. These sites covered a significant range of the various parameters with cumulative ESAL's ranging from about 10000 to over 3.5 million, asphalt concrete thicknesses between 50 and 300mm, and asphalt binders with original penetration at 25°C from 160 to 317 (0.1 mm units). As well, the sites were selected throughout the Province, from the south, north, east and western extremes. Temperature values were then assigned to each of the project sites based on July design temperatures as presented in the National Building Code[19]. The analysis, which is presented in the following Chapter, examined the role of each of the variables and the contribution each had on the observed rutting.

From the final selection of projects, each was inspected and one or more sites selected. Each site was accurately cross-sectioned using a survey level and the ruts were measured using a 1.8m straight edge. Table III-1 presents the projects which were included as a part of the data base.

From the cross-sections obtained twenty eight sites were selected to obtain core samples for laboratory analysis of mix characteristics. Additionally, to determine where the deformation was occurring (ie: densification, shoving, or base materials) nine sites were cored more extensively to allow the profile at the bottom of the pavement layer to also be determined. The selected sites were generally cored in each wheelpath and between the wheelpaths; the nine sites had additional cores taken in the shoulder and at centreline locations.

3.2 Historical Data

3.2.1 Materials

The review of the historical construction data allowed for the characteristics of the as-built pavement structures to be determined. While it was noted that a range of material properties were sought in the project selection, it should be pointed out that the majority of the projects were constructed with an asphalt equivalent to a 200-300 penetration grade. Alberta Transportation and Utilities asphalt supply sources were fairly consistent prior to 1978, though one source changed in 1972 resulting in a higher viscosity asphalt cement.

Table III-1 shows a summary of the historical asphalt test data for the selected projects. Figure III-1 presents the rheology information graphically and illustrates the range being examined. Appendix B contains the detailed historical data utilized for the analysis.

Aggregates used for asphalt pavements throughout the province have generally been well graded gravels from glaciofluvial deposits. Some older projects utilized aggregate topsizes up to 25mm, though 16mm and 12.5mm topsizes are the norm. Starting in about 1982 natural blend sands were used to open up the grading band, increasing the VMA and allowing more asphalt cement to be used, addressing the concern of durability. Figure III-2 shows the band of gradations represented by the designs for the forty sites.

Alberta Transportation and Utilities mix designs follow the Marshall Method of mix design. While some changes in testing procedures over the years has resulted in some differences in properties measured, similar properties were consistently sought. Though previous procedures do bring some of the older test values into question, no attempt was made to alter the historical data. The analysis phase discussed in Chapter 4 considers mix design densities, stabilities, flows, VMAs, air voids and asphalt contents.

3.2.2 Structure

Three typical pavement structure types exist in Alberta Transportation and Utilities highway network and were examined in this stage. These are asphalt concrete layed on granular base course (GBC) over prepared subgrade, asphalt concrete on cement stabilized sands (soil cement) over prepared subgrade, and full depth asphalt concrete placed directly on prepared subgrade. Both structure types which utilize base materials (granular or soil cement) were often constructed as a staged process. The staged construction procedure involves the initial construction of the base and prepared subgrade and a wearing surface of asphalt stabilized base course (ASBC), with the final pavement then placed in subsequent years. (Note: Soil cement structures are built almost exclusively using staged construction).

The projects included in the review were selected to cover the range of structural types as well as varying thicknesses of structure.

3.3 Current In-Place Data

3.3.1 Materials

As it was considered important to determine the current in-place characteristics of the pavement materials, an extensive field sampling program was undertaken. Twenty eight sites were cored to obtain information on the current material characteristics. Densities of the cored pavements were measured. The asphalt was extracted and the Abson recovery method (ASTM D 1856 79) was used to recover the aged binder. Rheology testing was conducted on the recovered asphalts and gradations were measured for each of the tested samples. The results of this testing is reported in the following Chapter.

3.3.2 Structure

The field sampling of the twenty-eight project sites allowed for confirmation of the historical structural data. Each of the cores obtained was measured for asphalt concrete thickness. At each site that had GBC or soil cement (SC) base, an exploratory hole was cored or dug by hand in order to confirm the base thickness; this normally was done at a single hole for each site tested. The results of these measurements are reported in the following Chapter.

3.4 Traffic

In Alberta, traffic volumes (average annual daily traffic,AADT), vary from a few hundred to about 30000 vehicles per day when looking at rural highways and main traffic corridors respectively. The proportion of trucks operating on these roadways varies, in the range of 5-25%.

Generally, Equivalent Single Axle Loads (ESALs) are used to measure highway loadings. This measure has also been adopted for this research. The ESAL measures used were determined by Alberta Transportation and Utilities as reported in their PMS database. The method used classifies trucks as either single unit or tractor trailer combinations and uses factors of 0.56 and 1.37 for determining the ESAL's. Projects with cumulative ESALs varying from about 10000 to over 3.5 million were selected.

3.5 Summary

This chapter has examined the approach taken to select projects for the field portion of this study. The objectives for the field study were presented and the philosophy for the selection of the project sites was discussed. This chapter detailed the historical materials and structural data for the selected sites and explained the field reconnaissance undertaken for collecting current data.

TABLE III-1 HISTORICAL DATA (Sites 1 to 20)

SITE	PROJECT	STRUCTURE			ESAL's	ORIGINAL ASPHALT		CLIMATIC	AGE
		ASPHALT CONCRETE	ASBC	BASE	(X1000)	PENETRATION (dmm,25C,5s)	ABSOLUTE VISCOSITY (Pa.s)	VALUE (°C)	(YEARS)
1	22:28	100mm (200-300)	yes	250mm GBC	89.4	263	39.8	28	5
2	1A:02	100mm (200-300)	no	150mm GBC	674	168	67.0	27	7
3	2A:06	200mm (200-300)	no	Fulldepth	133*	317	30.1	28	12
4	24:02	50mm (200-300)	yes	175mm GBC	551*	266	32.9	29	17
5	24:02	50mm (200-300)	yes	305mm GBC	551*	266	32.9	29	17
6	21:12	50mm (200-300)	yes	150mm CS	365*	285	30.0	29	17
7	21:12	75mm (200-300)	yes	150mm CS	365*	285	30.0	29	17
8	1:10	250mm (200-300)	no	Fulldepth	1710	243	29.1	29	12
9	41A:02	50mm (200-300)	yes	230mm GBC	412*	278	25.5	33	14
10	1:16	150mm (150-200)	no	280mm GBC	803*	SEAP		31	5
11	1:16	150mm (150-200)	no	280mm GBC	803*	SEAP		31	5
12	887:04	150mm RACP	no	100mm GBC	41.2	168	62.0	29	3
13	529:04	150mm RACP	no	100mm GBC	11.7*	208	67.5	28	4
14	512:02	150mm	no	Fulldepth	82.1*	295	34.5	28	10
15	524:04	125mm (200-300)	no	100mm GBC	86.5	266	43.5	28	5
16	507:02	125mm RACP	no	150mm GBC	23.7*	160	72.0	29	4
17	16:12	130mm (120-150)	yes	460mm GBC	627	166	76.5	28	4
18	22:30	100mm (200-300)	yes	250mm GBC	313	284	36.0	28	5
19	22:30	100mm (200-300)	yes	200mm GBC	576	311	34.9	28	6
20	11:12	100mm (200-300)	yes	50mm GBC	1449	N/A	N/A	29	31

* Estimated values based on 1986 data
(The ESAL values are cumulative over the life of the pavement)

TABLE III-1 HISTORICAL DATA (Sites 21 to 40)

SITE	PROJECT	STRUCTURE			ESAL's	ORIGINAL ASPHALT		CLIMATIC	AGE
		ASPHALT CONCRETE	ASBC	BASE	(X1000)	PENETRATION (dmm,25C,5s)	ABSOLUTE VISCOSITY (Pa.s)	VALUE (°C)	(YEARS)
21	11:10	100mm (150-200)	yes	250mm GBC	943	N/A	N/A	28	30
22	11:08	100mm (200-300)	yes	300mm GBC	182*	268	45.0	28	13
23	11:08	100mm (200-300)	yes	250mm GBC	179*	287	35.0	28	11
24	11:06	100mm (200-300)	yes	250mm GBC	109*	300	33.7	28	9
25	11A:02	75mm (200-300)	yes	N/A	121	274	38.1	28	17
26	2A:18	100mm (150-200)	yes	300mm GBC	1481	N/A	N/A	29	29
27	12:20	125mm (150-200)	no	100mm GBC	74.8*	182	69.7	30	9
28	12:20	50mm (200-300)	yes	200mm GBC	113*	270	44.8	30	15
29	9:14	63mm (200-300)	yes	150mm SC	310*	248	42.3	30	20
30	49:02	50mm (200-300)	yes	180mm SC	139	278	39.7	27	6
31	49:04	200mm (200-300)	no	Fulldepth	456*	238	29.8	27	18
32	35:12	100mm (200-300)	yes	150mm SC	376	286	23.7	27	12
33	35:16	100mm (200-300)	yes	300mm GBC	212*	222	34.6	27	10
	881:12	180mm	no	Fulldepth	77.4	268	43.4	29	9
	881:12	130mm (200-300)	no	100mm GBC	77.4	270	42.0	29	7
	2:38	50mm (200-300)	yes	150mm SC	328*	N/A	N/A	28	21
	2:40	50mm (200-300)	yes	150mm SC	265	258	29.9	28	19
	63:01	50mm (200-300)	yes	175mm SC	806	267	43.1	28	16
39	2:32	300mm (200-300)	no	Fulldepth	3686*	230	54.5	28	8
40	520:02	100mm (200-300)	no	Fulldepth	92.3	270	44.1	29	12

* Estimated values based on 1986 data
(The ESAL values are cumulative over the life of the pavement)

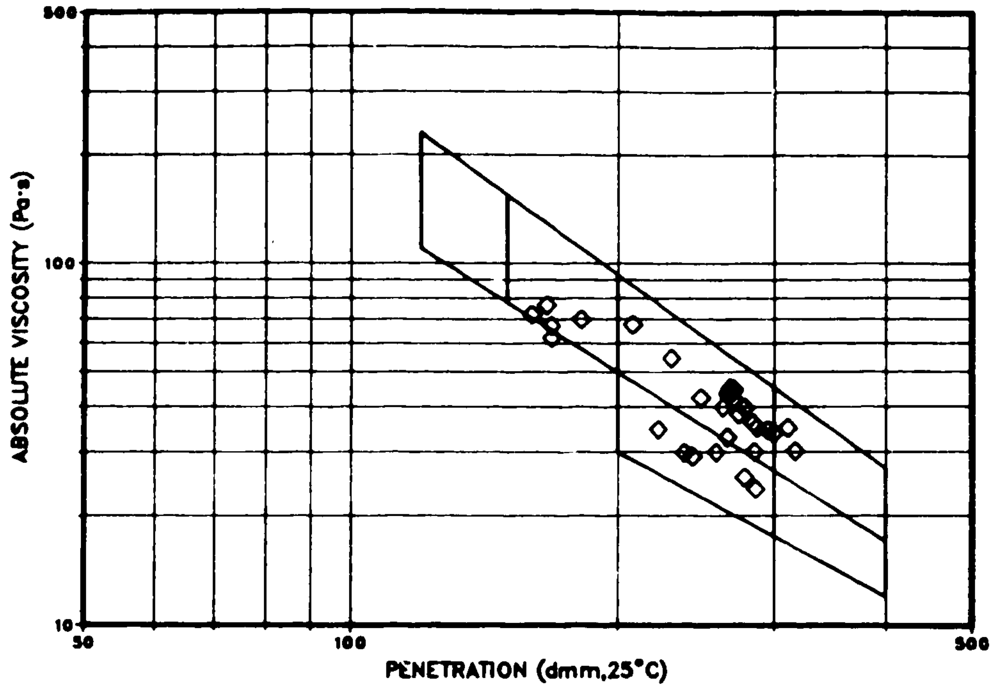


FIGURE III-1 RANGE OF ORIGINAL ASPHALT RHEOLOGY

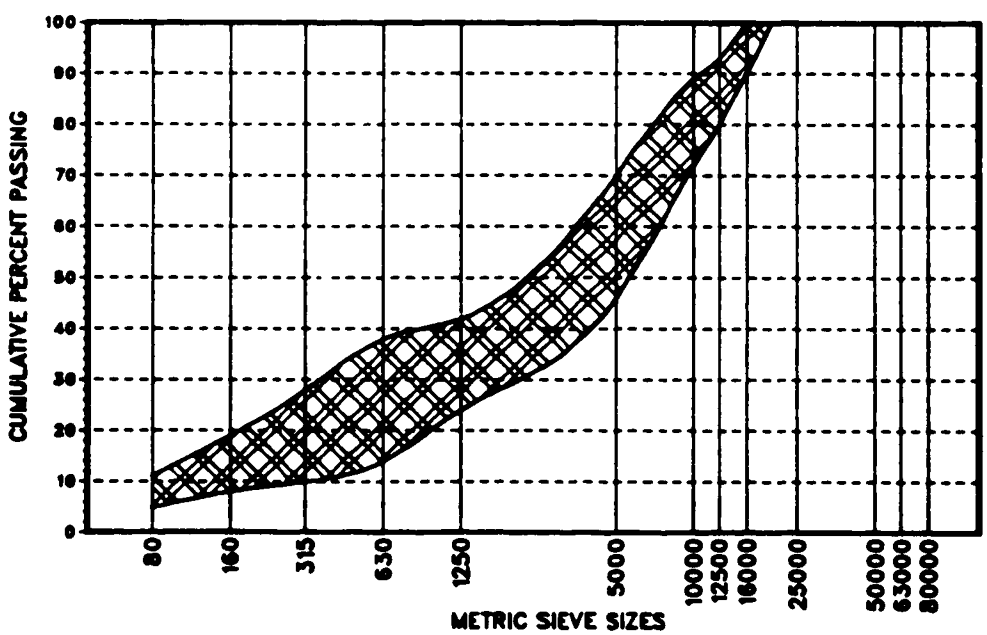


FIGURE III-2 RANGE OF AGGREGATE GRADINGS (DESIGN)

CHAPTER IV
STAGE I - RESULTS AND ANALYSIS

4.0 INTRODUCTION

Though the project selection process was aimed at selecting projects with uniform characteristics, most projects had specific characteristics which constituted a special case. The most common problem with various sites was the difference between lifts, because of the use of different materials. As well, numerous projects had more than one mix design reported and it was virtually impossible to determine which specific design was applicable for the selected site. The final approach taken utilized average values for the material characteristics to use in the analysis; however all the rut depth measurements at the cross-section site were used.

4.1 Field Test Results

The rut depths measured as part of the field testing program varied between 0 and 29 mm; however the average value was 4.6 mm. Table IV-1 shows the average of the rut depths measured for each site which were used in the analysis. The low rut depths measured shows that rutting was not a significant form of distress for the highways examined.

The surveyed cross-sections are contained in Appendix B. Examination of the plotted cross-sections suggests lateral movement or shoving of the mix in cases where rutting has occurred; this indicates that the rutting experienced is at least in part due to instability of the asphalt concrete.

Nine of the test sites were cored more extensively to allow the location of the rutting to be determined; that is, whether the rutting was confined to the asphalt concrete, or whether materials below the asphalt concrete also showed rutting to be present. Cores were extracted across the roadway width. The core depth was superimposed on the cross-section plot and the profile of the bottom of the pavement established. The occurrence of rutting below the asphalt concrete layer could then be noted. Figure IV-1 to IV-3 show these plots. It can be seen that sites 7, 10 and 26 show some rutting has occurred in the lower materials. It is difficult to determine the percentage of the rut depth which can be attributed to movement below the asphalt concrete. In some cases it would appear to be 100% as for three of the ruts in Figure IV-1, site 7; however for the nine projects examined it would appear that the occurrence is minimal for the most part. This is most clearly noted for Figure IV-3, site 39, which shows no significant movement in the lower layers for this extremely rutted site.

The aim of this research was to examine only the rutting in the asphalt concrete layer. The occurrence of rutting below this layer at some of the sites introduced deviations from this objective. However it was not considered practical, nor possible, to obtain sufficient data to determine the rutting in just the asphalt concrete layer for all of the selected sites. As a result, the measured overall rutting was used as the dependant variable in the correlations. Therefore the fact that this measurement may not always be a true reflection of the asphalt concrete layer's performance must be considered when interpreting the results. However, based on the nine sites examined, the cross-section plots and the pavement slabs discussed in Appendix A, it would appear that there is minimal risk in interpreting the results incorrectly due to other rutting.

4.2 Test Results from Coring

The Abson recovered asphalt binders were tested for penetration (dmm @ 25°C) and both absolute (Pa.s @ 60°C) and kinematic (mm^2/s @ 135°C) viscosities. Multiple tests were conducted for each test site, for the various samples obtained. Table IV-1 presents the average values for the sites tested. Figure IV-4 illustrates the range of binder rheology determined from the cores.

The degree of asphalt aging for the various projects was also examined. It was felt that if the rheological properties of the binder were to be correlated to the observed rutting, that the change of these properties, concurrent with loadings, could be important. Changes in viscosities and penetration, calculated as % retained penetration and viscosity ratio were included as additional material variables in the analysis.

The aggregate gradation was obtained as a part of the Laboratory testing on the retrieved cores. Figure IV-5 illustrates the range in gradations for each of the tested sites. The actual gradation data is presented in Appendix B.

Measurements of the retrieved cores allowed the historical structural data to be confirmed. Table IV-1 presents the average structural thicknesses determined for the selected sites.

4.3 Analysis of Field Data

The analysis of the field data involved correlating the measured characteristics with the observed pavement rutting. The observed rutting was considered as the dependant variable, with all other parameters considered as independant variables. In order to examine the various parameters involved, all valid data were entered into the Alberta Government's mainframe IBM computer system and stored in a FOCUS[20] database. The FOCUS "information control system" was utilized for data handling and statistical analysis and is explained in Appendix E.

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

McLeod's[4] Pen-Vis number was calculated as the measure of temperature susceptibility of the asphalt binder. The stiffness values were determined from McLeod's[4] modified nomograph for binder stiffnesses. The temperature values used for the stiffness calculations were those reported in the previous chapter. A loading frequency of 1/s was used to be consistent with the asphalt concrete mixes tested in the second stage of this investigation. Rut measurements were made in the outside wheel path (OWP) and inside wheel path (IWP) for each lane at each project site. For those sites located on divided highways, (sites 8,10,11,17,39), the rut data from the travel lane only was used in the analysis. The variables which were loaded into the field database are shown schematically in Figure IV-6.

As a first step in the analysis procedure the individual correlations of each parameter to the measured rut depth was examined, as well as the cross correlations to each of the other variables in the data base. Those variables which had extremely low correlations to both the rut depths (dependant variable) and to the other variables were discarded at this point. This resulted in a smaller number of variables for consideration in the model development.

This initial examination of the correlation matrix for all of the variables was followed by additional considerations of the identified parameters as various cross-products and transformations. The cross-products examined were based on the practical significance of each of the variables. For example the product of the temperature and cumulative ESAL's was examined as an indicator of the combined influence of temperature and loading on the development of the observed rutting. Additionally, log transformations of most variables were considered to determine if they were better correlated to the permanent deformation.

The result of this selection process led to the development of a number of empirical models containing the independant variables found to most significantly influence the development of rutting in the highway system.

Because there was only Abson recovered rheology data available for the 28 projects which were cored, the observed rutting was examined in terms of the design data (ie: data available at the design stage) separately. The results of this analysis showed the loading (daily cumulative ESAL's) to be the most significant contributor to the observed rutting. The original binder stiffness, which reflects climatic data and the rheology was the other significant variable which was brought into the model. The model developed for the design data was:

$$\text{Rut depth} = 2.6186 + 0.0060*(\text{Daily Cumulative ESAL's}) - 0.0023*(\text{Daily Cumulative ESAL's})*\text{Log}(\text{Original Binder Stiffness}) \quad \text{(IV-I)}$$

The regression coefficient of 0.6737 (r^2=0.4539) for this model indicates that only 45% of the variation in the data is explained by the model. The analysis of variance (ANOVA) for this model, which utilized data from 35 sites, is presented in Table IV-2. The calculated F value is 2.97 which indicates the model has lack-of-fit (L.O.F.) (probability ≤ 0.05)[26].

All of the variables were considered in the subsequent analysis, which reduced the number of observations to 28 sites, corresponding to the sites which were cored. The results of this analysis showed the stiffness of the Abson recovered asphalts to be the most significant variable, in conjunction with the traffic loadings. The model developed was;

$$\text{Rut Depth} = 2.9630 + 0.0076*(\text{Daily Cumulative ESAL's}) - 0.0024*(\text{Daily Cumulative ESAL's})*\text{Log}(\text{Abson Stiffness}) \quad \text{(IV-II)}$$

This model had a correlation coefficient of 0.7898 (r^2=0.6237) The calculate F value is 1.58 which does not suggest any L.O.F. for the model (probability ≤ 0.05) The ANOVA for this model are shown in Table IV-3.

4.4 STAGE I - Discussion

This chapter has presented the results of the field reconnaissance. The field measurements of rut depths on the 40 projects examined suggests a low incidence of rutting throughout the province. The rut profiles, presented in Appendix B, indicate that where rutting has occurred there is generally some evidence of the mix shoving. The coring program showed that some rutting is occurring below the asphalt concrete layer in a number of sites.

The model forms developed to explain the observed rutting measured at the sites used in this study identifies loading and asphalt stiffness as the significant variables. The significance of the first model is that the inputs required are available at the design stage. The model shows that for given loadings the stiffness of the binder will influence the amount of rutting experienced. Chapter VII discusses the practical implications suggested by the models developed in this Chapter.

The magnitude of the pure error of the measured rut depths tends to mask the effects of some factors and must be noted. As well, other significant correlations which exist, such as the ACP thickness, must be considered. These variables are not significant in models containing the cumulative ESAL's as a variable partly because of the correlations each have to the loadings; however they can be considered separately if care is taken to account for these secondary correlations. The fact that mix characteristics were not included in the model does not preclude their significance. The majority of the pavement sites in this investigation had mix characteristics near the design values, which may have affected the analysis. A significant range of mix characteristics was not a specific goal of this project. However it should be considered when examining the presented models, as the relationships are representative for the mix characteristics examined.

TABLE IV-1 STRUCTURAL AND RHEOLOGICAL DATA
(From Cores)(Sites 1 - 20)

SITE	STRUCTURE		ABSON EXTRACTED ASPHALT			
	ASPHALT CONCRETE	BASE	PENETRATION (dmm, 25°C)	ABSOLUTE VISCOSITY (Pa.s, 60°C)	KINEMATIC VISCOSITY (mm^2/s, 135°C)	MEASURED RUT DEPTHS (mm)
1	200mm	230mm GBC	106 (40.3)	167.9(4.2)	339	4.0
2	- mm	- mm GBC	-	-	-	0.0
3	208mm	Fulldepth	156 (49.3)	78.8 (2.6)	236	6.0
4	146mm	200mm GBC	127 (47.9)	89.1 (2.7)	226	7.5
5	114mm	308mm GBC	105 (39.7)	112 (3.4)	259	5.5
6	100mm	135mm CS	122 (42.7)	85.8 (2.9)	214	3.8
7	131mm	105mm CS	115 (40.3)	104 (3.5)	236	6.3
8	241mm	Fulldepth	118 (48.5)	79.7 (2.7)	237	6.0
9	- mm	- mm GBC	-	-	-	2.0
10	- mm	- mm GBC	-	-	-	9.5
11	- mm	- mm GBC	-		-	15.0
12	162mm	90 mm GBC	87 (51.6)	204 (3.3)	374	2.3
13	160mm	108mm GBC	91 (43.9)	216 (3.2)	405	3.3
14	165mm	Fulldepth	170 (57.3)	76.7 (2.2)	248	7.3
15	- mm	- mm GBC	-	-	-	5.8
16	123mm	150mm GBC	66 (41.1)	325 (4.5)	422	3.8
17	196mm	415mm GBC	82 (49.2)	282 (3.7)	456	3.5
18	160mm	350mm GBC	126 (44.2)	121 (3.4)	298	2.3
19	- mm	- mm GBC	-	-	-	1.0
20	- mm	- mm GBC	-	-	-	4.5

Note: Bracketed values under Penetration and Absolute viscosity are % retained penetration and viscosity ratio respectively.

TABLE IV-1 STRUCTURAL AND RHEOLOGICAL DATA
(From Cores)(Sites 21 - 40)

SITE	STRUCTURE		ABSON EXTRACTED ASPHALT			
	ASPHALT CONCRETE	BASE	PENETRATION (dmm, 25°C)	ABSOLUTE VISCOSITY (Pa.s, 60°C)	KINEMATIC VISCOSITY (mm²/s, 135°C)	MEASURED RUT DEPTHS (mm)
21	175mm	332mm GBC	84 (43.9)	289 (9.6)	480	7.5
22	- mm	- mm GBC	-	-	-	1.0
23	- mm	- mm GBC	-	-	-	3.5
24	- mm	- mm GBC	-	-	-	1.5
25	140mm	124mm GBC	129 (47.0)	117 (3.1)	276	3.3
26	144mm	407mm GBC	71 (41.9)	180 (9.0)	302	7.8
27	135mm	[illegible] GBC	77 (42.0)	308 (4.4)	432	3.0
28	102mm	[illegible] GBC	75 (27.8)	321 (7.2)	452	2.5
29	- mm	- mm GBC	-	-	-	2.8
30	121mm	140mm SC	99 (35.6)	184 (4.6)	345	5.0
31	187mm	Fulldepth	91 (38.3)	143 (4.8)	289	3.8
32	156mm	165mm SC	76 (26.7)	142 (6.0)	273	4.0
33	192mm	- mm GBC	109 (49.2)	91.4 (2.6)	235	2.8
34	156mm	Fulldepth	-	-	-	5.0
35	137mm	120mm GBC	119 (44.2)	162 (3.9)	352	4.0
36	117mm	147mm SC	95 (33.7)	161 (4.0)	337	5.8
37	103mm	115mm SC	130 (50.6)	82.2 (2.7)	271	3.5
38	73 mm	223mm SC	177 (66.3)	80.0 (1.9)	262	7.0
39	310mm	Fulldepth	194 (84.3)	84.6 (1.6)	264	28.5
40	173mm	Fulldepth	61 (22.5)	471 (10.7)	532	3.0

Note: Bracketed values under Penetration and Absolute viscosity are % retained penetration and viscosity ratio respectively.

TABLE IV-2 RUT PREDICTION MODEL USING DESIGN DATA

INTERCEPT	2.61869
MULTIPLE CORRELATION	.67376
STD. ERROR OF ESTIMATE	3.03042
R-SQUARED	.45395

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
1	1146.794	1493.5940	.6196	.00604090	.0010	5.8638
3	2175.898	2783.8833	.5680	-.00236642	.0006	-4.2814
DEPENDENT						
4	4.397	4.0726				

RUT DEPTH = 2.6186857 + 0.00604090*(Cumulative Daily ESAL's)

- 0.00236642*(Daily Cumulative ESAL's)*Log(Stiffness)

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	2	1091.7173	545.8586
DEVIATION FROM REGRESSION	143	1313.2288	9.1834
TOTAL	145	2404.9460	
Lack of Fit	35	643.7200	18.3922
Pure Error	108	669.5000	6.1991

F value = 2.97

TABLE IV-3 PREDICTION MODEL USING ABSON DATA

INTERCEPT	2.96304
MULTIPLE CORRELATION	.78981
STD. ERROR OF ESTIMATE	2.64943
R-SQUARED	.62379

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
1	1160.038	1619.5391	.7516	.00769300	.0014	5.3816
3	2852.423	3873.7708	.7197	-.00240128	.0006	-4.0179
DEPENDENT						
4	5.038	4.2782				

RUT DEPTH = 2.9630384 + 0.00769300*(Daily Cumulative ESAL's)

- 0.00240128*(Daily Cumulative ESAL's)*Log(Abson Stiffness)

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	2	1198.8315	599.4158
DEVIATION FROM REGRESSION	103	723.0085	7.0195
TOTAL	105	1921.8401	
Lack of Fit	25	244.0085	9.7603
Pure Error	78	479.0000	6.1410

F value = 1.58

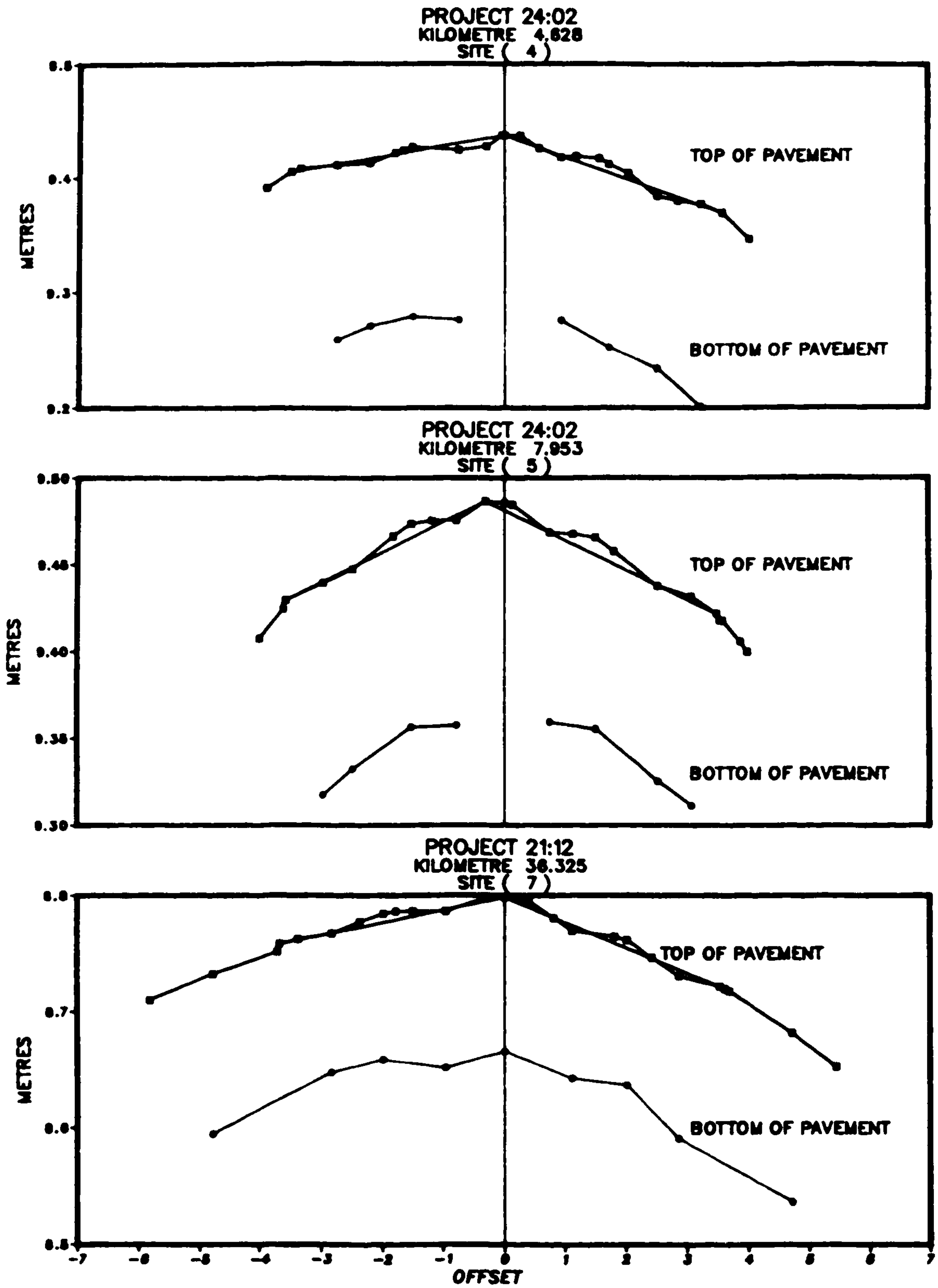


FIGURE IV-1 PROFILE BELOW PAVEMENT STRUCTURE

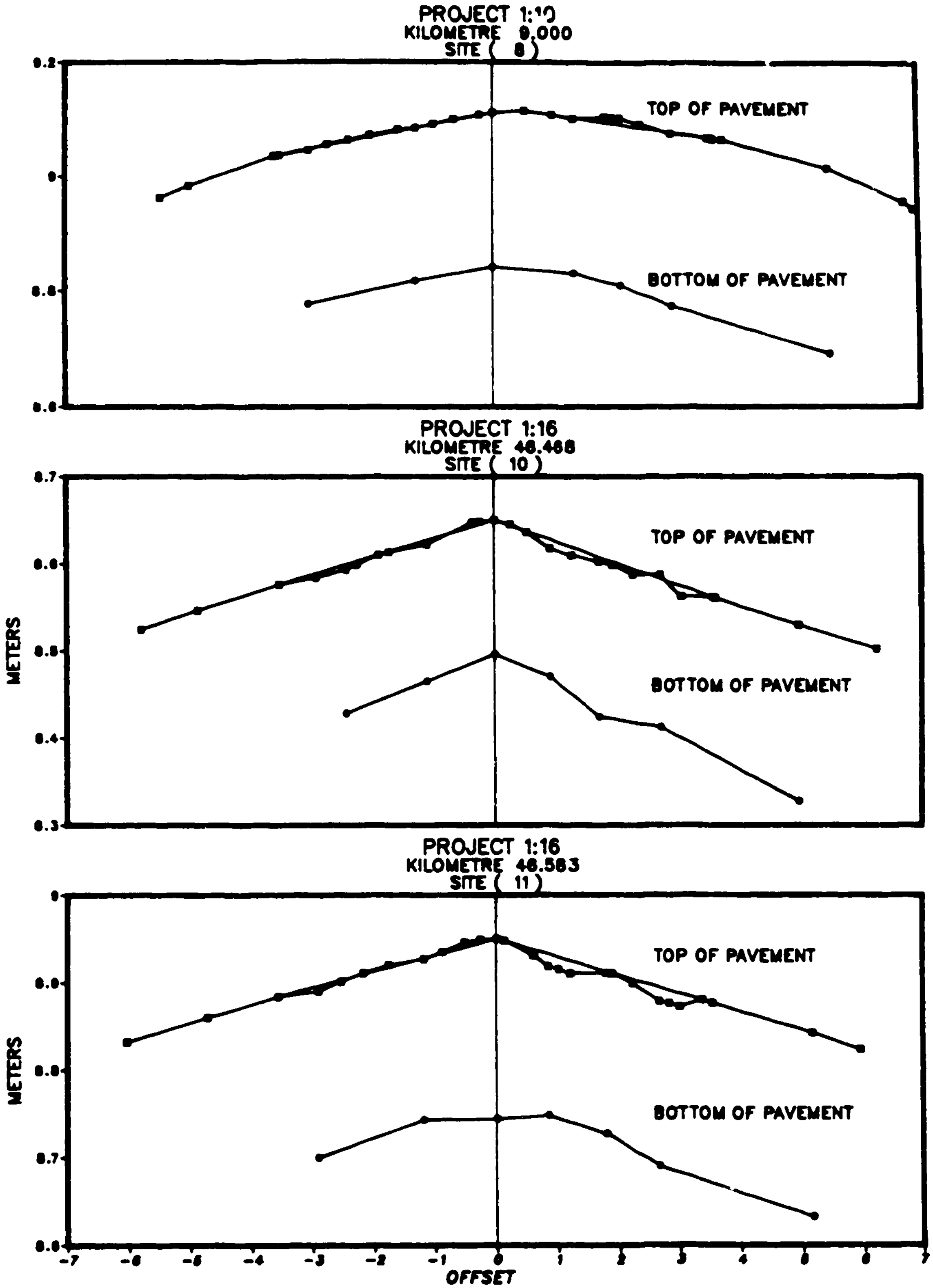


FIGURE IV-2 PROFILE BELOW PAVEMENT STRUCTURE

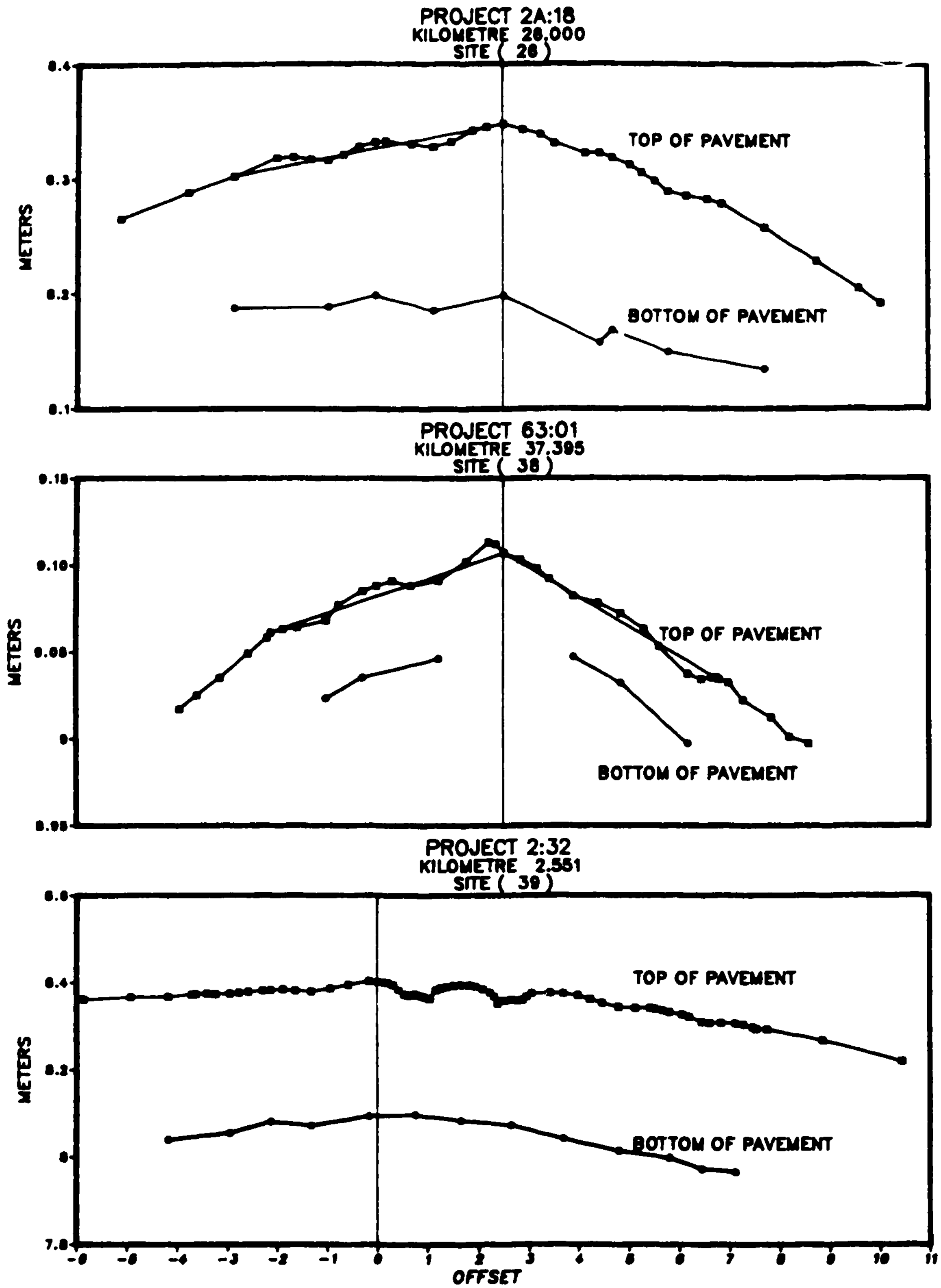


FIGURE IV-3 PROFILE BELOW PAVEMENT STRUCTURE

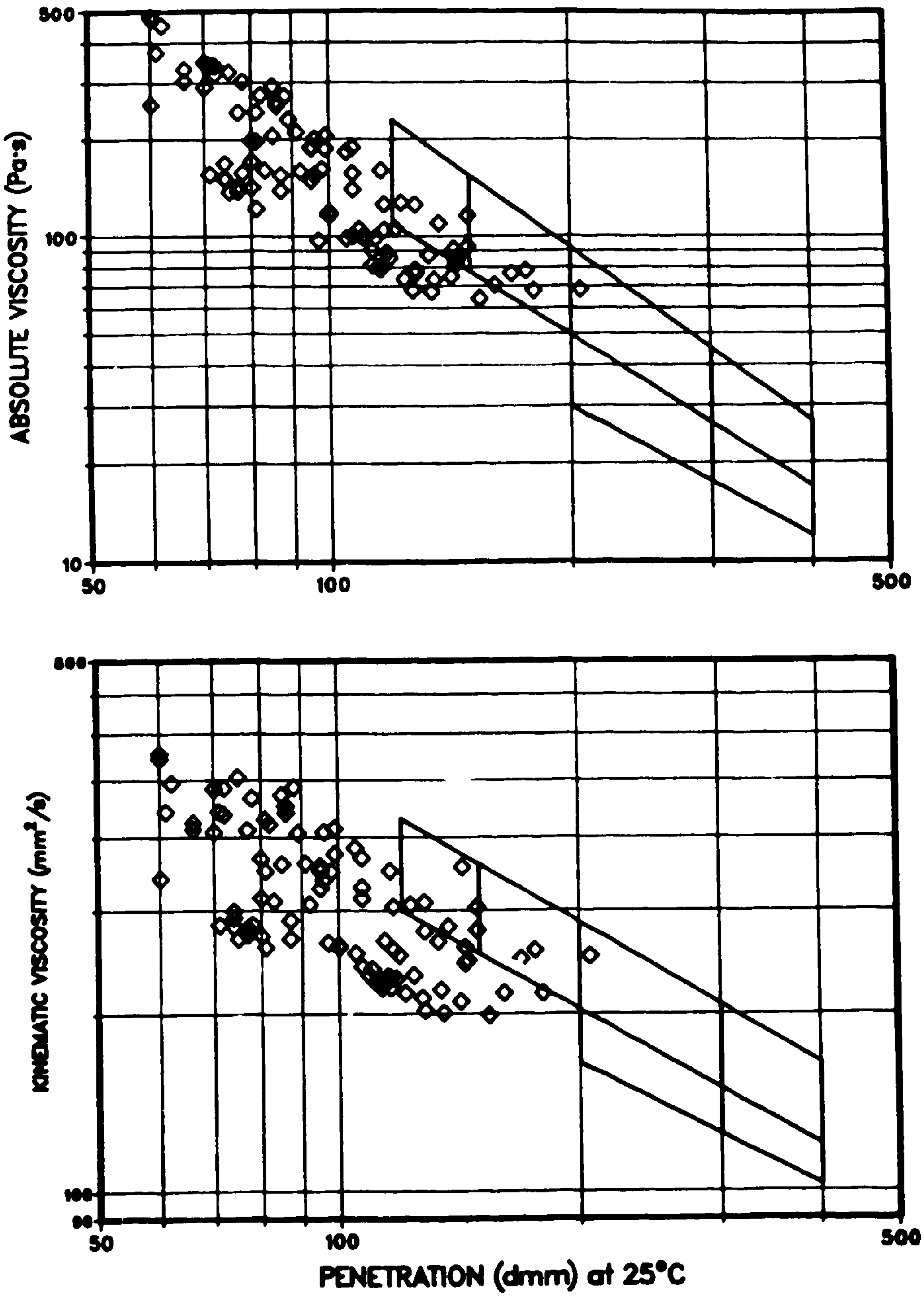


FIGURE IV-4 RHEOLOGY DATA (ABSON RECOVERED ASPHALT)

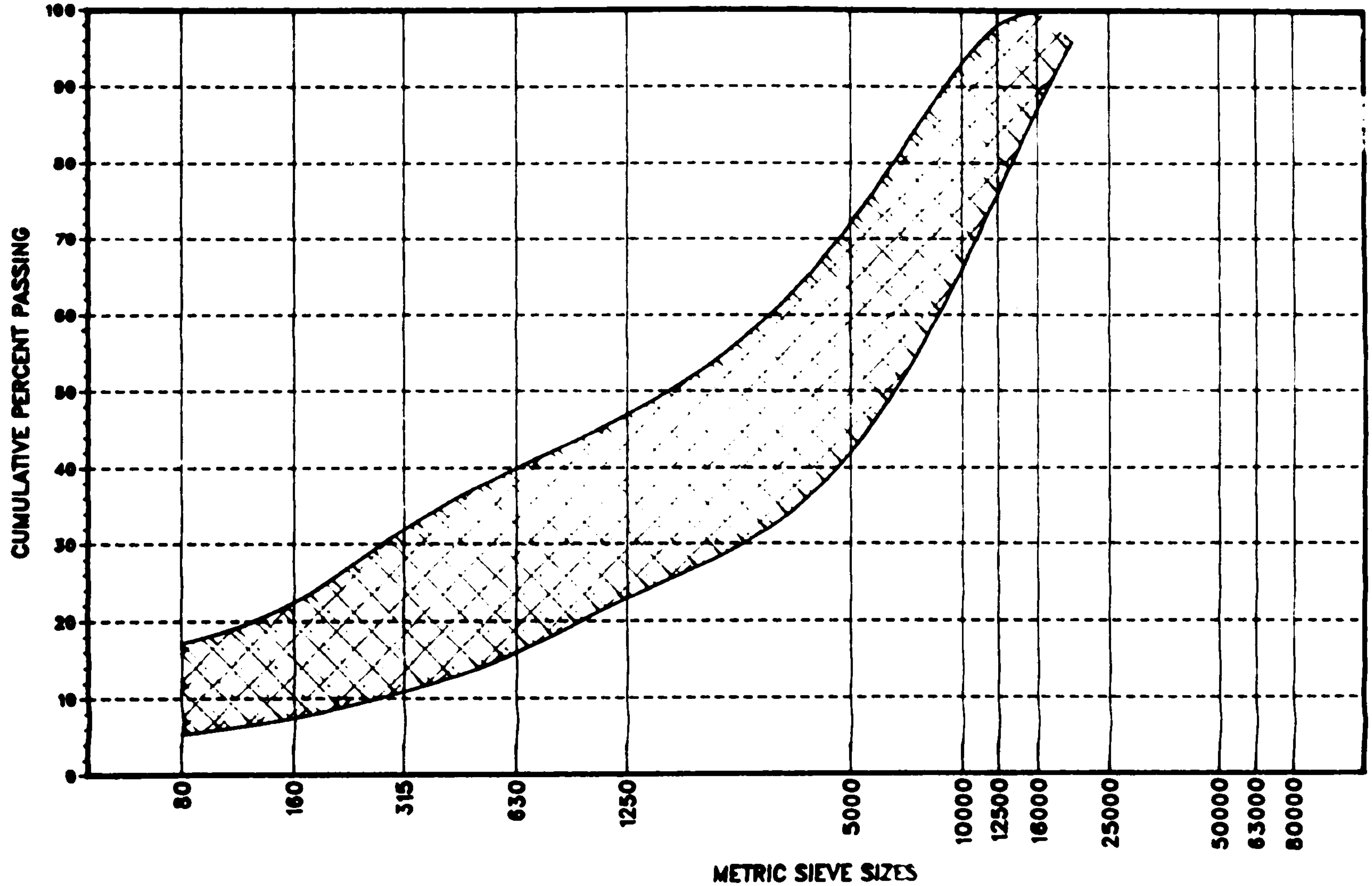


FIGURE IV-5 RANGE OF AGGREGATE GRADINGS (FROM EXTRACTED CORES)

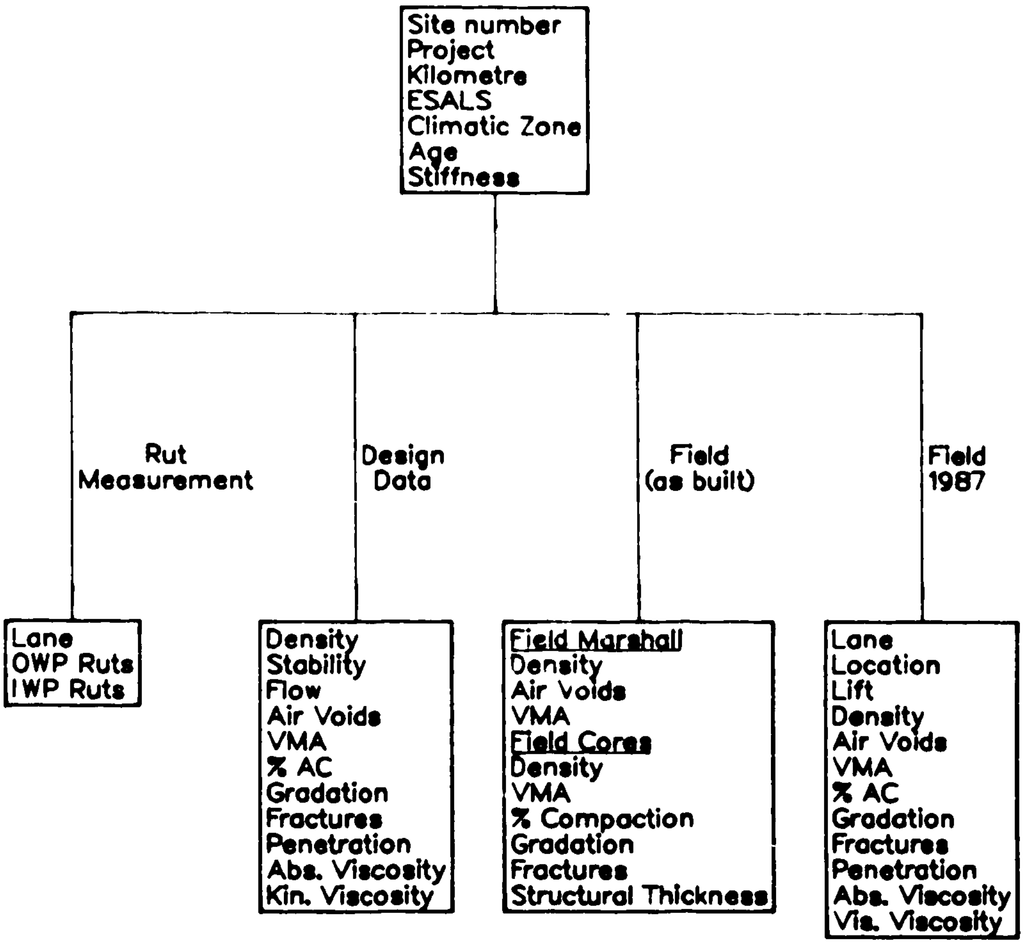


FIGURE IV-6 FIELD DATABASE VARIABLES

CHAPTER V

STAGE II - LABORATORY TESTING PROGRAM

5.0 INTRODUCTION

As previously noted in Chapter I, the second stage of the investigation involved a laboratory testing program conducted using repeated load triaxial testing equipment. This testing program was designed to determine the influence of the constituent components, of the asphalt concrete, on permanent deformation. The objectives for this stage are stated as follows:

1. To isolate the specific influence of the binder rheology on the permanent deformation of asphalt concrete mixes.
2. To examine the permanent deformation of recycled mixes under repeated loading; specifically to compare the behaviour of the predicted rheological properties of the recycled mixture to similar rheology of virgin mixes.
3. To quantify any improved characteristics of polymer modified binders in terms of resistance of asphalt concrete mixes to permanent deformation.
4. To determine the influence of aggregate characteristics on the permanent deformation of asphalt concrete mixes.
5. To examine the effect of temperature on permanent deformation, in relationship to mix stiffness and asphalt temperature susceptibility.
6. To examine the variation in results between tests on field and laboratory prepared mix samples.

The testing approach and material characteristics were determined to meet these objectives within the constraints of the facilities and materials available for this research.

5.1 Testing Approach

While the work conducted for this stage of the investigation is aimed at meeting the specific objectives stated, it is also intended to build upon previous work in this area. For this reason the testing approach follows that of Hadipour[15]. Hadipour utilized the repeated load triaxial test to attempt to simulate field conditions. The loading conditions have been governed to some extent by the limitations of the testing equipment. An applied vertical stress of 350 kPa and a lateral confining pressure of 35 kPa were used. A loading time and rest time of 0.5 seconds each was used. These loading conditions are consistent with Hadipour's previous work. The load was applied through pneumatic cylinders controlled by elect[illegible] solenoids. The signal to the solenoids (0.5 s on/off) was provided as a s[illegible] wave, the resultant loading curve 'felt' by the load cell was approximately a 0.[illegible] load build up time followed by approximately 0.3 s of constant load, followed by an immediate drop off. This loading time represents a vehicle speed of approximately 10 km/h[15]. While this long loading time is not realistic of highway traffic speeds it does reduce the required number of loading cycles required to reach a specific level of deformation. A detailed description of the repeated load triaxial testing is presented in Appendix C.

The goal of the laboratory testing was to develop relationships between permanent deformation and the binder rheology, the aggregate used and the test temperature. It was also desired to determine the effect of the different binder types (virgin, recycled and PMA).

5.2 Materials Characteristics

The materials utilized for the testing program represent asphalt concrete paving materials used on two projects in the Province of Alberta. Two aggregate sources, each with a different blend sand source, were used for the test mixtures. The two aggregate sources were the Goose Lake Pit, near Calgary and the Blackfalds Pit, near Red Deer. The measured properties of these aggregates are given in Table V-1 with the gradations plotted in Figure V-1. For the recycled mixtures, two sources of reclaimed asphalt pavement (RAP) were used, one RAP source for each of the aggregate sources. The aggregate data for the RAP material used is also shown in Table V-1 and Figure V-1. Both RAP materials were reclaimed from high traffic highways in Alberta because of excessive [illegible]ng. (Highway 1:10, East of Calgary, and Highway 2:26, North of Red Deer). Both materials were originally placed in the early 1970's.

The asphalt binder materials used represent the full spectrum of premium asphalts used by Alberta Transportation. These are 120-150A, 150-200A, 200-300A, and 300-400A grade asphalts. For this testing program the virgin asphalt cements were supplied by Husky Oil Limited. The rheological properties of the binders are given in Table V-2 while Figure V-2 illustrates these results with respect to the current specifications used by Alberta Transportation for their asphalt supply. It can be noted that the lower viscosity binder grades were not included in the testing program because there is virtually no use of these 'B' grade binder for highway paving in Alberta. The binder characteristics of the RAP from Highway 1:10 and Highway 2:26 determined on material recovered using the Abson test procedure (ASTM D 1856 79) are shown in Table V-2.

Two polymer modified asphalts (PMA) were also used in the testing program, one manufactured by Imperial Oil and one by Husky Oil. Both PMA products were manufactured in 1987 and are samples obtained from the field during the construction of test sections. The rheology data for the PMA binders are also given in Table V-2. It should be noted that PMA binders generally do not behave as Newtonian liquids at lower temperatures and therefore the test for Absolute viscosity (@ 60°C) should be done at a known shear rate to allow comparisons to be made. The testing of the Absolute viscosity for the PMA binders in this study were not conducted at specific shear rates and therefore these viscosities are not a good indication of the material characteristics at this temperature.

The binders in the recycled mixes were designed specifically to correspond to specific virgin asphalt grades. To determine the characteristics required of the virgin binders used to soften the recycled mixes, known logarithmic relationships were used as shown in Figures V-3 and V-4. Based on the values determined from these plots, paving grades of virgin asphalt binders were blended in the laboratory to obtain the desired asphalt blend. Using these blended asphalts, the recycled mix samples were prepared with rheological properties similar to those of the corresponding virgin samples as shown in Table V-3 and Figure V-2.

The use of soft virgin asphalts as the rejuvenating or softening agent in the recycling of asphalt pavements is the general practice in Canada, though many American agencies utilize special recycling additives. Alberta Transportation uses virgin binders exclusively for recycling and, as previously stated, one of the objectives of this work was to determine the physical behaviour of such mixes, with respect to the theoretical rheological properties predicted.

5.3 Mix Designs

Alberta Transportation conducted mix designs for the two projects from which the aggregate and RAP materials were obtained for this investigation. The mix designs were performed using the Marshall method of mix design as prescribed by the Asphalt Institute[21]. Additional Marshall briquettes were also formed, at the design asphalt contents, using the various asphalt grades used in this investigation, in order to obtain stability values for these mixes. The design data from this extra testing are given in Table V-4.

The recycled mix designs utilized the asphalt grades available in Alberta to obtain a resultant asphalt rheology in the range desired. For this research it was desired to maintain the recycled asphalt rheology as similar as possible to that of the virgin asphalt grades.

Mix design summaries are given in Figures V-5 to V-10. Each design summary shows the design characteristics of the mix determined from the design curves. The design curves are based on the best fit through four test results at each asphalt content. The design summaries also show the design aggregate gradings for the mix.

5.4 Testing Design

A factorial experiment was used as the experimental framework of the testing program. The testing considerations for the previously stated objectives are given below:

1. Use of binders with different rheologies.
2. Use of recycled mixes with predicted binder rheology simi[illegible] to virgin samples.
3. Use of polymer modified asphalts.
4. Use of different aggregates.
5. Use of varying test temperatures.

These various considerations were then designed into the testing program as follows:

Variable	**Range**	**Levels**
Binder Type	conventional,PMA,recycle	3
Binder [illegible]	120-150A,150-200A,200-300A,300-400A grade asphalts	4
Aggregate	Blackfalds pit,12.5 mm / Goose Lake pit,16 mm	2
Temperature	25°,35° and 45°C	2 and mid-points

These four variables were then considered at each of the levels indicated. Testing was conducted with combinations of each level for the four [illegible]iables. For example, the conventional asphalt was tested for each of the of the binder rheologies at each temperature and for both aggregate sources. This approach resulted in the following number of tests.

Goose Lake Pit

Sample Series	Description	Number of Samples
1LG	Virgin (120-150A)	(9 samples)
2LG	Virgin (150-200A)	(12 samples)
3LG	Virgin (200-300A)	(9 samples)
4LG	Virgin (300-400A)	(6 samples)
5LG	PMA mix (Husky)	(6 samples)
6LG	PMA mix (Imperial)	(6 samples)
7LG	R/V=25/75 (Equivalent 120-150A)	(9 samples)
8LG	R/V=25/75 (Equivalent 150-200A)	(12 samples)
9LG	R/V=25/75 (Equivalent 200-300A)	(9 samples)
10LG	R/V=25/75 (Equivalent 300-400A)	(6 samples)
11LG	R/V=50/50 (Equivalent 120-150A)	(3 samples)
12LG	R/V=50/50 (Equivalent 200-300A)	(3 samples)

Blackfalds Pit

Sample Series	Description	Number of Samples
1LB	Virgin (120-150A)	(9 samples)
2LB	Virgin (150-200A)	(12 samples)
3LB	Virgin (200-300A)	(9 samples)
4LB	Virgin (300-400A)	(6 samples)
5LB	PMA mix (Husky)	(6 samples)
6LB	PMA mix (Imperial)	(6 samples)
7LB	R/V=15/85 (Equivalent 120-150A)	(9 samples)
8LB	R/V=15/85 (Equivalent 150-200A)	(12 samples)
9LB	R/V=15/85 (Equivalent 200-300A)	(9 samples)
10LB	R/V=15/85 (Equivalent 300-400A)	(6 samples)
11LB	R/V=35/65 (Equivalent 120-150A)	(3 samples)
12LB	R/V=35/65 (Equivalent 200-300A)	(3 samples)

Testing triplicate samples, the total number of 102 X 204 (mm) samples = 180 for this testing design. Field samples were prepared in addition to these for comparative testing, the field sample program is discussed in Appendix D.

5.5 Test Specimens

The asphalt concrete specimens used for the testing program were formed 102mm in diameter by 204mm using a California kneading compactor. The specimens were prepared to result in uniform densities throughout and with air voids in the 3 - 4% range. Air void determination was based on properties determined from Marshall designs performed on each of the mixes.

Because it was felt that uniform sample density would contribute to more uniform test results, trial and error was used to determine the best procedure to result in uniform density throughout the sample. Following previous sample preparation work of this type by Alberta Transportation, the procedure utilized required approximately 4.1 Kg of mix compacted into the 102mm X 204mm split molds using the California Kneading compactor. The first third of the mix was compacted using 25 blows at 1.72 MPa (250psi), 65 blows at the same pressure on the second third of mix, and 110 blows at 3.45 MPa (500psi) for the final third. The number of blows did vary for some of the mixes to obtain a uniform target density. It can be noted that the mixes used for these trials required less compactive effort than those utilized in Hadipour's[15] work, and highlights the fact that other mixes would need to be similarly evaluated.

Following the sample formation, the density and height were measured and the air voids were calculated. The formation of the field samples is discussed separately in Appendix D.

5.6 Summary

The design of the Laboratory testing program and the material considerations have been presented in detail in this Chapter. The design data for the binders, aggregates and mix characteristics were presented along with the design approach for the actual testing program. The results of this testing program, and subsequent evaluation is presented in the following Chapter.

TABLE V-1 AGGREGATE DATA

METRIC SIEVES	GOOSE LAKE PIT (C)	(F)	JOHNSON BLEND SAND	RAP HW 1	BLACKFALDS PIT (C)	(F)	BALL BLEND SAND	RAP HW 2
16000	100	-	-	100	-	-	-	-
12500	-	-	-	98	100	-	-	98
10000	64	100	-	92	87	100	-	87
5000	33	81	-	69	47	85	-	66
1250	18	41	-	42	24	46	100	45
630	14	33	100	35	19	36	99	38
315	11	26	97	27	14	27	88	23
160	7.9	18.6	28.2	20.2	9.1	19.7	21.8	14.7
80	5.5	14.2	8.6	15.4	5.9	16.0	6.7	10.7
FRACTURES	78 %				84 %			
ABSORPTION	0.53				0.93			
BULK S.G.	2.636				2.570			

TABLE V-2 BINDER RHEOLOGY DATA

ASPHALT	PENETRATION (dmm)	ABS VISC (Pa.s)	KIN VISC mm^2/s	S.G.	ATFOT % LOSS	PEN dmm	ABS VISC Pa.s
120-150A	142	113	320	1.029	0.249	83	257
150-200A	173	87.5	289	1.029	0.284	94	202
200-300A	273	46.4	209	1.023	0.366	140	107
300-400A	373	28.6	172	1.021	0.705	181	76.4
SC3000	too soft	7.3	79	1.020	5.556	247	40.3
Husky PMA	160	*288	725	1.020	0.527	111	582
Imp. PMA	82	*798	874	1.013	0.645	47	1402
RAP-HW 1	90	160	310	-	0.922	57	350
RAP-HW 2	64	268	395	-	0.950	43	610

* The shear rates for the two PMA absolute viscosities were not equal.

TABLE V-3 PREDICTED RHEOLOGY OF RECYCLE SAMPLES

SAMPLE SERIES	DESCRIPTION	r/v	VIRGIN BINDERS	VIRGIN BLEND RHEOLOGY PEN dmm	ABS VISC Pa.s	KIN VISC mm^2/s	PREDICTED MIX RHEOLOGY PEN dmm	ABS VISC Pa.s	KIN VISC mm^2/s
7LG	25/75 (Hard)	28/72	63% (120-150) /27% (150-200)	152	105	310	132	119	310
8LG	25/75 (Med)	28/72	71% (150-200) /29% (200-300)	198	71	261	158	89	272
9LG	25/75 (Soft)	28/72	10% (200-300) /90% (300-400)	365	30	175	247	48	205
10LG	25/75 (Softest)	28/72	67% (300-400) /33% (SC3000)	545	18	134	330	34	170
11LG	50/50 (Hard)	60/40	31% (150-200) /69% (200-300)	240	54	230	132	104	278
12LG	50/50 (Soft)	60/40	SC3000	1400	7.3	79	270	46	180
7LB	15/85 (Hard)	14/86	45% (120-150) /55% (150-200)	160	99	304	140	115	315
8LB	15/85 (Med)	14/86	71% (150-200) /29% (200-300)	200	72	260	170	86	277
9LB	15/85 (Soft)	14/86	32% (200-300) /68% (300-400)	340	33	182	270	44	205
10LB	15/85 (Softest)	14/86	59% (300-400) /41% (SC3000)	522	21	143	390	30	166
11LB	35/65 (Hard)	34/66	71% (150-200) /29% (200-300)	200	72	260	135	112	300
12LB	35/65 (Soft)	34/66	74% (300-400) /26% (SC3000)	540	20	141	261	48	200

This Table shows the percentages of paving grade asphalts which were blended together in the laboratory to obtain the desired consistancy (virgin blend rheology) in order to have the recycled binder in the mix with the desired rheology (predicted mix rheology).

TABLE V-4 MIX DESIGN DATA

SAMPLE SERIES	BINDER	DESIGN DENSITY	DESIGN % A.C.	MARSHALL STABILITY	MARSHALL FLOW	DESIGN AIR VOIDS	DESIGN VMA
1LG	120-150A	2,372.	5.4	9,200.	2.3	4.4	14.6
2LG	150-200A	2,366.	5.4	9,000.	2.2	4.6	14.8
3LG	200-300A	2,370.	5.4	9,700.	2.1	4.4	14.7
4LG	300-400A	2,375.	5.4	7,150.	2.0	4.2	14.5
5LG	Husky PMA	2,371.	5.4	11,000.	2.3	4.2	14.6
6LG	Imp. PMA	2,375.	5.4	10,900.	2.4	4.0	14.5
7LG	eq 120-150	.	.	.	.	.	.
8LG	eq 150-200	.	.	.	.	.	.
9LG	eq 200-300	.	.	.	.	.	.
10LG	eq 300-400	.	.	.	.	.	.
11LG	eq 120-150	.	.	.	.	.	.
12LG	eq 200-300	.	.	.	.	.	.
1LB	120-150A	2,345.	5.9	10,500.	2.5	3.2	13.8
2LB	150-200A	2,344.	5.9	10,150.	2.3	3.2	13.9
3LB	200-300A	2,341.	5.9	8,250.	2.0	3.3	14.0
4LB	300-400A	2,341.	5.9	7,900.	2.1	3.3	14.0
5LB	Husky PMA	2,340.	5.9	12,550.	2.8	3.1	14.0
6LB	Imp. PMA	2,335.	5.9	11,250.	2.9	3.3	14.2
7LB	eq 120-150	2,327.	5.7	11,400.	2.5	3.8	14.3
8LB	eq 150-200	2,334.	5.7	11,500.	2.4	3.5	14.1
9LB	eq 200-300	2,327.	5.7	9,950.	2.3	3.8	14.3
10LB	eq 300-400	2,338.	5.7	9,800.	2.1	3.4	13.9
11LB	eq 120-150	2,350.	5.5	13,600.	2.8	3.3	13.3
12LB	eq 200-300	2,356.	5.5	12,600.	2.5	3.0	13.1

Note: [illegible]o extra Marshall Stability testing was done on the recycle LG series (Goose [illegible]ke Pit) because of a shortage of material.

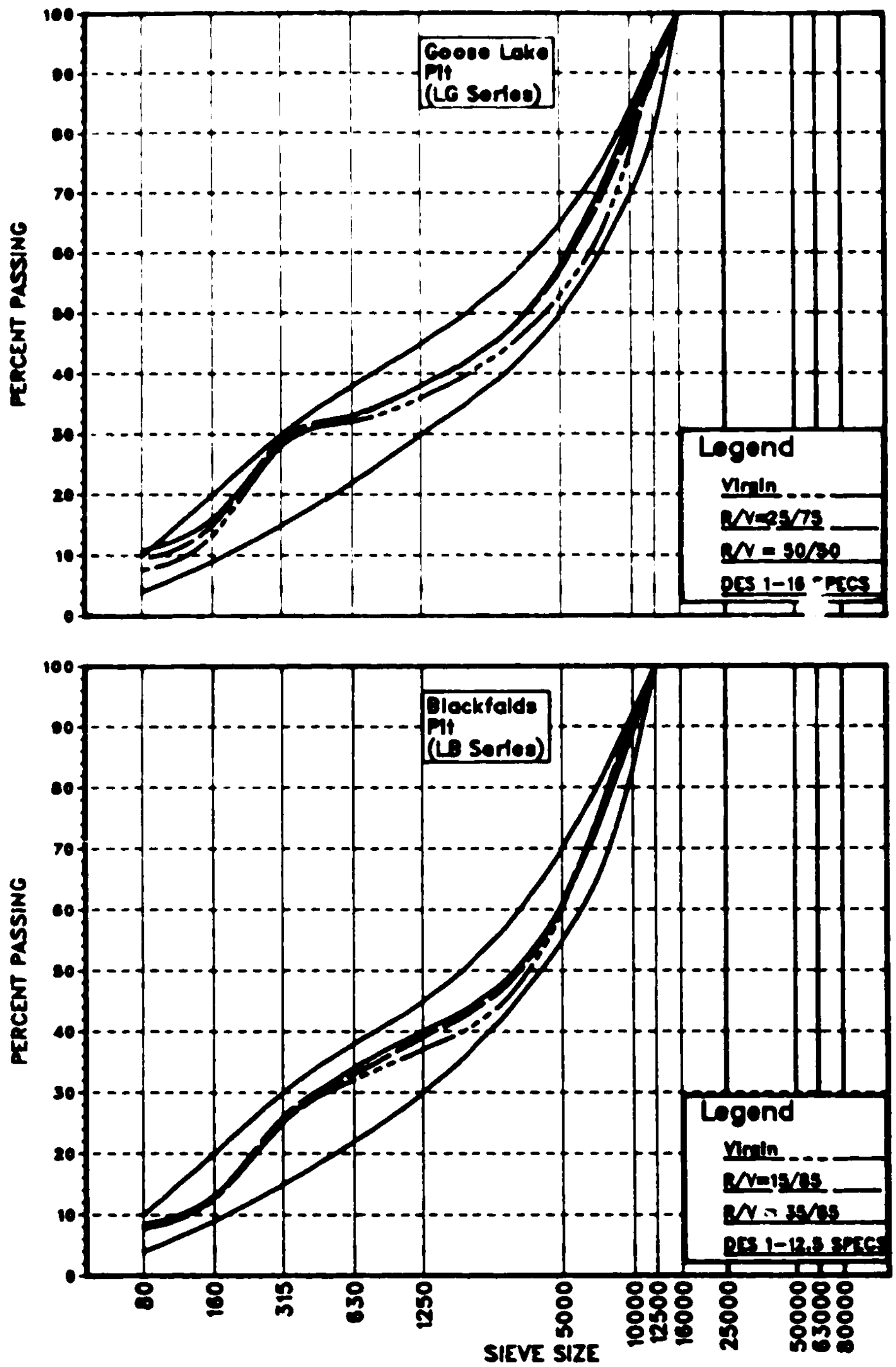


FIGURE V-1 AGGREGATE GRADATIONS (For All Formed Specimens)

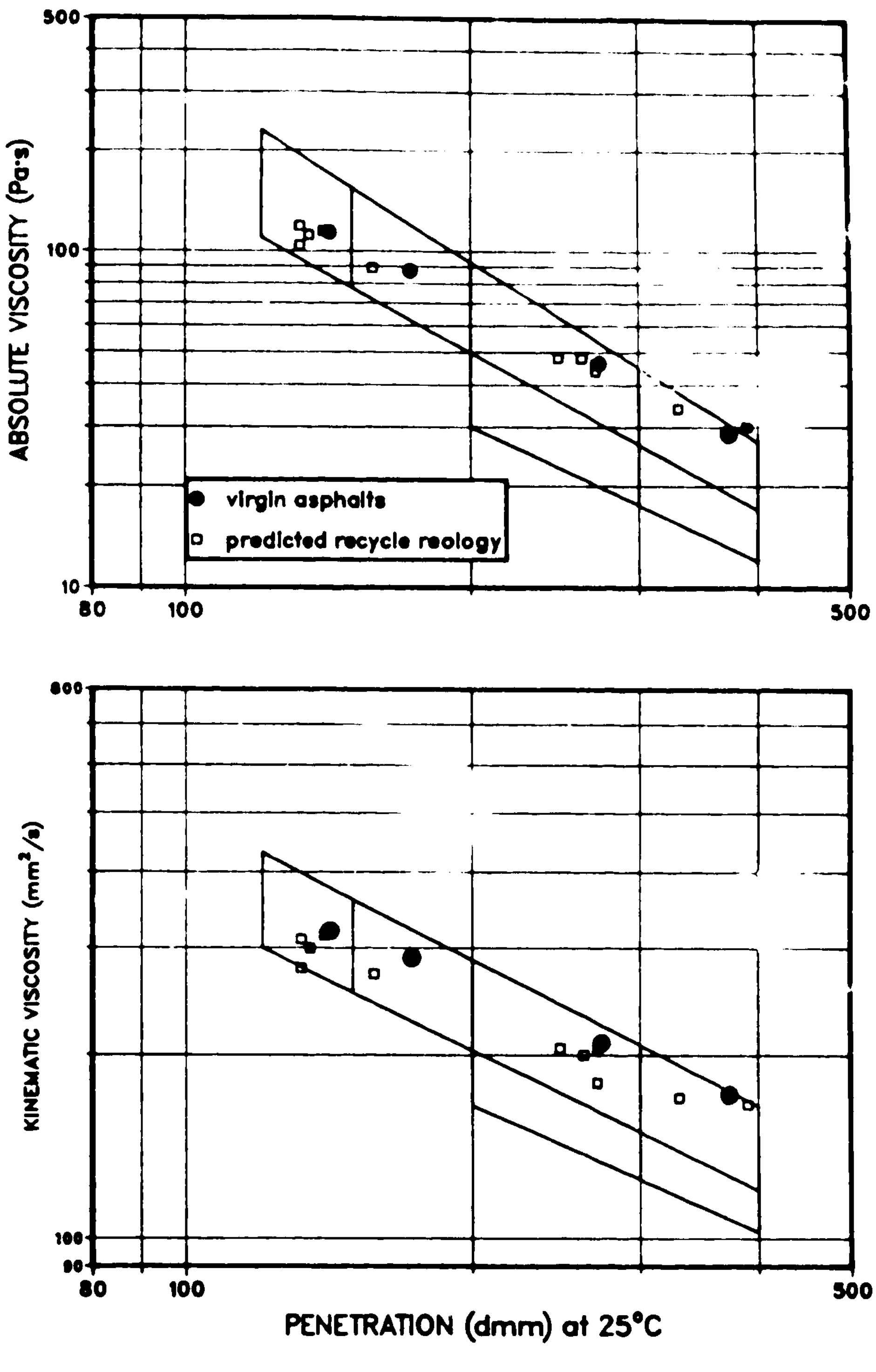


FIGURE V-2 ASPHALT SPECIFICATIONS AND RHEOLOGY RESULTS

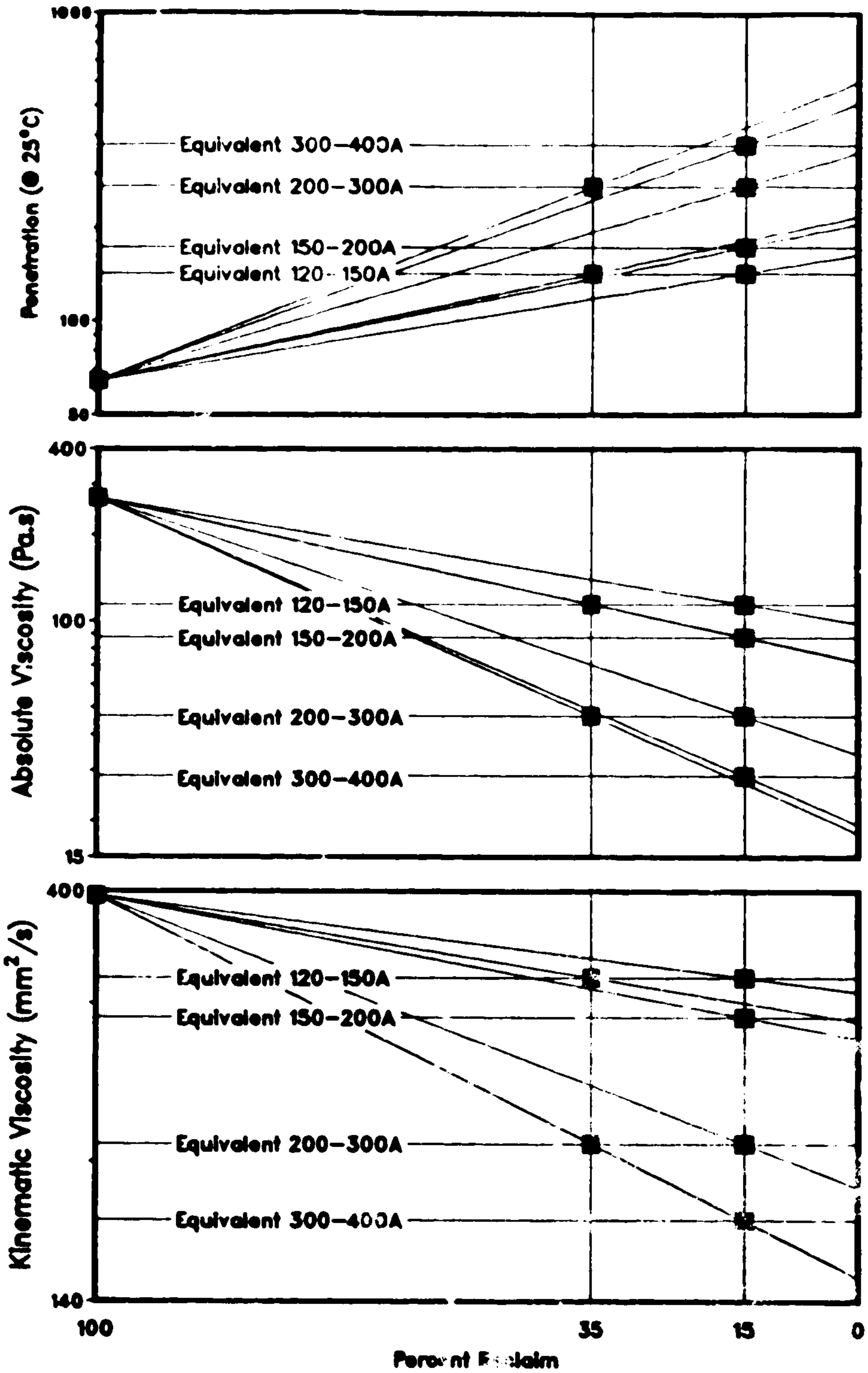


FIGURE V-3 RHEOLOGY DESIGN FOR RECYCLE MIX - GOOSE LAKE PIT (LG Series)

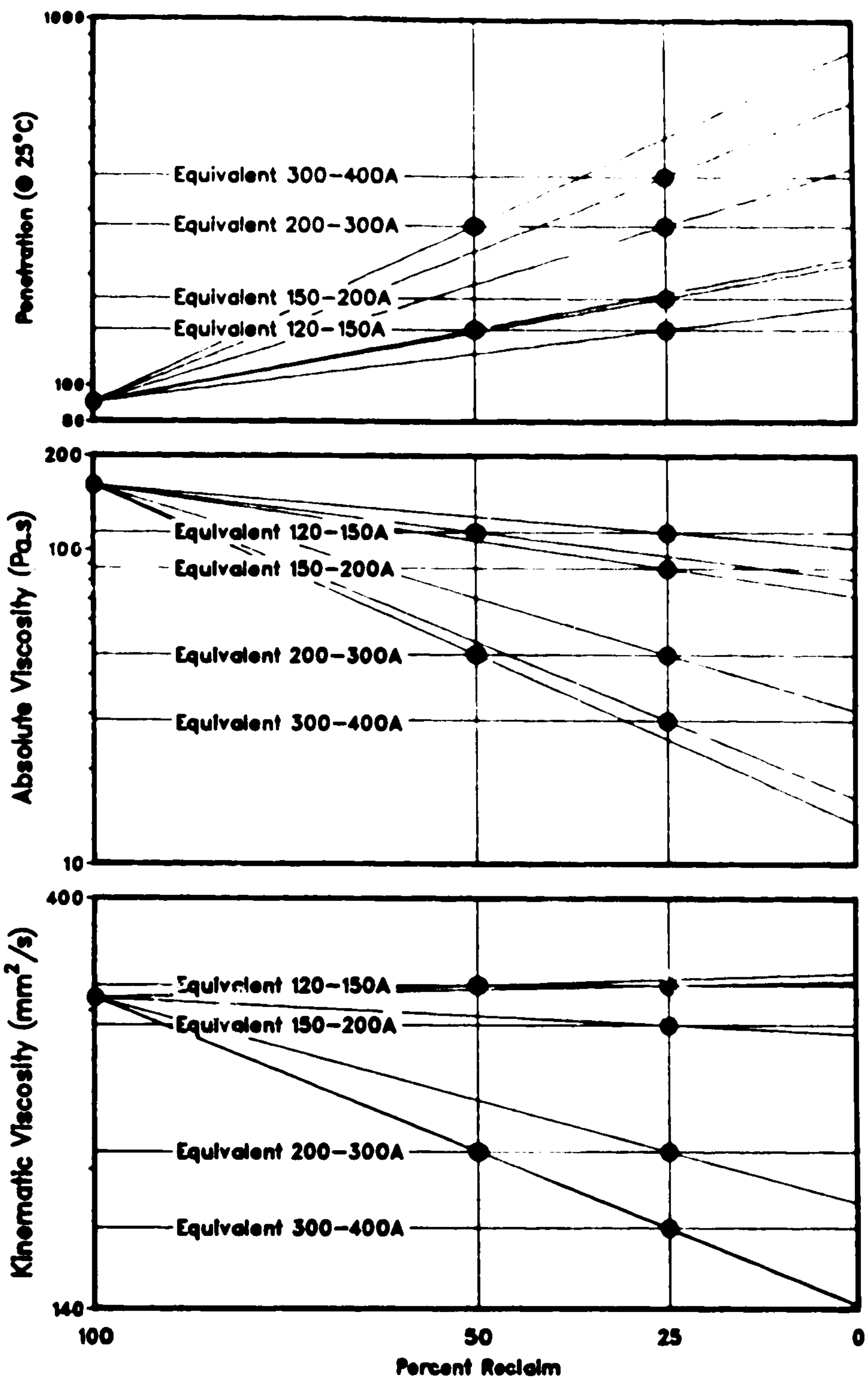


FIGURE V-4 RHEOLOGY DESIGN FOR RECYCLE MIX - BLACKFALDS PIT (LB Series)

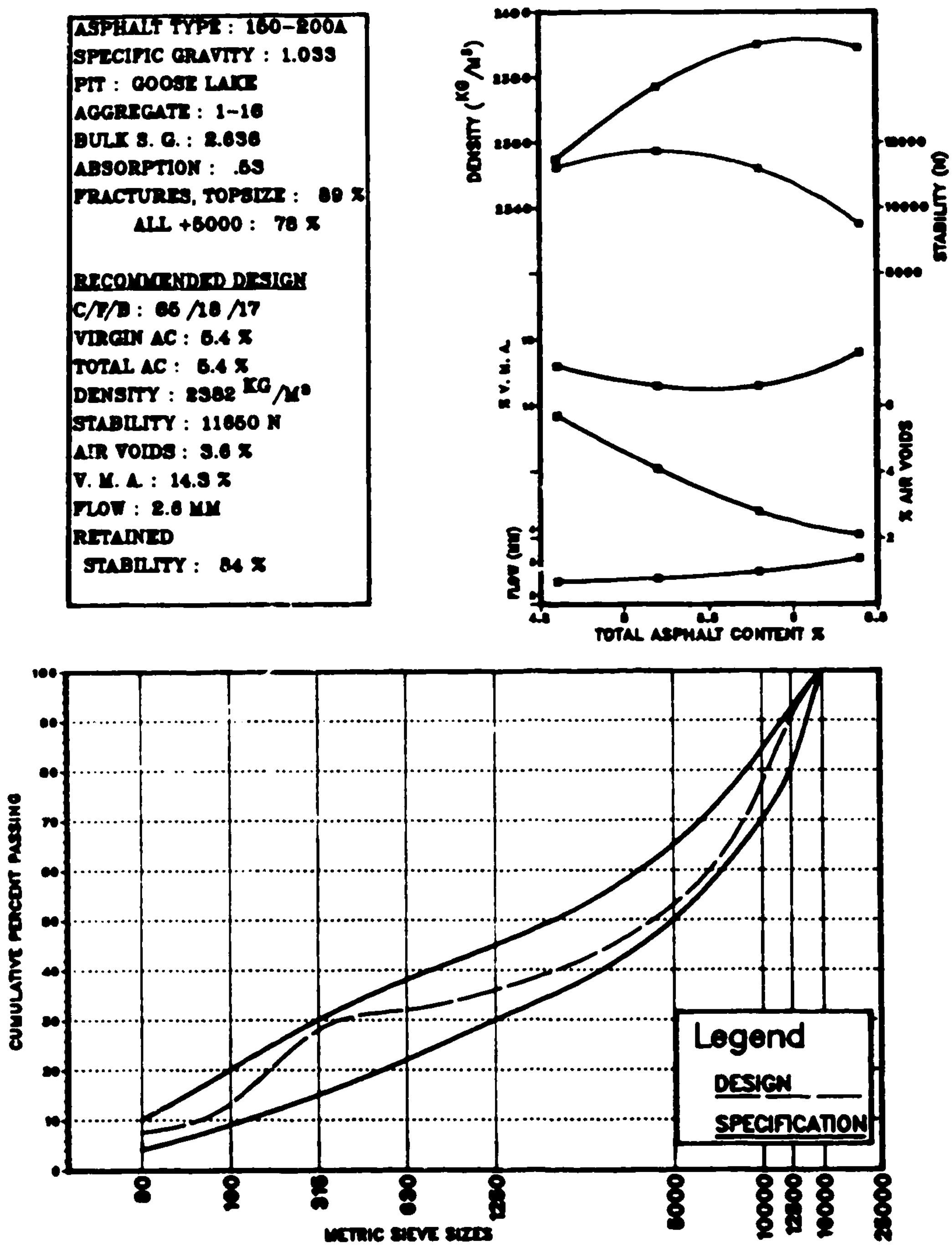


FIGURE V-5 GOOSE LAKE PIT - MARSHALL DESIGN FOR VIRGIN MIX

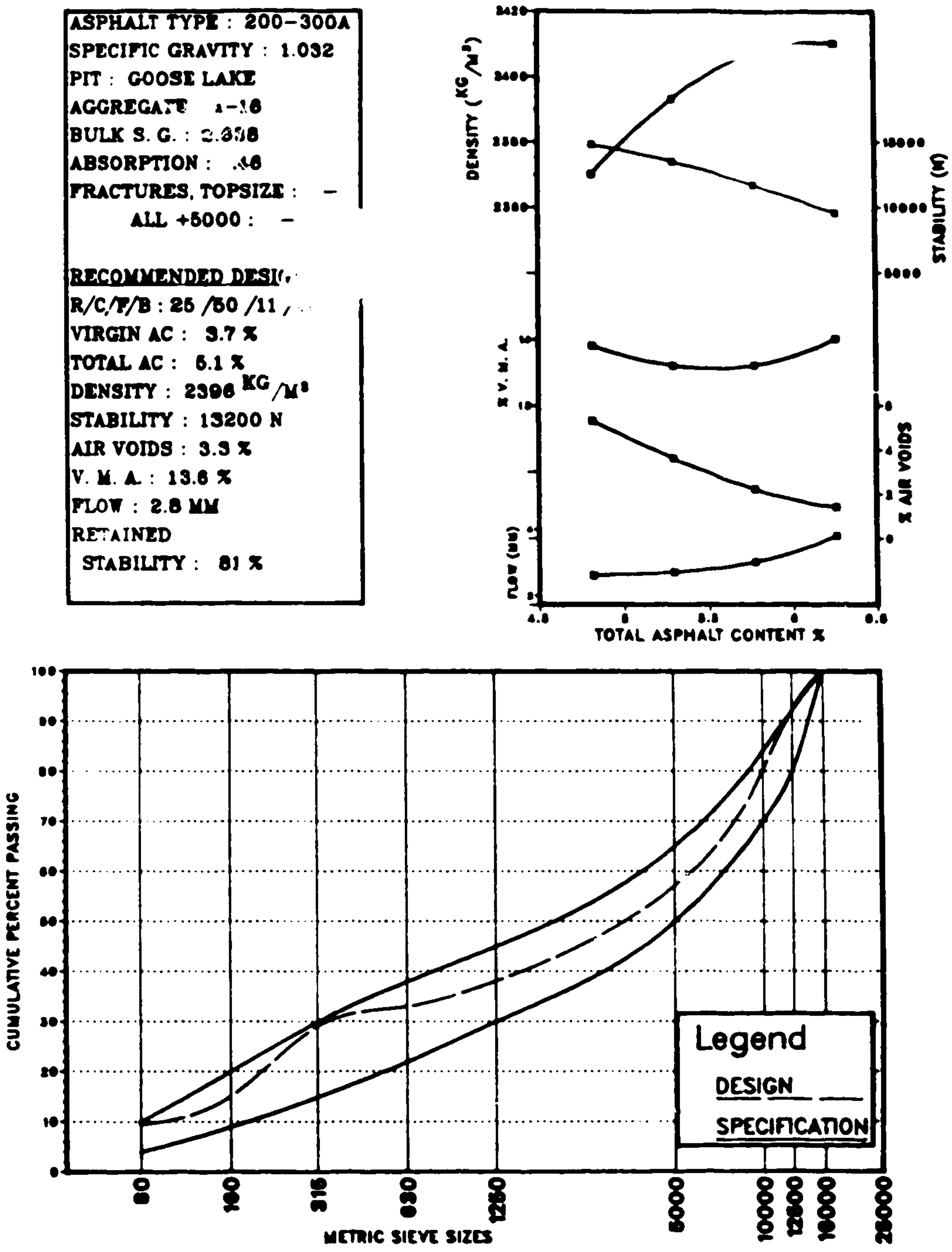
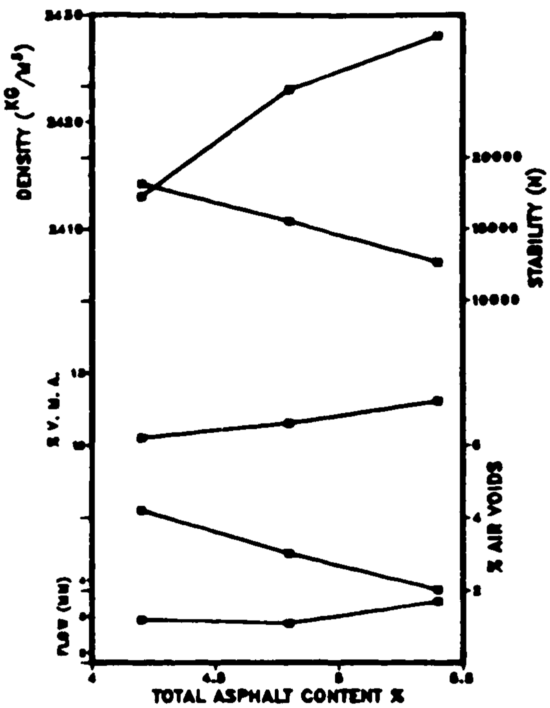


FIGURE V-6 GOOSE LAKE PIT - MARSHALL DES'GN FOR R/V = 25/75

ASPHALT TYPE : 200-300A
SPECIFIC GRAVITY : 1.028
PIT : GOOSE LAKE
AGGREGATE : 1-[illegible]
BULK S. G. : 2.636
ABSORPTION : .66
FRACTURES, TOPSIZE : -
ALL +5000 : -

RECOMMENDED DESIGN
R/C/F/B : 50 /40 / 0 /10
VIRGIN AC : 1.9 %
TOTAL AC : 4.7 %
DENSITY : 2421 $^{KG}/_{M^3}$
STABILITY : 15950 N
AIR VOIDS : 3.2 %
V. M. A. : 12.3 %
FLOW : 2.7 MM
RETAINED
STABILITY :



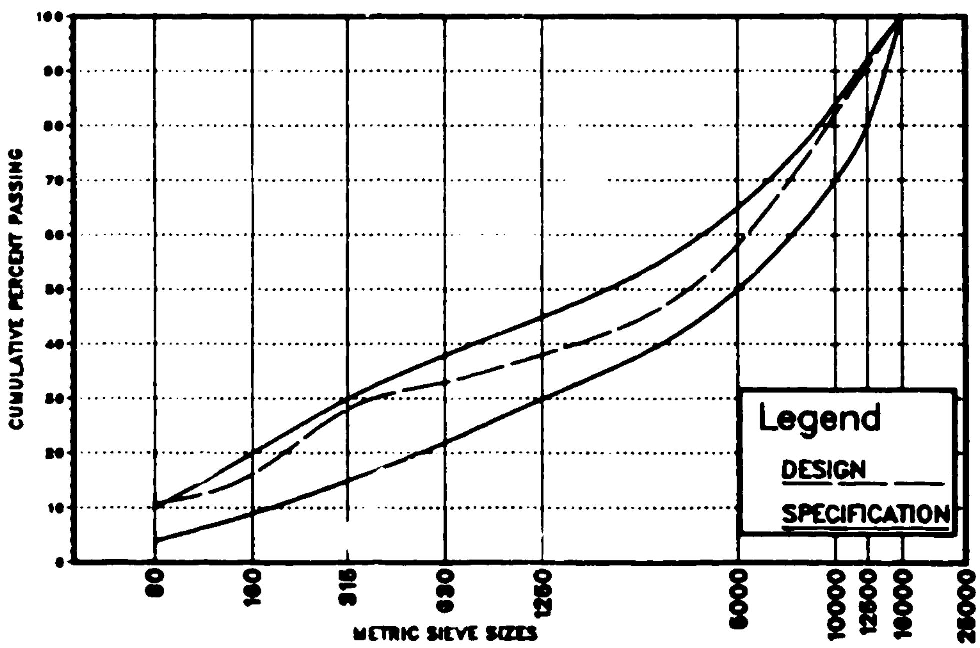
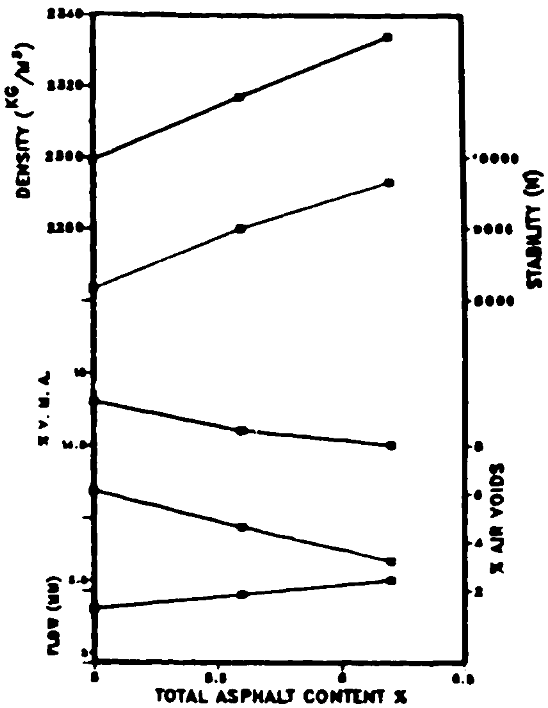


FIGURE V-7 GOOSE LAKE PIT - MARSHALL DESIGN FOR R/V = 50/50

ASPHALT TYPE : 200-300A
SPECIFIC GRAVITY : 1.027
PIT : BLACKFALDS
AGGREGATE : 1-12.5
BULK S. G. : 2.570
ABSORPTION : .93
FRACTURES, TOPSIZE : -
ALL +5000 : 84 %

RECOMMENDED DESIGN
C/F/B : 70 /18 /12
VIRGIN AC : 5.9 %
TOTAL AC : 5.9 %
DENSITY : 2326 KG/M³
STABILITY : 9350 N
AIR VOIDS : 3.9 %
V. M. A. : 14.5 %
FLOW : 2.5 MM
RETAINED
STABILITY : 77 %



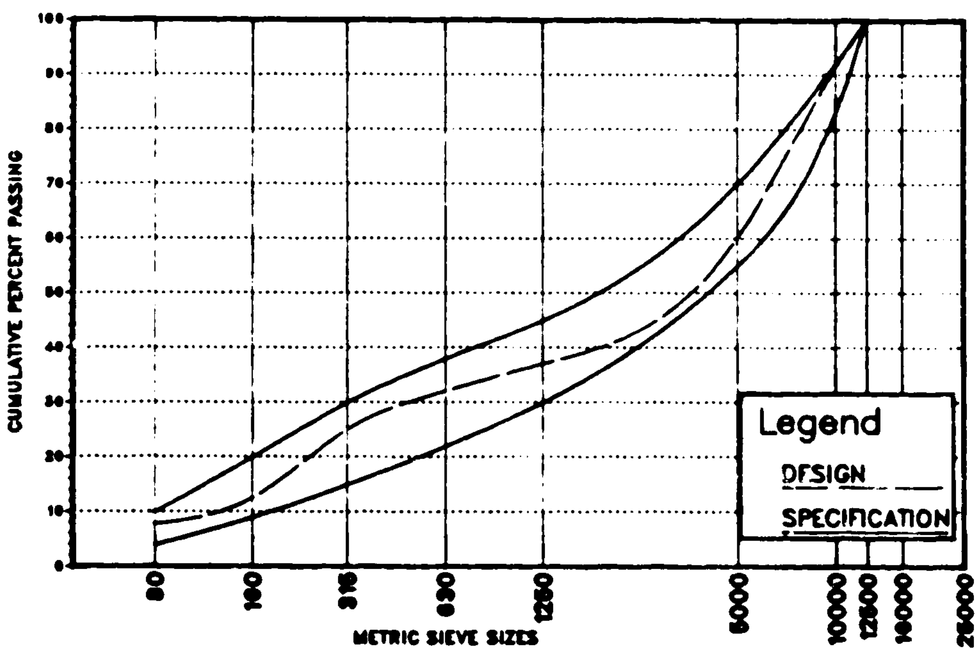
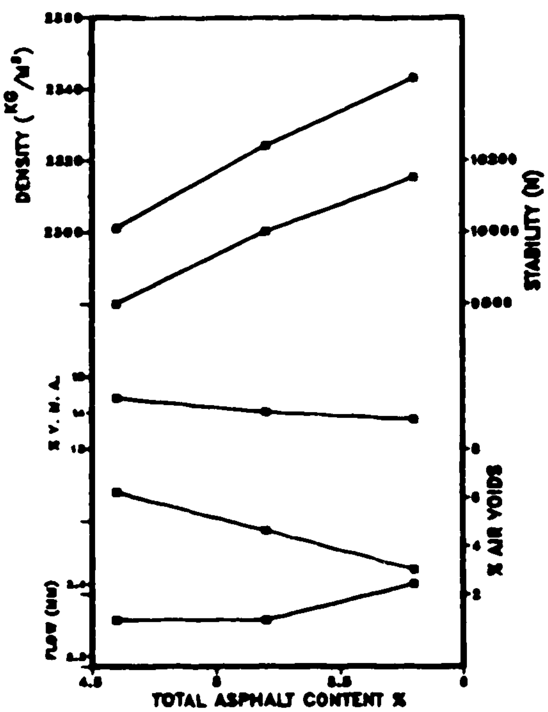


FIGURE V-8 BLACKFALDS PIT - MARSHALL DESIGN FOR VIRGIN MIX

ASPHALT TYPE : 200-300A
SPECIFIC GRAVITY : 1.027
PIT : BLACKFALDS
AGGREGATE : 1-12.5
BULK S. G. : 2.570
ABSORPTION : .79
FRACTURES, TOPSIZE : -
ALL +5000 : -

RECOMMENDED DESIGN
R/C/F/B : 15 /60 /13 /12
VIRGIN AC : 4.9 %
TOTAL AC : 5.7 %
DENSITY : 2340 $^{KG}/_{M^3}$
STABILITY : 10050 N
AIR VOIDS : 3.3 %
V. M. A. : 13.9 %
FLOW : 2.4 MM
RETAINED
STABILITY : 72 %



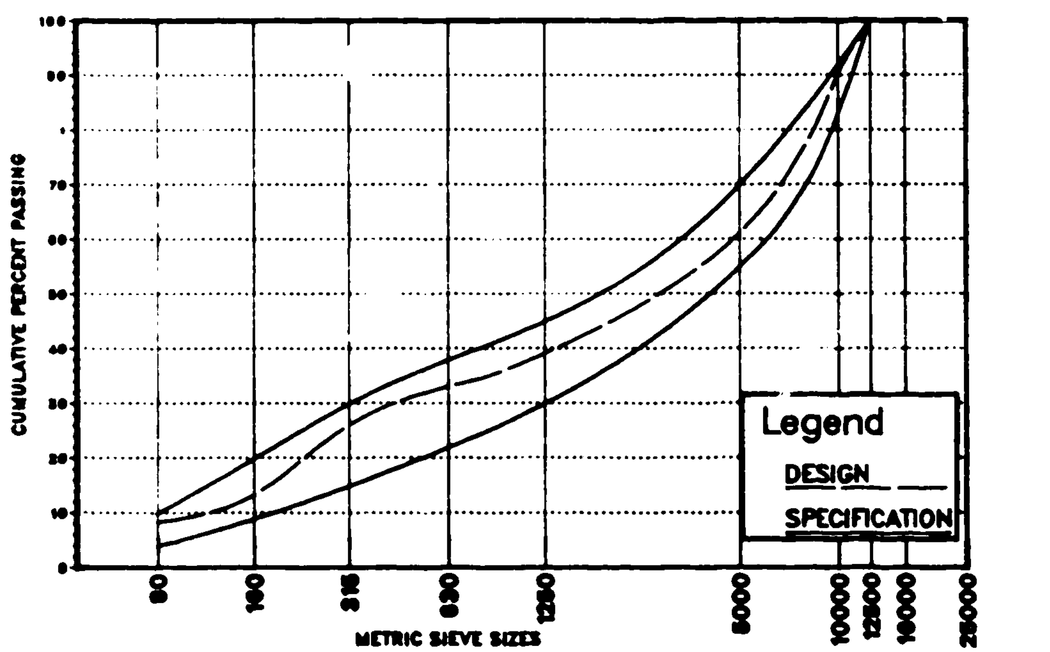


FIGURE V-9 BLACKFALDS PIT - MARSHALL DESIGN FOR R/V = 15/85

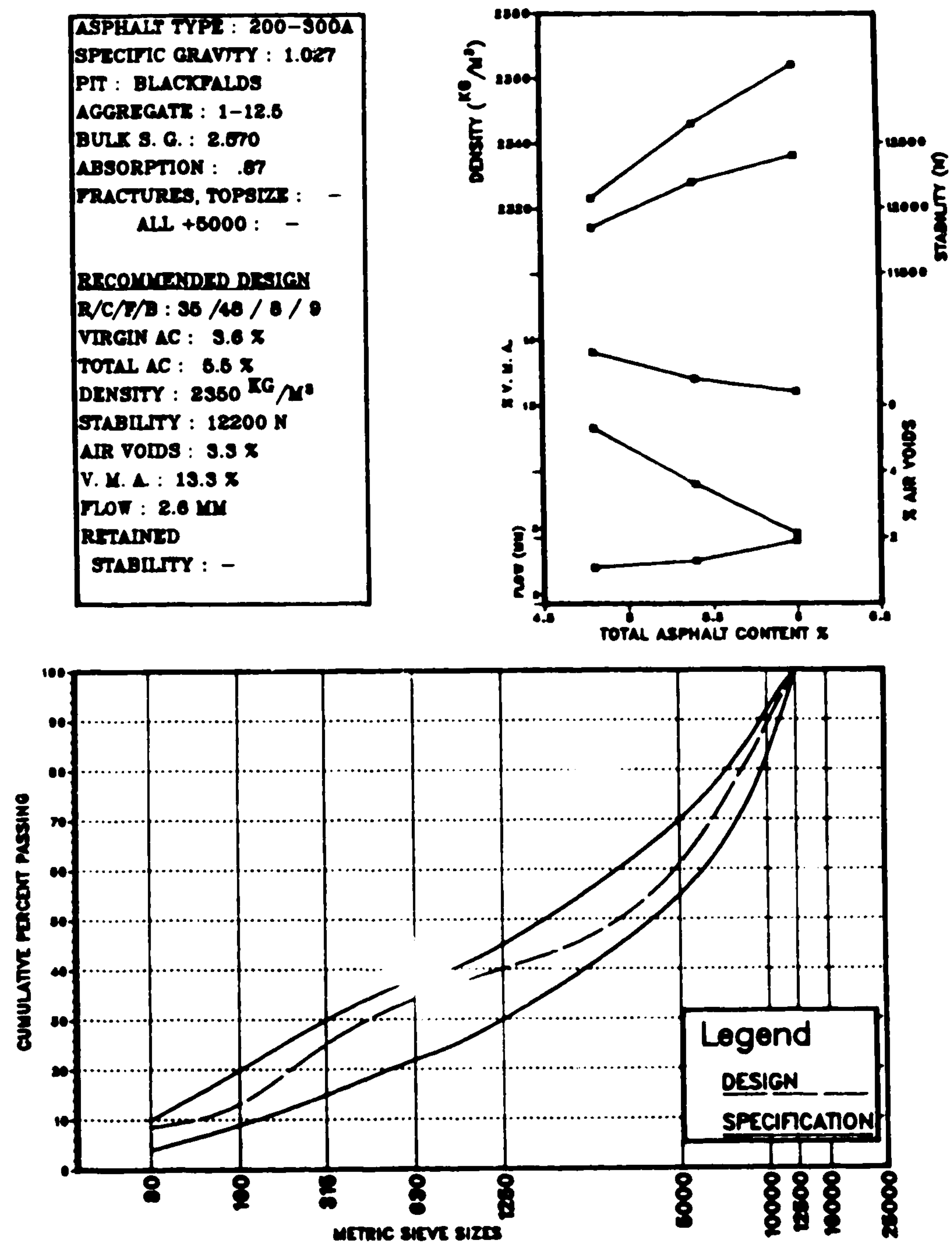


FIGURE V-10 BLACKFALDS PIT - MARSHALL DESIGN FOR R/V = 35/65

CHAPTER VI

STAGE II - RESULTS AND ANALYSIS

6.0 INTRODUCTION

This Chapter presents the results of the repeated load triaxial testing, the results of laboratory testing of the formed test specimens, and the overall analysis of the repeated load test data. The laboratory test results which further characterize the test specimens are reported first, followed by the results of the repeated load tests. The data analysis procedure is then presented, explaining the statistical procedures used to interpret the test results. The actual analysis of the data is then given, followed by a discussion of the results of the analysis.

6.1 Results

6.1.1 Materials Characterization

Samples from all of the various mixes were tested in the laboratory to further characterize their material properties. Prior to being subjected to the repeated load testing, densities of all samples were measured and the air voids calculated. Following the repeated load tests selected samples were tested to determine the current asphalt rheology of the asphalt recovered using the Abson procedure. The results of the rheology testing on the Abson recovered asphalt cements are given in Table VI-1. The results show the 12.5 mm topsize aggregate from the Blackfalds Pit (LB series) has consistently softer characteristics than the samples formed from the 16 mm topsize aggregate from the Goose Lake Pit (LG series). These differences in rheology between 'equivalent' grade asphalts for the two aggregate sources vary from minor (Samples 5LG and 5LB) to significant (Samples 10LG and 10LB). While no specific cause has been attributed to these differences they must be conside..d when the permanent deformation data (recorded as percent permanent strain) from the repeated load triaxial testing is examined.

The densities determined for the formed samples, the calculated air voids and the calcul...ed binder stiffnesses are reported in Table VI-2. The reported densities and voids can be c npared to the m x designs reported in Chapter V, which indicate that all sample series are near tre design values. The binder stiffne ses reported in Table VI-2 were calculated using McLeod's method of determ temperature susceptibility and using his modification to Heukelom's and Klomp's version of van der Poel's nomograph for determining modulus of stiffness of asphalt cements[4].

6.1.2 Repeated Load Test Results

The eated load triaxial testing measured the permanent strain for each sample at ous repetitions of the axial load. Results from each individual sample were recorded for the analysis of the data. For convenience, average strain values are reported in this Chapter.

Each sample series (ie. 1LG, 2LG, etc.) represent mixes with the same characteristics. Repeat tests were done for all series to allow for outlier test results and to allow the calculation of the testing error at the analysis stage. Figure VI-1 shows an example set of results for a series of tests (repeats) conducted at a single temperature. This figure gives an indication of the variance between the tests conducted or nples of the same mix. Samples which exhibited unusually large strains for no apparent reason (outliers) were removed prior to the analysis stage. Figures VI-2 to VI-5 show plots for the average strain results for each series of tests at each of the three test temperatures. The numeric strain data for the plots presented are given in Tables VI-3 to VI-7. Examination of the strain data shows that strains varied from about 0.5% at 90000 load repetitions for the 6LG series (PMA) at 25°C to about 9% for the 4LB series (300-400A asphalt grade) at the same conditions. At the higher temperature of 45°C and at only 20000 load pulses the strains were about 1.2 and 11% respectively for the same two mixes.

Examination of the strain curves also show the LB series of mixes consistently underwent more strain than the LG series. While this at first may appear to be a function of the aggregates, the differences noticed in the Abson rheology test must be noted.

An overview of the results show that the strains increased markedly as the temperatures increased. The temperature increases can be seen to have significant affect on the calculated binder stiffness shown in Table VI-2. As well the observed strain generally increased as the binder stiffness decreased between grades, however this was not observed in all cases. Specifically the LB series virgin asphalt grades at the 35° and 45°C test temperatures did not consistently show increases corresponding to the softer asphalt grades. The recycle mixes can be seen to have strained somewhat less than the virgin samples while the PMA mixes underwent the least strain.

6.2 Analysis

6.2.1 Approach

The analysis portion of the repeated load triaxial testing program was aimed at meeting the six objectives, stated in Chapter V. In order to examine the various parameters involved, all valid strain data were downloaded to the Alberta Government's mainframe IBM computer system and subsequently loaded into a database utilizing the FOCUS "Information Control System"; details of the data handling using the FOCUS software are explained in detail in Appendix E.

The FOCUS software includes a statistical analysis package which was used to determine numerous statistics required to interpret the data. The database was constructed with measures of the binder rheology and stiffness, aggregate topsize, design and mix characteristics. Figure VI-6 shows the variables included in the data base.

The objective of the analysis was to identify those parameters with a significant influence on the strain observed in the laboratory. The approach taken involved the development of empirical models to explain the observed strain in terms of the various mix parameters. Regression analysis was used to build the empirical models.

One of the assumptions of the method of least-squares is that the observations have equal variance at each level. Examination of Figure VI-1 suggests that the variance of the strain observations are not constant at each level, but rather would tend to increase as strain increased. The variance was checked by examining the replicated tests and this showed showed significant changes in variance as the strain increased. Since the least squares procedure requires constant variance, a transformation of the dependant variable is necessary to stabilize variance.

Other researchers[15, 16] have successfully used logarithmic transformations with similar data to stabilize the variance and this transformation was conducted for this study. The resulting variance was improved, however it was still non-constant. A graphical method which plots the log of the estimated standard deviation (S) against the log of the dependant variable (strain) was used to find an appropriate transformation[22]. The graphical method assumes a power relation between the standard deviation, σ, and the average strain value, the slope of the plotted line then tells the value of the power for such a relationship.

Using this method a straight line plot was obtained for strains above 1.6%, or 1500 load repetitions for the data examined. At lower strains the variance of the data was already relatively constant. The slope of the plot was calculated at $1.86 \cong 2$ which corresponds to an inverse transformation for stabilization of the variance.

The data was then re-examined using only strain values for load pulses greater than 1500. The 1/strain transformation applied to the strain data resulted in constant variance (probability ≤ 0.05). For the same data range the log(strain) transformation gave constant variance at the 1% level (probability ≤ 0.01) but not at the 5% level.

All model forms were checked for L.O.F. by examining the pure error and lack-of-fit components of the residual mean squares[26]. The pure error was estimated from the replicated trials.

A further check for lack-of-fit of the chosen model was conducted by examining plots of the residuals. The residuals plotted against the predicted variable or any of the component variables of the model should appear as a random scatter of points. Any trends in the residual plots suggest dependence and inadequacy of the chosen model[26].

6.2.2 Model Development

Following the approach presented in the previous section models were developed to explain the strain observed in the repeated load triaxial testing.

Each of the three binder types (virgin asphalt, recycled asphalt and polymer modified asphalt) were evaluated separately. For each of the different mix types evaluated, a measure of the binder stiffness was found to be the single most significant variable contributing to the observed strain. This was found to be true for the relationships for both the log and inverse transformations of the strain.

The previous work of both Hadipour[15] and Morris[16] support the use of the log transformation as the dependent variable. The work presented by Hadipour does not directly discuss the the question of variance of the dependant variable at different levels. However Morris's work discusses the stability of the variance and concludes that the log transformation results in the best form of the dependant variable. It was felt to be more desirable to use the log transformation and so the significance of the two transformations was examined from an engineering viewpoint. On the one hand it was considered that the calculated statistics indicated that the inverse transformation of the strains was the best transformation to stabilize the variance. Conversely, because previous work of this nature had shown acceptable results it was felt that there was no reason to believe that the variance was not constant, though a low probability had been calculated. Based on these considerations it was decided to use the log transformation for the model development.

The models determined for each of the binder types are presented in the following sections, as are the models developed for the combined binder types. Each section discusses the relationships suggested by the proposed models.

6.2.3 Conventional Asphalt Series Model

The best fitting model determined for the virgin asphalt samples (series 1LG-4LG, 1LB-4LB) was the following:

$$\text{Log}(\epsilon) = -.9521 + 0.5851 * \log(N) - 0.1079 * \log(N) * \log(\text{Abson Stiffness}) \quad \text{(VI-I)}$$

where ϵ = permanent strain (%)
N = number of load applications
Abson stiffness = calculated stiffness of Abson recovered binder

This model is presented as the best model determined for the data, however some L.O.F. is still present. Table VI-8 presents the ANOVA for the model. The r^2 value of 0.8732 for this model suggests a relatively good fit, and when the pure error is taken into consideration the r^2 can be seen to be 93% of the maximum r^2 possible. The F value for this model is 1.67 indicating L.O.F. (Probability $\leq$ 0.05). Examination of residual plots did not indicate the model deficiencies.

Possibilities to explain this statistical L.O.F. of the model are numerous. Variations exist between the Abson rheology of single sample series which were not accounted for, variations of mix design characteristics between samples, variations in stress levels due to 'barreling' of the cylindrical samples which were not measured, end effects and tilting, as well as unmeasurable differences in aggregate between samples.

Though variations caused by such influences would be considered when examining the pure error of the testing, it is possible that such factors would be required to obtain a better model fit. As well, aggregate effects were examined in terms of topsize and gradation. However other means of quantifying the aggregate may be necessary to properly account for its influence.

Based on the preceding discussion the obtained model fit appears to be reasonable, and while it is recognized that the model does not explain all observed strains, it identifies a very meaningful relationship.

The model presented shows the strain to be dependant on the number of load applications, and on the stiffness of the Abson recovered asphalt. The stiffness value can be seen to act with the number of load repetitions. Both terms appear reasonable based on mechanistic considerations. Their inclusion can be seen to be valid, based on the significant 't' statistics shown for the respective coefficients.

Using a load repetition of 80000, and a temperature of 25°C, reductions in the predicted strain of 11, 37 and 53% between adjacent asphalt grades can be calculated. The percent reductions are the greatest between 300-400A and

200-300A and the least between 150-200A and 120-150A grade asphalts. The values determined from the model give a clear indication of the relative effects of adjacent asphalt grades.

6.2.4 Recycled Asphalt Model

Using the recycled asphalt binder series (7LG ...G, 7LB-12LB) the following model was determined.

$$\text{Log}(\epsilon) = -0.9519 + 0.5634 \log(N) \qquad \text{(VI-11)}$$

$$-0.7652 * \log(N) * \log(\text{Abson stiffness})$$

$$-0.0295 * \log(N) * \log(\text{Abson Stiffness}) * \log(\%\ \text{RAP})$$

where ϵ = permanent strain (%)
N = number of load applications
Abson stiffness = calculated stiffness of Abson recovered binder
% RAP = percentage of reclaimed asphalt pavement in mix.

As with the model for the virgin mixes, the r^2 of 0.8313 suggests a good fit and again represents 93% of the fit possible based on the pure error calculated. The ANOVA for this model are given in Table VI-9.

The F value has been calculated to be 2.99, suggesting that the model has a significant amount of L.O.F. (Probability ≤ 0.05).

The considerations discussed in the previous section are also valid for the recycle mixes. As well the recycled mixes have the additional complication of an additional variable, the reclaim content. While the inclusion of the percent reclaim in the model is supported by the previous work of Hadipour[15] and by the statistics of the analysis conducted for this portion of the study, the mechanistic influence of the reclaim content is not clear. The following paragraphs discuss the practical significance of the percent reclaim (% RAP).

One of the assumptions when designing recycled mixes is that the reclaimed asphalt binder and the new binder used as a rejuvenator will flux together during mixing and behave as a homogeneous binder. If this total fluxing does not occur during mixing, then it could be expected that the characteristics of the softer material would influence the overall mix stiffness. However since the analysis is accounting for the measured rheological properties (though not necessarily representative of the physical behaviour of the mix) the % RAP would have to influence the mix in some alternate way.

The possibility that the % RAP is not a causation variable must be examined. Table VI-10 shows a correlation matrix with the % RAP and the binder rheological properties. Of note are the correlations of the % RAP to the Penetration and Viscosities of the designed binder compared to the Abson recovered binder. The % RAP is significantly correlated to the Abson recovered asphalt properties, but not to the designed binder properties. This observation raises the same question of cause and effect; however the review provides insight to the original question of whether the % RAP has a real effect on the mix behaviour.

The recycle mixes for this study were designed to have resultant rheological properties similar to the various virgin asphalts utilized, hence since all ratios were targetted to at least two resultant grades, no correlation is expected or possible. After mixing, the aging which occurred, as measured on the Abson recovered asphalt cements, indicates a correlation to the % RAP in the mix. Previous work[24] conducted on numerous field mixes specimens suggested no correlation of % RAP to the resultant asphalt rheology.

Based on the preceding discussion it can be concluded that the causation question is important to the understanding of fluxing of asphalt cements during recycling, but not necessarily for this study. The fact that there is a correlation between the % RAP and the rheology of the Abson recovered asphalt suggests that the apparent correlation of strain to % RAP is caused by the resulting rheology; whether the resulting rheology is actually a result of the % RAP will require further evaluation.

Overall then, it is not clear specifically how the reclaim material contributes to the observed strain. However, until a cause and effect can be determined the reclaim content is the best measure available. Examining the presented model, the influences of the binder stiffness and of the reclaim content can be examined.

Using the model determined, calculations show that strains will be reduced in the order of 30-40% with the change from a 15% reclaim content to a 50% content. Alternatively reductions in strain are indicated at about 60% for rheology properties equivalent to a 120-150A asphalt cement as compared to a 300-400A grade binder. These calculations show both the RAP content and the binder stiffness exhibit significant affects on the predicted strain.

6.2.5 PMA Model

The third binder type, Polymer modified asphalt, included sample series 5LG, 6LG, 5LB and 6LB. The model developed for these series differed from the previous two models in that the stiffness measurement which best explained the observed strain was that calculated based on the original rheology. The previous two models included stiffness calculated based on the rheology of the Abson recovered asphalt. The model describing the behaviour of the PMA mixes was:

$$\text{Log}(\epsilon) = 0.6323 + .3618*\log(N) - 0.0979*\log(N)*\log(\text{stiffness}) \quad \text{(VI-III)}$$

where ϵ = permanent strain (%)
N = number of load applications
Stiffness = calculated stiffness of original binder

This model has an r^2 value of 0.8794 which is over 94% of the possible r^2 when the pure error is considered. The ANOVA for this model is given in Table VI-II.

As indicated by the r^2 value, this model represents a relatively good fit. However the calcula[illegible] statistic is [illegible].28 which indicates L.O.F. (Probability $\leq$ 0.05) for the model. However, the fact that this model, for the PMA mixes, exhibits the best fit of the three mix types may be significant. It suggests that as the binder stiffnesses change (increases) substantially, other influences become relatively less significant. This would then suggest that for all three models there is still some influence which is not being accounted for, and this influence is less for the PMA mixes.

The model determined shows the influence of the binder calculated stiffness on the observed strain. This is significant because it suggests that conventional binder rheology tests, which are used for conventional asphalts, also can be used to evaluate a PMA binder. This then allows the comparison of the PMA rheology to virgin asphalt rheology and as this data would indicate the observed strains are significantly lower for the PMA mixes.

6.2.6 Combined Models

In order to evaluate the relative influences of the different binder types, analysis was conducted using data from two mix types together. The virgin samples were treated as the standard mix. The same model form determined for the individual analysis was also found to be the best fit obtainable for the combined data.

Virgin/Recycle Model

Using both the virgin and recycle mix test results the following model was developed:

$$\begin{aligned} Log(\epsilon) = {} & -0.9494 + 0.5766*Log(N) \\ & - 0.1041*Log(N)*Log(Abson\ Stiffness) \\ & - 0.0135*Log(N)*Log(Abson\ Stiffness)*Log(\%\ RAP) \end{aligned} \qquad (VI-IV)$$

where
ϵ = permanent strain (%)
N = number of load applications
Abson Stiffness = calculated stiffness of Abson recovered binder
% RAP = percent reclaimed asphalt pavement in mix.

This can be seen to be of the same form as the model developed for the recycle mix alone. The ANOVA for this model is given in Table VI-12. The r^2 value of 0.8655 is 91% of the theoretical maximum r^2 The calculated F value is 2.6, which shows a L.O.F. for this model (Probability ≤ 0.05).

The significance of the combined model given here is that it indicates similar behaviour between the two different mix types, dependant on the stiffness value and the reclaim content. The inclusion of the reclaim content in the model does show that there is a different response expected for recycled mixes.

Virgin/PMA Model

The combined data from the virgin and PMA mixes was used to attempt to describe the relative strain characteristics of the two types of binders. The model developed was:

$$\mathrm{Log}(\epsilon) = -0.9119 + 0.4885 * \text{[illegible]} \qquad \text{(VI-V)}$$

$$- 0.1\text{[illegible]}75 * \mathrm{Log}(N) * \mathrm{Log}(\text{S[illegible]s}) - 0.1568 * \mathrm{Dummy}$$

where ϵ [illegible] strain (%)
N = [illegible]r of load applications
Stiffness = calculated stiffness of original asphalt
Dummy = dummy variable, -1 for virgin mixes, +1 for PMA mixes.

The ANOVA for this model are given in Table VI-13. The r^2 of 0.9000 for this model is 94% of the maximum possible. The F value was calculated as 2.48, indicating L.O.F. for this model (probability ≤ 0.05).

The need for the dummy variable in this model is very significant. It shows that though both mixes are greatly influenced by the binder stiffness, there is some fundamental differences in these influences. This difference may be a reflection of the method used to calculate the binder stiffness, which was developed for conventional asphalt cements. If this is the case the use of experimentally determined stiffnesses may remove the need for the dummy variable.

6.3 Stage II - Discussion

The results of the analysis of the repeated load testing examined in this Chapter show binder stiffness as the significant variable in predicting the observed strain. The analysis also showed that the RAP content played a role in the mix deformation characteristics, though the mechanics of this role were not quantified. While models were determined which could explain all of the observed variations, significant results were obtained. Overall the binder stiffness was shown to greatly influence the observed strain of the asphalt concrete samples. Because this was found for all mix types it allows comparisons to be made on the basis of the rheological properties of the different binders.

The testing has generally shown that harder asphalts will strain less, though for the virgin mixes the absolute value of these differences may not be that significant from a practical viewpoint. More significant differences in strain values were observed when comparing the virgin specimen's strains to those experienced by the recycle mixes and the PMA mixes. Based on this testing it can be seen that a change to a recycle mix or to a PMA mix would result in significant changes in strain as opposed to changes to harder asphalt grades in the normal range. Both the recycle and PMA mixes exhibited significantly stiffer rheologies, especially the higher recycle ratios (11 and 12 series). The relative effect of the binder types to strain was shown to be the virgin mixes as least resistant, with recycle mixes and PMA mixes exhibiting increasingly more resistance to permanent deformation.

TABLE VI-1 ABSON RECOVERED BINDER RHEOLOGY DATA

SERIES	PENETRATION (dmm, 25°C, 5s)	ABSOLUTE VISCOSITY (Pa.s, 60°C)	KINEMATIC VISCOSITY (mm^2/s, 135°C)
1LG	78	358	534
2LG	82	30[illegible]	483
3LG	133	137	330
4LG	188	85.6	261
5LG	111	761	1272
6LG	46	[illegible]523	2095
7LG	54	647	640
8LG	64	537	582
9LG	103	188	364
10LG	103	445	495
11LG	25	19343	3407
12LG	50	1613	780
1LB	89	281	4[illegible]2
2LB	10[illegible]	217	[illegible]
3LB	182	95.2	282
4LB	221	67.4	235
5LB	117	643	1247
6LB	84	1352	1205
7LB	76	385	524
8LB	87	386	521
9LB	138	134	342
10LB	164	104	278
11LB	51	848	742
12LB	102	203	369

TABLE VI-2 MIX CHARACTERISTICS AND STIFFNESS DATA

Series	Density (Kg/m³)	Air Voids (%)	STIFFNESS (KPa) Virgin Binder 25°C	35°C	45°C	Abson Binder 25°C	35°C	45°C
1LG	2392	3.6	392.0	58.9	15.0	1079.0	245.3	54.0
2LG	2395	3.5	294.3	49.1	14.7	981.0	196.2	40.2
3LG	2401	3.3	117.7	34.3	7.8	490.5	73.6	19.6
4LG	2389	3.8	68.7	-	4.7	225.6	-	16.7
5LG	2391	3.5	294.3	-	19.6	686.7	-	46.1
6LG	2[illegible]7	4.2	882.9	-	49.1	3237.3	-	176.6
7LG	2387	4.0	392.4	58.9	14.8	1962.0	392.4	78.5
8LG	2383	4.0	294.3	49.1	14.7	1863.9	294.3	49.1
9LG	2388	3.0	117.7	29.4	8.8	549.4	107.9	26.5
10LG	2376	4.5	981.0	-	5.9	608.2	-	29.4
[illegible]LG	2374	5.3	-	58.9	-	-	1962.	-
[illegible]G	2389	4.4	-	29.4	-	-	490.5	-
1LB	2335	4.0	392.0	58.9	15.0	981.0	166.8	39.2
2LB	2330	4.1	294.3	49.1	14.7	686.7	117.7	27.5
3LB	2347	3.6	117.7	34.3	7.8	206.0	49.1	10.8
4LB	2337	4.0	68.7	-	4.7	166.8	-	10.8
5LB	2335	4.1	294.3	-	19.6	490.5	-	29.4
6LB	2335	4.0	882.9	-	49.1	941.8	-	50.0
7LB	2334	3.7	392.4	58.9	15.0	1177.2	215.8	98.1
8LB	2333	3.7	294.3	49.1	14.7	882.9	147.2	37.3
9L	2340	3.6	981.0	24.5	6.9	392.4	73.6	17.7
10LB	2330	4.0	49.0	-	4.9	294.3	-	11.8
11LB	2348	3.4	-	63.8	-	-	490.5	-
12LB	2346	3.5	-	24.5	-	-	117.7	-

TABLE VI-3 PERCENT PERMANENT STRAIN - VIRGIN LG SERIES

Temperature	N	1LG	2LG	3LG	4LG
25	10	0.177 (0.02)	0.206 (0.07)	0.287 (0.15)	0.501 (0.36)
	50	0.317 (0.06)	0.394 (0.08)	0.526 (0.18)	0.701 (0.41)
	100	0.394 (0.06)	0.485 (0.08)	0.649 (0.18)	0.797 (0.43)
	1000	0.672 (0.05)	0.844 (0.05)	1.160 (0.16)	1.225 (0.45)
	5000	0.918 (0.06)	1.201 (0.04)	1.694 (0.10)	1.791 (0.39)
	10000	1.042 (0.08)	1.416 (0.11)	2.002 (0.04)	2.206 (0.33)
	20000	1.187 (0.13)	1.710 (0.15)	2.355 (0.04)	2.759 (0.26)
	30000	1.268 (0.17)	1.905 (0.[illegible])	2.487 (0.03)	3.183 (0.22)
	40000	1.329 (0.22)	2.043 (0.21)	2.6[illegible]5 (0.03)	3.523 (0.22)
	50000	1.387 (0.25)	2.147 (0.25)	2.778 (0.06)	3.806 (0.24)
	60000	1.452 (0.26)	2.234 (0.28)	2.891 (0.15)	4.049 (0.28)
	70000	1.510 (0.28)	2.316 (0.32)	2.998 (0.21)	4.268 (0.31)
	80000	1.559 (0.31)	2.386 (0.36)	3.087 (0.26)	4.496 (0.31)
	90000	1.610 (0.33)	2.445 (0.40)	3.180 (0.30)	4.709 (0.32)
35	10	0.341 (0.17)	0.498 (0.16)	0.561 (0.01)	
	50	0.546 (0.19)	0.744 (0.17)	0.902 (0.04)	
	100	0.642 (0.19)	0.857 (0.17)	1.072 (0.07)	
	1000	1.056 (0.14)	1.376 (0.21)	1.861 (0.06)	
	5000	1.507 (0.20)	2.029 (0.35)	2.844 (0.02)	
	10000	1.810 (0.29)	2.460 (0.47)	3.563 (0.08)	
	20000	2.267 (0.47)	3.073 (0.70)	4.608 (0.22)	
	30000	2.655 (0.62)	3.582 (0.96)	5.348 (0.28)	
	40000	3.027 (0.75)	4.036 (1.25)	5.914 (0.35)	
	50000	3.388 (0.86)	4.440 (1.52)	6.380 (0.43)	
	60000	3.760 (0.99)	4.780 (1.71)	6.769 (0.50)	
	70000	4.061 (1.07)	5.020 (1.77)	7.127 (0.58)	
	80000	4.426 (1.19)	5.280 (1.88)	7.456 (0.66)	
	90000	4.860 (1.37)	5.575 (2.00)	7.792 (0.77)	
45	10	0.528 (0.07)	0.655 (0.01)	0.815 (0.05)	1.000 (0.56)
	50	0.851 (0.05)	0.987 (0.03)	1.261 (0.08)	1.344 (0.56)
	100	1.003 (0.05)	1.182 (0.05)	1.509 (0.09)	1.555 (0.60)
	1000	1.773 (0.09)	2.301 (0.15)	2.974 (0.07)	2.906 (0.99)
	5000	2.927 (0.45)	4.225 (0.11)	5.386 (0.27)	5.209 (1.51)
	10000	3.823 (0.78)	6.111 (0.39)	7.051 (0.56)	7.383 (1.18)
	20000	5.337 (1.20)	9.473 (1.82)	9.649 (0.50)	10.71 (0.05)
	30000	6.841 (1.50)	10.37 (-)	11.39 (0.42)	12.56 (-)
	40000	8.321 (2.70)			
	50000	9.316 (3.23)			
	60000	10.07 (3.43)			
	70000	12.80 (-)			

Bracketed values are standard deviation for the average strain values

TABLE VI-4 PERCENT PERMANENT STRAIN - VIRGIN LB SERIES

Temperature	N	1LB	2LB	3LB	4LB
25	10	0.254 (0.11)	0.257 (0.04)	0.307 (0.07)	0.207 (0.27)
	50	0.460 (0.17)	0.525 (0.13)	0.538 (0.09)	0.585 (0.10)
	100	0.549 (0.20)	0.627 (0.14)	0.639 (0.10)	0.693 (0.11)
	1000	0.829 (0.26)	0.985 (0.15)	1.035 (0.08)	1.186 (0.04)
	5000	1.083 (0.21)	1.354 (0.16)	1.525 (0.10)	2.161 (0.52)
	10000	1.245 (0.15)	1.608 (0.16)	1.904 (0.20)	3.029 (1.16)
	20000	1.477 (0.00)	1.951 (0.15)	2.499 (0.42)	4.383 (2.35)
	30000	1.651 (0.13)	2.203 (0.14)	2.997 (0.65)	5.510 (3.46)
	40000	1.799 (0.25)	2.411 (0.13)	3.465 (0.91)	6.408 (4.37)
	50000	1.929 (0.36)	2.592 (0.12)	4.062 (1.30)	7.143 (5.15)
	60000	2.045 (0.47)	2.767 (0.09)	4.617 (1.74)	7.746 (5.78)
	70000	2.162 (1.58)	2.930 (0.03)	4.151 (1.85)	8.259 (6.31)
	80000	2.277 (0.69)	3.097 (0.04)	4.412 (2.13)	8.701 (6.76)
	90000	2.389 (0.81)	3.262 (0.14)	4.671 (2.38)	
35	10	0.385 (0.03)	0.557 (0.14)	0.657 (0.23)	
	50	0.580 (0.05)	0.810 (0.14)	0.917 (0.28)	
	100	0.667 (0.05)	0.928 (0.14)	1.039 (0.30)	
	1000	1.143 (0 [illegible]	1.541 (0.14)	1.662 (0.41)	
	5000	1.927 (0 [illegible]	2.491 (0.19)	2.495 (0.48)	
	10000	2.660 (1. [illegible]	3.259 (0 31)	3.130 (0.57)	
	20000	3.986 (1.88)	4.387 [illegible]	4.061 (0.74)	
	30000	5.304 (2.29)	5.24[illegible]	4.849 (0.95)	
	40000	6.804 (2.17)	5.927 [illegible]	5.565 (1.1[illegible])	
	50000	8.634 (1.38)	6.502 (0.80)	6.260 (1.45)	
	60000	10.79 (0.13)	7.021 (0.86)	6.997 (1.82)	
	70000	11.71 (0.08)	7.503 (0.91)	7.716 (2.20)	
	80000		7.959 (0.96)	8.473 (2.66)	
	90000		8.39[illegible] (0.99)		
45	10	0.790 (0.35)	0.39[illegible] [illegible]26)	0.650 (0.33)	0.865 (0.20)
	50	1.041 (0.35)	0.71[illegible] [illegible]37)	1.016 (0.36)	1.266 (0.17)
	100	1.195 (0.37)	0.907 [illegible].42)	1.236 (0.39)	1.511 (0.20)
	1000	2.289 (0.55)	2.245 (0.60)	2.666 (0.66)	3.092 (0.67)
	5000	5.266 (1.60)	5.185 (0.78)	5.411 (1.47)	6.259 (0.71)
	10000	8.222 (2.29)	7.574 (0.46)	7.497 (1.87)	9.514 (1.12)
	20000	11.67 (0.51)	10.75 (0.51)	10.20 (1.88)	11.83 (0.27)
	30000		12.72 (-)		
	40000				
	50000				
	60000				
	70000				

Bracketed values are standard deviation for the average strain values

TABLE VI-5 PERCENT PERMANENT STRAIN - RECYCLE LG SERIES

Temperature	N	7LG	8LG	9LG	10LG
25	10	0.275 (0.28)	0.154 (0.06)	0.219 (0.08)	0.100 (0.01)
	50	0.380 (0.29)	0.263 (0.08)	0.361 (0.07)	0.189 (0.01)
	100	0.427 (0.30)	0.313 (0.09)	0.426 (0.06)	0.230 (0.01)
	1000	0.606 (0.35)	0.489 (0.11)	0.643 (0.07)	0.376 (0.01)
	5000	0.758 (0.40)	0.637 (0.14)	0.847 (0.09)	0.506 (0.01)
	10000	0.842 (0.44)	0.720 (0.15)	0.971 (0.13)	0.601 (0.02)
	20000	0.938 (0.50)	0.820 (0.18)	1.148 (0.20)	0.717 (0.05)
	30000	1.003 (0.54)	0.885 (0.21)	1.286 (0.26)	0.789 (0.06)
	40000	1.053 (0.58)	0.937 (0.23)	1.403 (0.32)	0.845 (0.06)
	50000	1.093 (0.60)	0.985 (0.26)	1.511 (0.37)	0.890 (0.06)
	60000	1.130 (0.63)	1.025 (0.30)	1.604 (0.41)	0.928 (0.07)
	70000	1.160 (0.65)	1.062 (0.34)	1.685 (0.45)	0.961 (0.07)
	80000	1.191 (0.67)	1.099 (0.38)	1.765 (0.48)	0.992 (0.07)
	90000	1.229 (0.68)	1.142 (0.41)	1.832 (0.51)	1.008 (0.08)
35	10	0.388 (0.12)	0.347 (0.08)	0.320 (0.03)	
	50	0.552 (0.13)	0.505 (0.09)	0.485 (0.05)	
	100	0.628 (0.14)	0.568 (0.09)	0.541 (0.08)	
	1000	0.899 (0.16)	0.820 (0.10)	0.840 (0.11)	
	5000	1.140 (0.22)	1.060 (0.16)	1.315 (0.03)	
	10000	1.304 (0.25)	1.214 (0.23)	1.657 (0.15)	
	20000	1.519 (0.31)	1.409 (0.36)	2.166 (0.32)	
	30000	1.684 (0.35)	1.539 (0.45)	2.594 (0.47)	
	40000	1.814 (0.39)	1.642 (0.54)	2.956 (0.59)	
	50000	1.921 (0.42)	1.736 (0.62)	3.271 (0.68)	
	60000	2.018 (0.46)	1.817 (0.69)	3.576 (0.72)	
	70000	2.106 (0.49)	1.890 (0.76)	3.876 (0.75)	
	80000	2.188 (0.53)	1.961 (0.81)	4.180 (0.77)	
	90000	2.272 (0.58)	2.032 (0.87)	4.510 (0.80)	
45	10	0.433 (0.15)	0.445 (0.15)	0.518 (0.18)	0.602 (0.18)
	50	0.634 (0.12)	0.655 (0.18)	0.731 (0.19)	0.871 (0.15)
	100	0.746 (0.11)	0.765 (0.20)	0.853 (0.20)	1.034 (0.10)
	1000	1.291 (0.04)	1.337 (0.[illegible])	1.600 (0.31)	2.043 (0.35)
	5000	2.204 (0.09)	2.316 (0.07)	3.119 (0.38)	3.877 (0.88)
	10000	2.920 (0.18)	3.218 (0.23)	4.359 (0.57)	5.479 (1.15)
	20000	3.900 (0.34)	4.604 (0.82)	5.951 (0.88)	8.384 (1.25)
	30000	4.485 (0.71)	5.659 (1.26)	7.152 (1.19)	10.63 (1.41)
	40000	4.925 (1.22)	6.581 (1.45)	8.154 (1.47)	11.39 (1.64)
	50000	5.059 (1.29)	7.427 (1.55)	9.026 (1.73)	11.72 (-)
	60000		8.214 (1.58)	9.309 (1.96)	
	70000		8.973 (1.59)	10.55 (2.12)	

Bracketed values are standard deviation for the average strain values

TABLE VI-8 PERCENT PERMANENT STRAIN RECYCLE LB SERIES

Temperature	N	7LB	8LB	9LB	
25	10	0.271 (0.22)	0.153 (0.02)	0.415 (0.15)	[illegible] (0.11)
	50	0.384 (0.23)	0.319 (0.06)	0.598 (0.20)	[illegible] (0.06)
	100	0.276 (-)	0.392 (0.06)	0.679 (0.22)	[illegible] (0.03)
	1000	0.606 (0.20)	0.625 (0.08)	0.931 (0.18)	[illegible] (0.06)
	5000	0.765 (0.15)	1.833 (0.07)	1.315 (0.21)	[illegible]5 (0.11)
	10000	0.873 (0.09)	0.953 (0.07)	1.575 (0.25)	[illegible]55 (0.09)
	20000	1.023 (0.01)	1.089 (0.05)	1.939 (0.30)	[illegible]361 (0.02)
	30000	1.139 (0.11)	1.170 (0.04)	2.202 (0.3[illegible])	[illegible].754 (0.05)
	40000	1.243 (0.19)	1.228 (0.03)	2.099 (-)	3.082 (0.08)
	50000	1.343 (0.29)	1.270 (0.02)	2.243 (-)	3.367 (0.09)
	60000	1.440 (0.38)	1.303 (0.01)	2.355 (-)	3.622 (0.08)
	70000	1.536 (0.47)	1.324 (0.01)	2.440 (-)	4.845 (0.05)
	80000	1.622 (0.57)	1.344 (0.02)	2.532 (-)	4.053 (0.01)
	90000	1.710 (0.66)	1.358 (0.03)	2.591 (-)	4.249 (0.07)
35	10	0.365 (0.05)	0.301 (0.08)	0.529 (0.03)	
	50	0.568 (0.09)	0.464 (0.09)	0.769 (0.05)	
	100	0.655 (0.11)	0.536 (0.10)	0.890 (0.05)	
	1000	1.074 (0.27)	0.828 (0.13)	1.585 (0.05)	
	5000	1.627 (0.59)	1.169 (0.17)	2.721 (0.09)	
	10000	2.048 (0.79)	1.417 (0.24)	3.690 (0.11)	
	20000	2.737 (1.12)	1.775 (0.38)	5.219 (0.19)	
	30000	3.396 (1.48)	2.045 (0.50)	6.419 (0.35)	
	40000	4.049 (1.91)	2.279 (0.61)	7.416 (0.50)	
	50000	4.708 (2.37)	2.484 (0.72)	8.284 (0.63)	
	60000	5.346 (2.85)	2.681 (0.82)	9.078 (0.74)	
	70000	5.986 (3.38)	2.884 (0.92)	9.757 (0.84)	
	80000	6.658 (3.97)	3.091 (1.01)	10.35 (0.88)	
	90000	7.401 (4.64)	3.314 (1.09)	10.89 (0.89)	
45	10	0.543 (0.03)	0.490 (0.04)	0.220 (0.07)	0.527 (0.03)
	50	0.793 (0.01)	0.711 (0.08)	0.441 (0.10)	0.852 (0.06)
	100	0.917 (0.01)	0.827 (0.09)	0.571 (0.10)	1.066 (0.09)
	1000	1.705 (0.09)	1.460 (0.02)	1.446 (0.07)	2.803 (0.70)
	5000	3.351 (0.32)	2.482 (0.37)	2.992 (0.51)	7.034 (2.66)
	10000	4.657 (0.47)	3.391 (0.76)	4.295 (0.86)	
	20000	6.498 (0.54)	4.947 (1.38)	5.898 (0.95)	
	30000	7.760 (0.55)	6.897 (1.04)	7.125 (1.00)	
	40000	8.455 (0.28)	9.653 (0.66)	8.154 (1.00)	
	50000	9.027 (-)	10.54 (-)	9.083 (1.02)	
	60000	9.744 (-)	11.79 (-)	9.775 (0.82)	
	70000	10.46 (-)	12.99 (-)	10.61 (1.07)	

Bracketed values are standard deviations for the average strain values

TABLE VI-7 PERCENT PERMANENT STRAIN - PMA SERIES

Temperature	N	5LG	6LG	5LB	6LB
25	10	0.145 (0.03)	0.075 (0.00)	0.177 (0.08)	0.111 (0.02)
	50	0.260 (0.05)	0.142 (0.00)	0.307 (0.10)	0.201 (0.03)
	100	0.315 (0.06)	0.175 (0.01)	0.366 (0.10)	0.242 (0.03)
	1000	0.519 (0.07)	0.286 (0.02)	0.562 (0.11)	0.370 (0.04)
	5000	0.695 (0.08)	0.356 (0.02)	0.705 (0.14)	0.456 (0.02)
	10000	0.785 (0.11)	0.385 (0.02)	0.780 (0.15)	0.484 (0.02)
	20000	0.884 (0.15)	0.419 (0.01)	0.848 (0.17)	0.512 (0.02)
	30000	0.964 (0.21)	0.437 (0.02)	0.881 (0.18)	0.525 (0.03)
	40000	1.022 (0.25)	0.459 (0.03)	0.904 (0.19)	0.536 (0.03)
	50000	1.107 (0.33)	0.465 (0.04)	0.921 (0.20)	0.547 (0.04)
	60000	1.215 (0.47)	0.461 (0.02)	0.936 (0.21)	0.558 (0.04)
	70000		0.467 (0.02)	0.946 (0.21)	0.568 (0.05)
	80000		0.469 (0.02)	0.959 (0.22)	0.584 (0.07)
	90000		0.472 (0.02)	0.969 (0.22)	0.596 (0.09)
45	10	0.529 (0.10)	0.356 (0.07)	0.537 (0.13)	0.434 (0.21)
	50	0.769 (0.17)	0.530 (0.03)	0.739 (0.11)	0.617 (0.21)
	100	0.881 (0.20)	0.604 (0.01)	0.828 (0.11)	0.694 (0.21)
	1000	1.384 (0.36)	0.848 (0.09)	1.226 (0.05)	0.963 (0.21)
	5000	2.135 (0.81)	1.017 (0.14)	1.710 (0.10)	1.194 (0.20)
	10000	2.637 (1.17)	1.089 (0.17)	2.017 (0.21)	1.309 (0.19)
	20000	3.152 (1.49)	1.176 (0.18)	2.391 (0.39)	1.424 (0.17)
	30000	3.446 (1.61)	1.222 (0.20)	2.624 (0.51)	1.481 (0.15)
	40000	3.647 (1.64)	1.263 (0.22)	2.805 (0.58)	1.526 (0.13)
	50000	3.842 (1.65)	1.302 (0.24)	2.965 (0.61)	1.564 (0.13)
	60000	4.021 (1.67)	1.335 (0.24)	3.112 (0.65)	1.597 (0.12)
	70000	4.178 (1.70)	1.366 (0.26)	3.321 (0.75)	1.634 (0.12)

Bracketed values are standard deviation for the average strain values

TABLE VI-8 STATISTICS FOR MODEL VI-1
(Virgin Mixes)

INTERCEPT	-.95219
MULTIPLE CORRELATION	.93448
STD. ERROR OF ESTIMATE	.10302
R-SQUARED	.87326

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
1	4.043	.4709	.4022	.58510590	.0105	55.9646
2	8.487	2.6959	-.4859	-.10799730	.0018	-59.1383
DEPENDENT						
3	.497	.2889				

$$\text{Log}(\epsilon) = -0.95219225 + 0.58510590*\text{LOG}(N) - 0.10799730*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})$$

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	2	45.5557	22.7779
DEVIATION FROM REGRESSION	623	6.6116	.0106
TOTAL	625	52.1674	
Lack of Fit	237	3.3518	0.0141
Pure Error	386	3.2598	0.0084
			F value = 1.67

TABLE VI-9 STATISTICS FOR MODEL VI-II
(Recycle Mixes)

INTERCEPT	-.95196
MULTIPLE CORRELATION	.91178
STD. ERROR OF ESTIMATE	.14117
R-SQUARED	.83133

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
1	4.084	.4824	.3209	.56350023	.0124	45.4818
2	9.588	2.7301	-.5653	-.07651961	.0054	-14.1043
3	12.989	4.4357	-.6091	-.02956472	.0032	-9.2604
DEPENDENT						
4	.232	.3431				

$$\text{Log}(\epsilon) = -0.95196152 + 0.56350023*\text{LOG}(N)$$
$$- 0.07651961*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})$$
$$- 0.02956472*\text{Log}(N)*\text{Log}(\text{Abson Stiffness})*\text{Log}(\%\ \text{RAP})$$

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	3	74.4591	24.8197
DEVIATION FROM REGRESSION	758	15.1067	.0199
TOTAL	761	89.5658	
Lack of Fit	294	9.8958	0.0336
Pure Error	464	5.2114	0.0112
			F value = 2.99

TABLE VI-10 CORRELATION MATRIX FOR RAP AND RHEOLOGY RESULTS

STATISTICAL PARAMETERS (N = 20)

VAR = 1	% RAP	MEAN =	16.9000	STD =	16.4569
VAR = 2	Penetration (design)	MEAN =	223.5000	STD =	94.1418
VAR = 3	Abs. Viscosity (design)	MEAN =	69.2025	STD =	34.3801
VAR = 4	Kin. Viscosity (design)	MEAN =	235.6750	STD =	61.1749
VAR = 5	Penetration (Abson)	MEAN =	104.7000	STD =	51.7942
VAR = 6	Abs. Viscosity (Abson)	MEAN =	1319.1934	STD =	4257.4922
VAR = 7	Kin. Viscosity (Abson)	MEAN =	602.3643	STD =	677.9246

SYMMETRIC CORRELATION COEFFICIENT MATRIX

	%RAP	PEN	ABS	KIN	ABPEN	ABABS	ABKIN
%RAP	1.0000	-.1197	.1482	.0495	-.6197	.5266	.5884
(Virgin)							
Pen	-.1197	1.0000	-.8208	-.7113	.7735	-.2491	-.3544
Absolute	.1482	-.8208	1.0000	.9687	-.7254	.2606	.3719
Kinematic	.0495	-.7113	.9687	1.0000	-.6135	.1775	.2767
(Abson)							
Pen	-.6197	.7735	-.7254	-.6135	1.0000	-.4139	-.5422
Absolute	.5266	-.2491	.2606	.1775	-.4139	1.0000	.9874
Kinematic	.5884	-.3544	.3719	.2767	-.5422	.9874	1.0000

TABLE VI-11 STATISTICS FOR MODEL VI-III (PMA Mixes)

INTERCEPT	-.63232
MULTIPLE CORRELATION	.93781
STD. ERROR OF ESTIMATE	.09685
R-SQUARED	.87949

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
4	4.079	.4737	.2038	.36189592	.0145	24.9393
5	8.432	2.8483	-.7505	-.09796804	.0024	-40.5954
DEPENDENT						
6	.018	.2778				

$$\text{Log}(\epsilon) = -0.63231725 + 0.36189592*\text{LOG}(N) - 0.09796804*\text{Log}(N)*\text{Log}(\text{Stiffness})$$

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	2	16.2244	8.1122
DEVIATION FROM REGRESSION	237	2.2231	.0094
TOTAL	239	18.4475	
Lack of Fit	89	0.9671	0.0108
Pure Error	148	1.2559	0.0084
			F value = 1.28

TABLE VI-12 STATISTICS FOR MODEL VI-IV (Virgin/Recycle Mixes)

INTERCEPT	-.94945
MULTIPLE CORRELATION	.93034
STD. ERROR OF ESTIMATE	.12694
R-SQUARED	.86553

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
4	4.065	.4775	.3093	.57667267	.0084	68.6588
5	9.091	2.7685	-.5571	-.10413331	.0016	-63.9685
6	7.131	7.2525	-.5427	-.01359841	.0005	-25.2003
DEPENDENT						
7	.351	.3458				

Log(ϵ) = - 0.94945192 + 0.57667267*LOG(N)

- 0.10413331*Log(N)*Log(Abson Stiffness)

- 0.01359841*Log(N)*Log(Abson Stiffness)*Log(% RAP)

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	3	143.5473	47.8491
DEVIATION FROM REGRESSION	1384	22.3012	.0161
TOTAL	1387	165.8485	
Lack of Fit	534	13.8300	0.0259
Pure Error	850	8.4711	0.0099

F value = 2.59

TABLE VI-13 STATISTICS FOR MODEL VI-V (Virgin/PMA Mixes)

INTERCEPT	-.91192
MULTIPLE CORRELATION	.94872
STD. ERROR OF ESTIMATE	.11314
R-SQUARED	.90007

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
1	4.053	.4717	.2580	.48857605	.0093	52.7'78
2	7.202	2.6762	-.6179	-.10749453	.0017	-63.1238
3	-.446	.8957	-.6004	-.15686083	.0045	-34.7792
DEPENDENT						
4	.364	.3573				

$$\text{Log}(\epsilon) = -0.91191858 + 0.48857605*\text{LOG}(N) - 0.10749453*\text{Log}(N)*\text{Log}(\text{Stiffness}) - 0.15686083*\text{DUMMY}$$

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	3	99.3803	33.1268
DEVIATION FROM REGRESSION	862	11.0334	.0128
TOTAL	865	110.4137	
Lack of Fit	328	6.5168	0.0199
Pure Error	534	4.5157	0.0084
			F value = 2.48

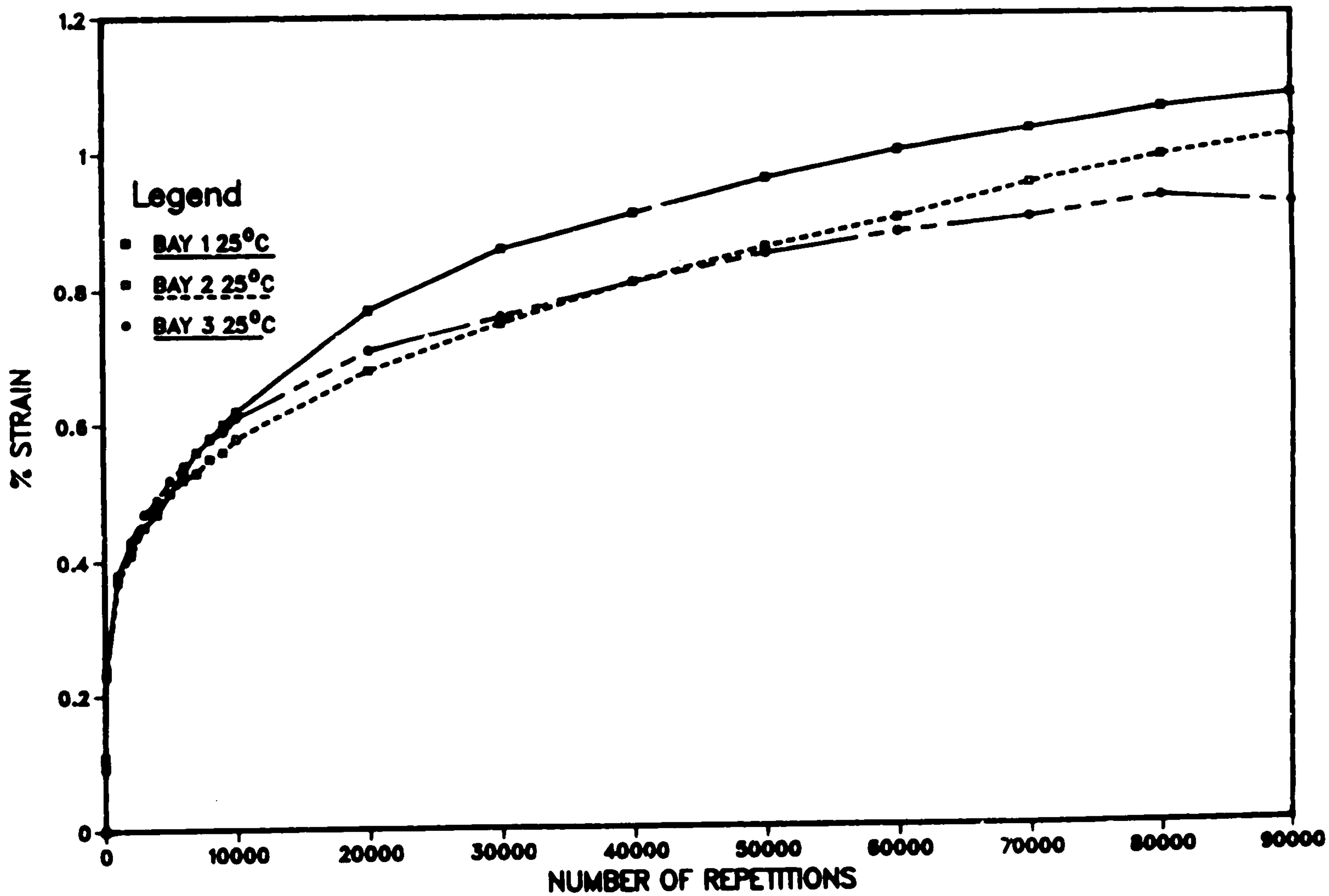


FIGURE VI-1 EXAMPLE STRAIN DATA FOR REPEATED TESTS

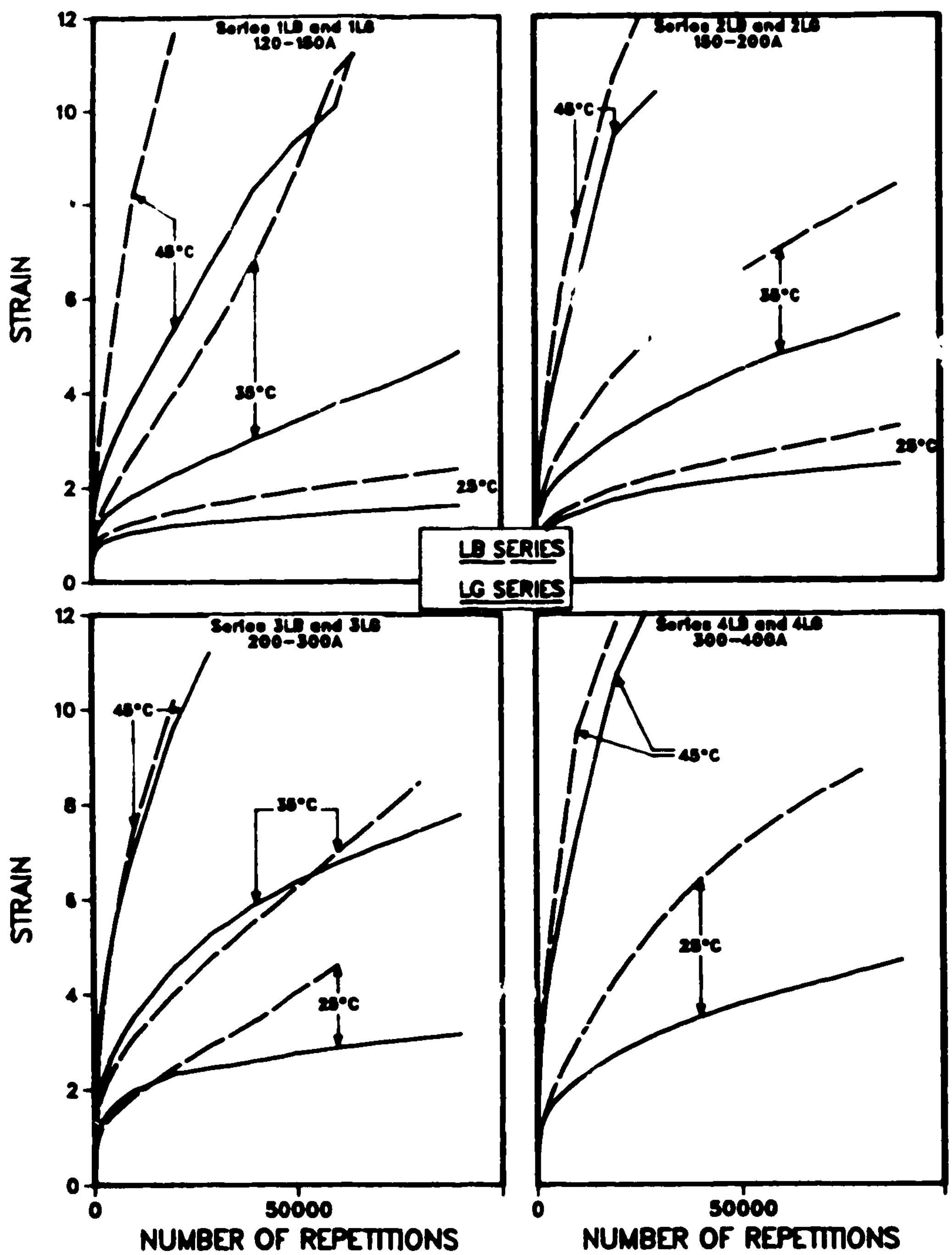


FIGURE VI-2 PERMANENT DEFORMATION CURVES FOR VIRGIN MIXES

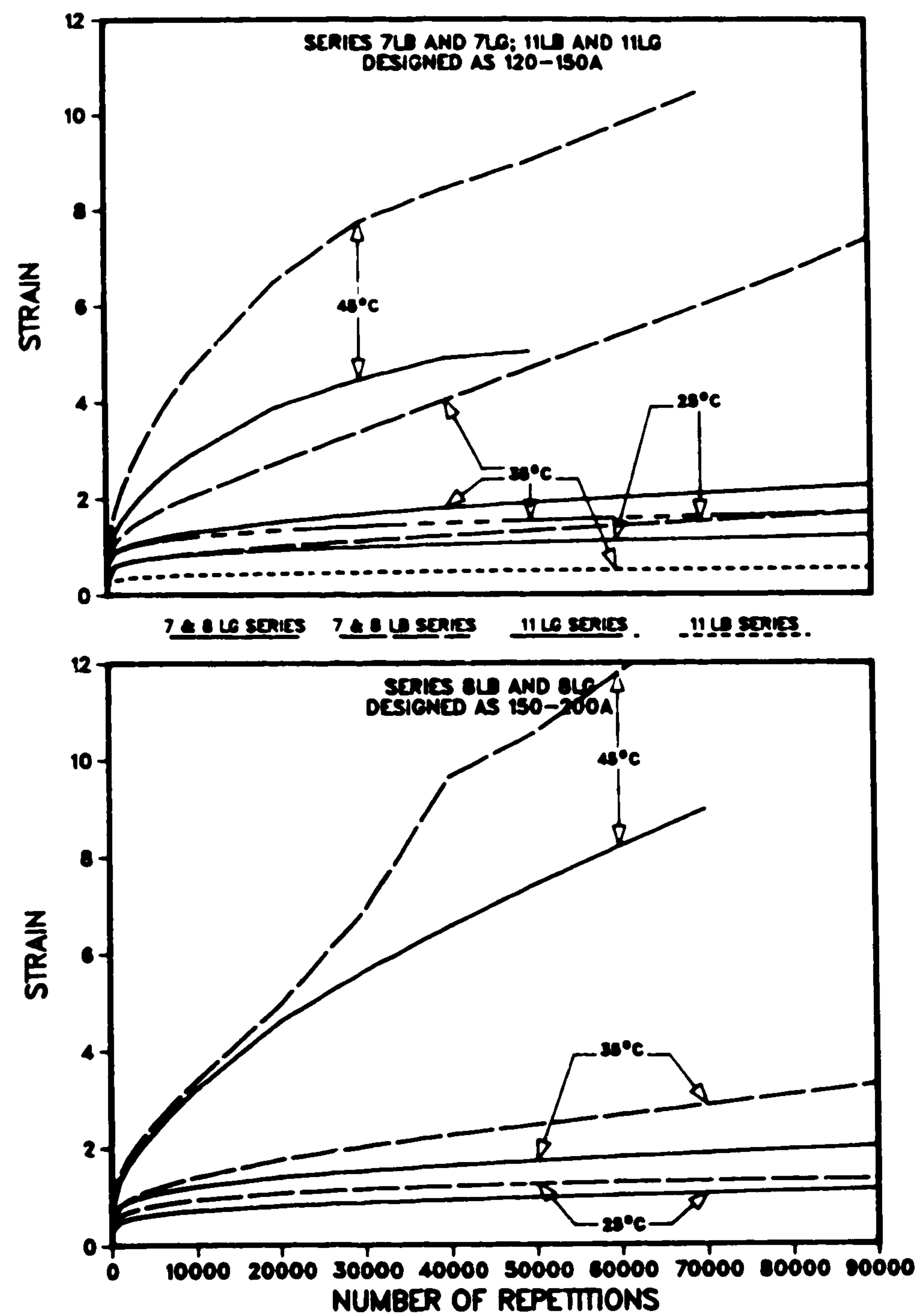


FIGURE VI-3 PERMANENT DEFORMATION CURVES FOR RECYCLE MIXES

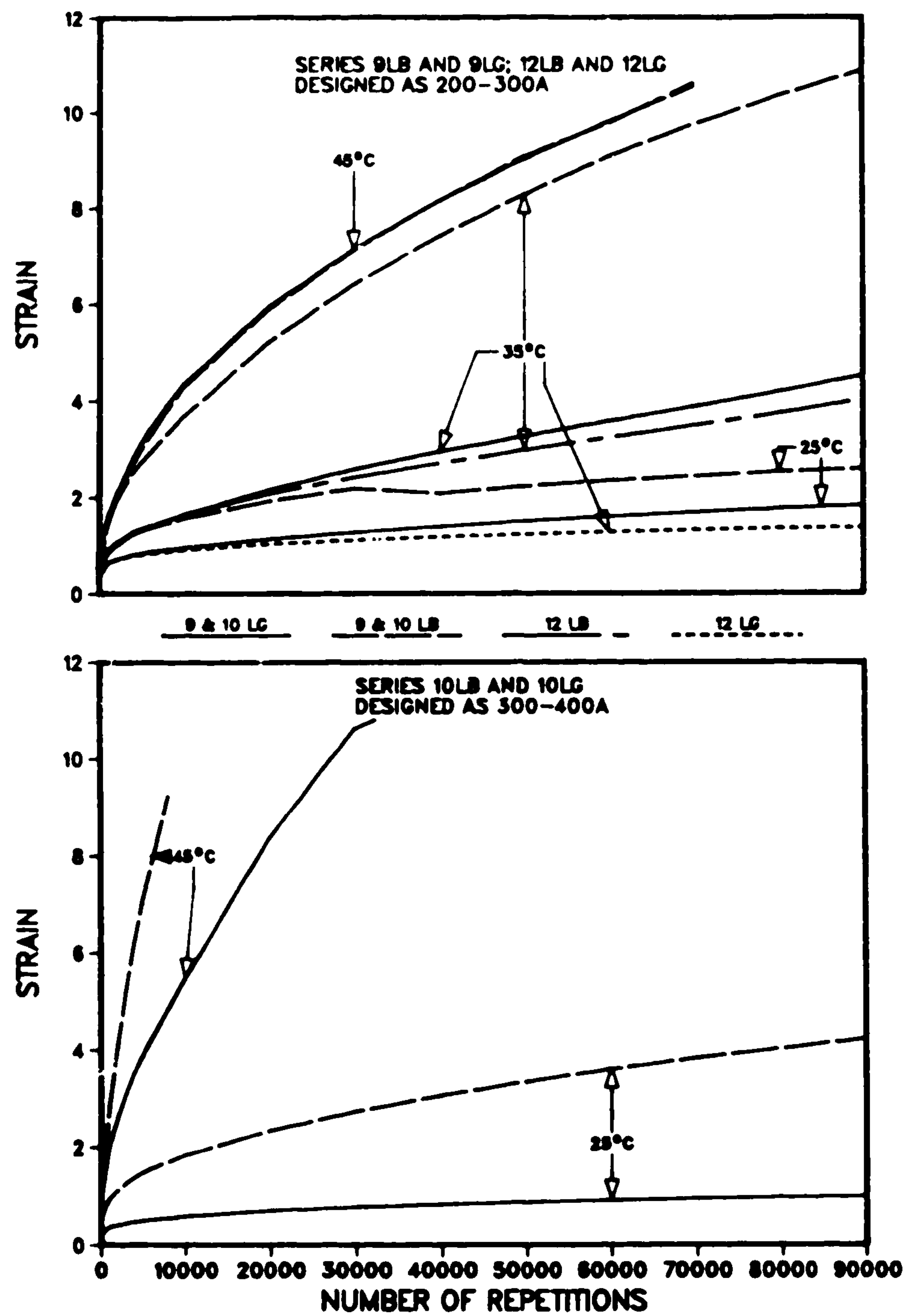


FIGURE VI-4 PERMANENT DEFORMATION CURVES FOR RECYCLE MIXES

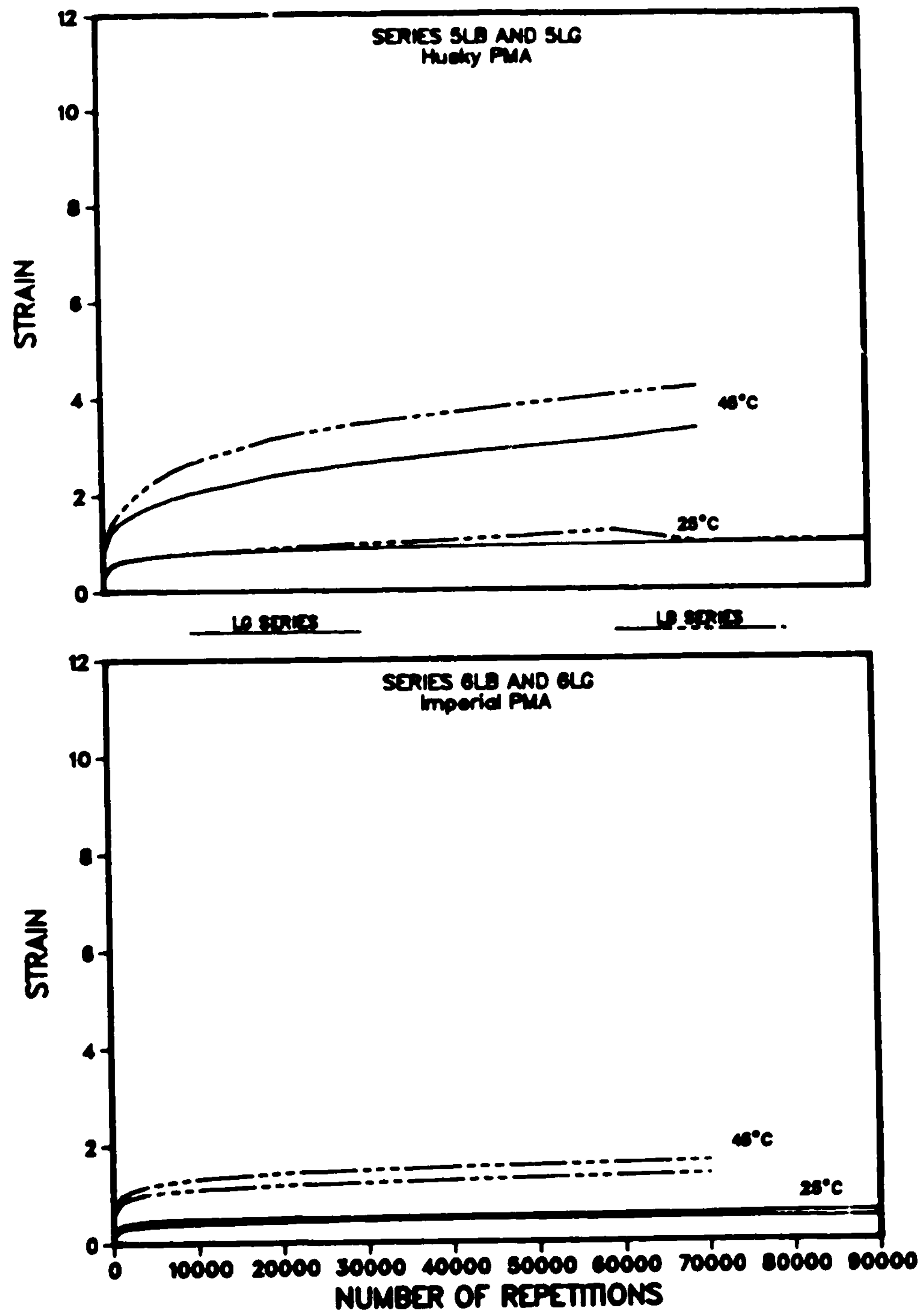


FIGURE VI-5 PERMANENT DEFORMATION CURVES FOR PMA MIXES

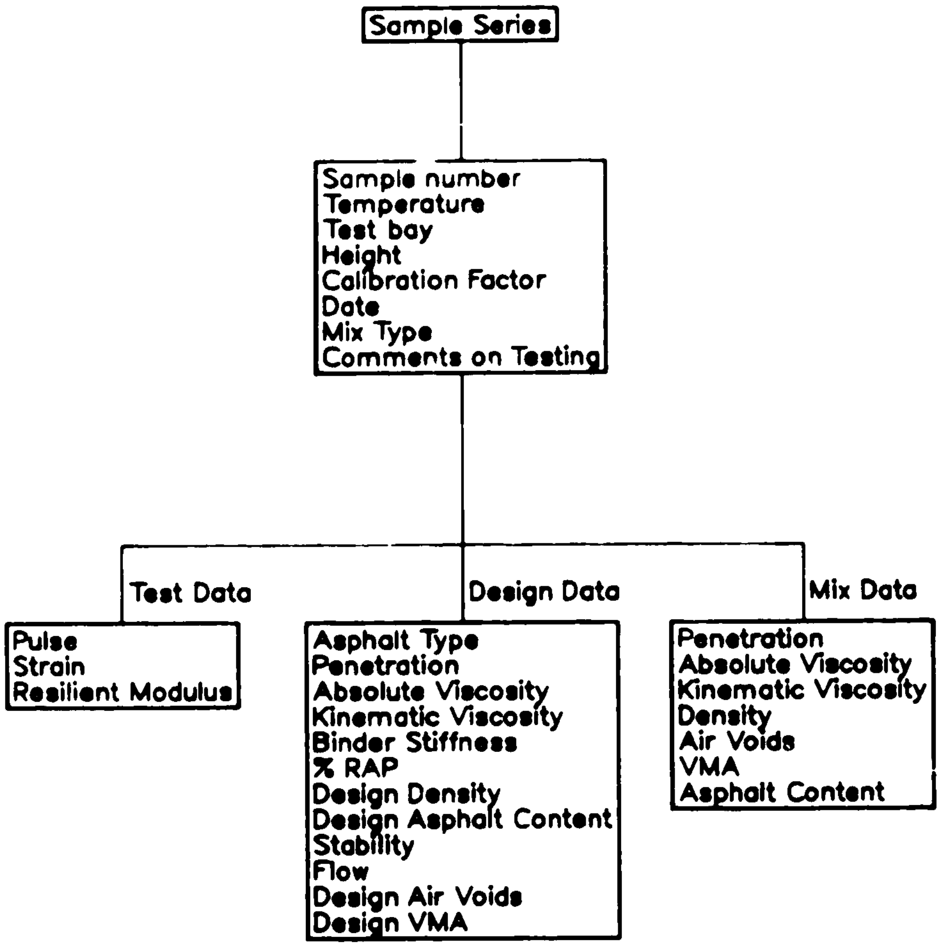


FIGURE VI-6 TRIAXIAL DATABASE VARIABLES

CHAPTER VII
DISCUSSION

7.0 OVERVIEW

This study was undertaken to determine the contribution of the asphalt binder to the permanent deformation of asphalt concrete subjected to repeated loadings under both laboratory and field conditions. The research examined conventional asphalt binders, recycled asphalt binders and polymer modified binders. As well, the mix design characteristics and the characteristics of the asphalt concrete were included in the analysis. The research was conducted in two distinct stages; one which examined rutting of in-place pavements, and one which examined the behaviour of various mixes subjected to repeated load triaxial testing in the laboratory.

7.1 Limitations of Analysis

The main focus of this study was on the binder influence on the permanent deformation characteristics of asphalt concrete pavements. However as was presented previously, the aggregate grading was also considered on a limited basis and mix characteristics were considered in the analysis. The mix characteristics were not designed into the experiment. All mix characteristics, for both the laboratory samples and the site samples were at or near targetted design values. This lack of significant range in these characteristics could possibly have resulted in other significant main effects or interactions not being observed within the analysis conducted for this study.

7.2 Stage I - Field Study

The field evaluation identified the loading and binder characteristics as statistically significant in explaining the observed rutting in the field. The two models presented in Chapter IV indicate that for 2500 daily cumulative ESAL's, increases in rut depths from 10 to 20% are predicted for changes of a single asphalt grade (from 150-200A to 200-300A). The calculations for these values utilized the rheology data for the virgin asphalts used in the repeated loading portion of this study. The Abson values were averaged for the data available and used to examine the changes predicted by the model.

While the predicted differences show significant percent changes, the absolute difference is less significant. For example a moderate 10mm rut depth would change to 12mm; a 30mm rut would increase to 36mm. Therefore the differences are less dramatic than might be suggested by the percentages; however these small differences would be important from a safety viewpoint at some borderline cases.

7.3 Stage II - Laboratory Study

The laboratory evaluation presented in Chapter VI identified the number of repetitions of load and measures of binder stiffness as the significant variables for describing the observed permanent deformation, similar to the models developed for the field data.

The relationships developed from stage II of this investigation show changes in strain of the same order of magnitude as the models determined in stage I. As for stage II the absolute differences in strain of the asphalt concrete, from the effects of adjacent grades of asphalts, is minimal. The differences resulting from the use of PMA become much more significant as do recycled binders which exhibit stiffer rheologies.

7.3.1 Laboratory Testing of Field Formed Samples

Appendix D presented the data and analysis of the laboratory testing conducted on the mix specimens formed in the field using asphalt concrete produced in a recycling asphalt plant. The results of that portion of the testing program showed that the deformation of these samples were also represented by models in which the loading, stiffness of the Abson recovered asphalt, temperature and percentage of reclaim material were the significant variables. This was the intent of examining these mixes.

The field samples also showed less aging of the recycled mix than the laboratory samples. This leads to the conclusion presented in the previous section, that the %RAP is less significant than might otherwise be concluded; however the statistical significance of the %RAP term in the field sample model shows that more work is warranted to confirm the actual role of the RAP.

7.4 Field Versus Laboratory Results

The results of the two stages of this study are complimentary. Both the field data and the Laboratory test data show the binder rheology to be a statistically significant factor in terms of the amount of permanent deformation observed.

Also, models developed for both stages of the study predict relative changes in permanent deformation of the same order of magnitude for adjacent asphalt grades.

7.5 Design Considerations

The results of this work offer practical guidance in the design of asphalt concrete pavements for highway structures within the Province of Alberta.

When selecting the asphalt characteristics for a pavement structure the objective is to optimize the high temperature stability and low temperature shrinkage cracking characteristics of the pavement[25]. The combined findings of the field and laboratory stages of this investigation allow general guidelines to be proposed for the selection of an acceptable asphalt grade. These guidelines are based on the field data, which identified critical traffic loadings and the influence of the binder, and the laboratory test data, which showed the specific influence of the three binder types. Recycled binders are considered within these guidelines, only in terms of the resulting rheological properties. Only polymer modified binders with characteristics similar to those tested during this study should be considered with respect to these guidelines. As well the limitations imposed by the range of binders examined in this study in terms of temperature susceptibility (ie: high versus low viscosities) should be considered along with these suggested asphalt grades.

The rational for the design guidelines is based on the models developed in the two stages of the study. Models (IV-I) and (IV-II) developed for the field data in stage I show only slight rut depths (< 5 mm) for daily cumulative ESAL's of 1370. For daily cumulative ESAL's of 2740 the rut depths become more major (<

10 mm). For lower traffic volumes resulting in slight rutting, 200-300A asphalt is adequate, a 300-400A would also be sufficient. For heavier traffic volumes the stiffer 150-200A grade will reduce rutting and should be used, as expected ESAL's increase 120-150A or PMA binders should be considered. The following are suggested as appropriate asphalt grades.

Suggested Design Grades

Design Life Traffic (Cumulative ESAL's)	Asphalt Grade
500,000>	200-300A or softer
500,000< >1,000,000	200-300A
>1,000,000	150-200A, 120-150A, or PMA

Examination of the data presented in Chapter III (Table III-1) shows that numerous roadways will not be subjected to high traffic loadings in the entire design life. Other highways such as highway 1, 2 and 16, will carry over a million ESAL's early in their life. Such observations should be considered in the selection of a design asphalt grade such that, when possible, low temperature performance can be optimized.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Stage I - Field Sites

The field stage of this study examined the in-place rutting of highway pavements at 40 sites throughout the province. The sites examined were located in all geographical areas, between each of the four borders. Traffic loadings varied from about 10000 ESAL's to over 3.5 million ESAL's. The field study addressed the severity of rutting throughout the province, and the location of the rutting within the pavement structure. Models were developed to explain the observed rutting in terms of traffic and the asphalt characteristics. Based on the field stage of the study the following conclusions can be made.

1. The field data shows that due to the low traffic volumes the majority of Alberta highways do not have rutting as a major form of distress.

2. Based on the field data collected and the subsequent analysis and model development, pavements subjected to less than $0.5*10^6$ ESAL's over their design life are not likely to experience significant rutting.

3. The rut profiles measured at each of the field sites show evidence of mix shoving associated with the rutting, indicating an inability of the asphalt concrete to resist the applied shear forces.

4. The models developed to explain the measured rut depths at the field sites were significant based on material properties despite the observations that movement in the lower structural layers was present at a number of the sites examined.

5. The temperature values utilized in the analysis of the field data were not significant as main effects. The model developed based on the design data includes the temperature data indirectly in terms of the calculated binder stiffness.

6. The range of aggregate gradations examined were not found to have a significant effect on the observed rutting. This suggests that in order to improve rutting resistance the aggregate engineering must look at other characteristics than just gradation.

Stage II - Laboratory Testing

The laboratory stage of this study utilized repeated load triaxial test equipment to examine the relative permanent deformation characteristics of asphalt concretes using four grades of virgin asphalt, two aggregate sources, four recycle ratios and two polymer modified asphalts. Testing was conducted at 25°, 35° and 45°C. Strain values measured for different mixes varied from about 0.5% (≅1mm) at 90000 load repetitions to over 12% (≅24mm) at 30000 load repetitions. The results obtained from the laboratory testing were used to develop various models to explain the relative influence of the different material characteristics on the permanent deformation of the asphalt concrete mixes. The laboratory results and subsequent analysis allows the following conclusions to be made.

1. The strain of the various mix samples measured during the repeated load triaxial testing was significantly effected by the test temperature. This illustrates the effect of the binder stiffness and the binder temperature susceptibility on the resulting strain experienced by the asphalt concrete.

2. The laboratory test program showed that the PMA mixes exhibited the lowest percent permanent strain. The data shows that PMA mixes reduce the strain levels in the order of 50% at the 25°C test temperature, compared to a 120-150A asphalt concrete mix.

3. Based on the results of the repeated load triaxial testing, a change between 150-200A and 200-300A grade asphalt might be expected to reduce strains as much as 30-40%. This is consistent with the field data.

4. Recycled mixes exhibited less strain during the repeated load testing than the virgin asphalt mixes. While this appears to be directly related to the influence of the resulting rheology, the specific influence of the % RAP requires further study.

5. Because the rheology of the binder after the mixing process influences the permanent deformation characteristics of an asphalt concrete, the aging of the binder during mixing must be addressed.

6. The use of repeated load triaxial test equipment shows potential for modelling the in place pavement performance in terms of permanent deformation.

8.2 Recommendations

In order to utilize the findings of this study and to build on the data collected, the following recommendations are presented.

1. The design guidelines presented in the Chapter VII discussion should be adopted by Alberta Transportation and Utilities for consideration in the design process.

2. Studies should be conducted to determine the cause of the increase binder aging noted in the laboratory prepared recycled mixes.

3. Work should continue in the characterization of polymer modified asphalts, specifically in terms of current conventional test methods.

4. The data collected for this study should be further utilized by Alberta Transportation and Utilities and should be added to for further analysis.

5. Further work should be conducted to examine the effect of aggregate characteristics, particular to Alberta sources, on the permanent deformation of asphalt concrete.

6. Further work is required with respect to determining meaningful temperature measures at the various field sites in order to better describe the interaction of temperature and loading in the field.

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APPENDIX A

PAVEMENT SLAB SECTIONS

APPENDIX A
PAVEMENT SLAB SECTIONS

Background

Two pavement sections were removed from the travel lanes at Kilometers 9.0 and 10.1 of the westbound lane of Highway 1:12. The slabs were cut the full depth of the pavement using a concrete saw and the sections were removed using a backhoe. As the sawing process is water cooled, the excess water made it impossible to observe any subgrade deformation. However observations of the slab layers made it apparent that the rut depths observed in the pavement surface are the result of movement of the asphalt mix itself as the lower levels were observed to be virtually undistorted. Figures A-1 & A-2 show the cross sections traced from the slab sections removed from the roadway. Core densities from the one slab section are also shown on the figure and indicate little change in pavement density across the lane width. The core heights show that the pavement wheel ruts can be accounted for in the ACP mixture and suggest movement of the mix with little accompanying volume change.

This project was overlaid in 1975, and had been originally paved in 1955. Construction reports indicate that the top two lifts, placed in 1975, utilized a 150-200 penetration grade asphalt while the two lower lifts incorporated 200-300 penetration grade asphalt.

The construction data is summarized in Tables A-1 and A-2 and represents the historical data available. Abson extractions performed during 1987 allowed the existing rheological properties and aggregate gradation to be determined. Each lift of the pavement was tested individually and the results are shown in Tables A-3 and A-4.

Summary

This project provided the initial impetus for examining rutting which occurred only in the asphalt concrete layer. The pavement slabs taken at the two locations showed extensive movement was experienced with the asphalt concrete resulting in the observed rutting.

The abson recovered binder from the top lift indicates that little aging occurred in the 11 years that the pavement had been in place; the lower lift was found to be significantly harder than the top.

TABLE A-1 TOP LIFT PAVEMENT DATA

	MARSHALL DATA				CORE DATA			
	DENSITY	AIR VOIDS	% A.C.	VMA	DENSITY	% COMPACTION	AIR VOIDS	VMA
FIELD	2326	2.7	6.3	15.1	2302	99.	3.7	5.9
DESIGN	2300	4.1	6.5	16.4				

AGGREGATE GRADING

SIEVES	16000	10000	5000	1250	630	315	160	80
FIELD	100	81	62	34	22	15	11	8
DESIGN	100	82	67	37	23	15	11	9

BINDER

	ORIGINAL			ATFOT			
	Absolute Viscosity	Penetration @ 25	@ 4	% Loss	Absolute Viscosity	Penetration @ 25	@ 4
Ave	60.6	189	53	0.23	1244	110	59
S.D.	2.5	5.4	-	-	-	-	-
n	6	6	2	2	2	2	2

TABLE A-2 LOWER LIFT PAVEMENT DATA

	MARSHALL DATA				CORE DATA			
	DENSITY	AIR VOIDS	% A.C.	VMA	DENSITY	% COMPACTION	AIR VOIDS	VMA
FIELD	-	-	5.2	-	2268	98.9	-	-
DESIGN	2302	5.8	5.7	19.4				

AGGREGATE GRADING

SIEVES	20000	16000	10000	5000	125	[illegible]	315	160	80
FIELD	100	88	66	52	34	[illegible]	15	11	9
DESIGN	100	88	70	55	36	[illegible]	15	11	8

BINDER DATA

No binder data is available for this portion of the project constructed in 1956. Alberta Transportation records do show that the binder was a 200-300 penetration grade asphalt.

TABLE A-3 RESULTS FROM RECOVERED SAMPLES
TOP LIFT

	%	SIEVES										Absolute	Kinematic
Location	A.C.	16000	12500	10000	5000	1250	630	315	160	80	Penetration	Viscosity	Viscosity
Shoulder	6.2	99	92	82	61	36	25	14	9.2	7.4	133	113	294
Wheelpath	6.2	99	90	81	58	34	23	13	8.4	6.5	122	130	322
Combined	5.9	100	94	83	61	36	24	13	8.4	6.6	90	209	380

TABLE A-4 RESULTS FROM RECOVERED SAMPLES
BOTTOM LIFT

	%	SIEVES										Absolute	Kinematic
Location	A.C.	16000	12500	10000	5000	1250	630	315	160	80	Penetration	Viscosity	Viscosity
Shoulder	5.3	96	83	71	52	35	25	15	10.8	8.5	57	223	321
Wheelpath	5.1	97	83	71	51	35	25	15	11.2	8.8	69	194	3[illegible]7
Combined	5.0	95	83	70	46	30	22	14	10.2	7.9	47	338	364

Note: The shoulder and wheelpath samples are from Km 9.0 and the combined sample was taken from cores at Km 10.1.

CROSS SECTION OF PAVEMENT CUT FROM

HIGHWAY 1:12

Km 9

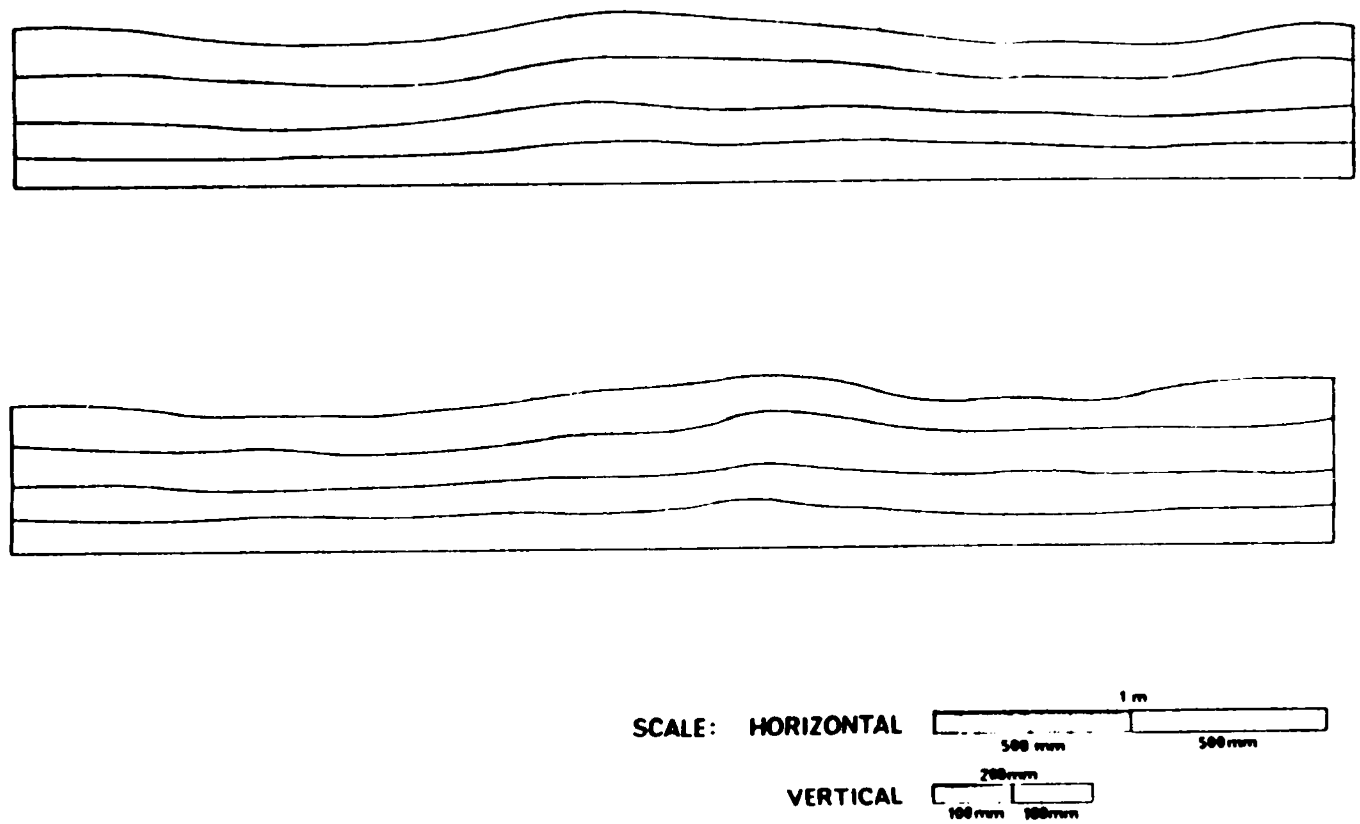

FIGURE A-1 CROSS SECTION OF PAVEMENT SLAB SHOWING RUTTING IN THE DIFFERENT LIFTS

CROSS SECTION OF PAVEMENT CUT FROM

HIGHWAY 1:12, km 10.1

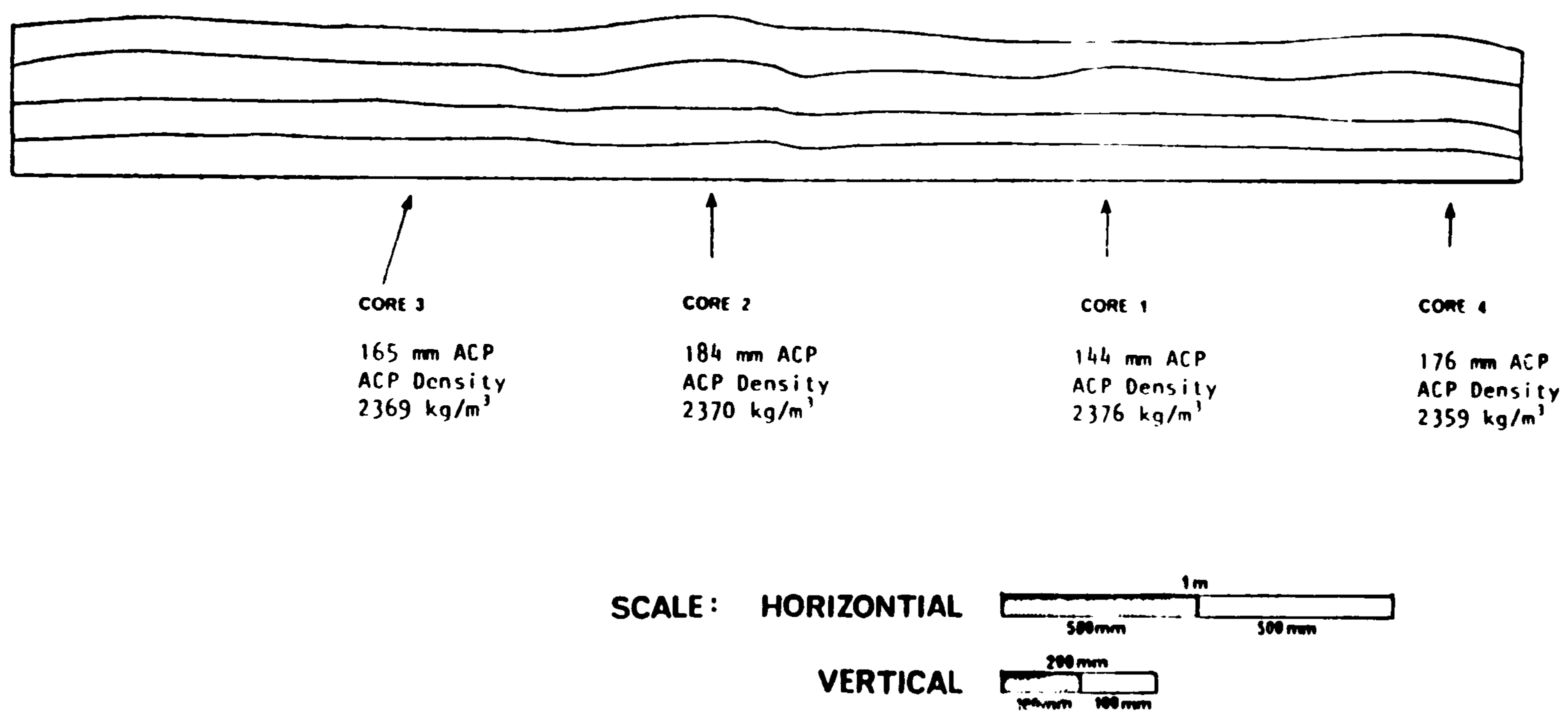

FIGURE A-2 CROSS SECTION OF PAVEMENT SLAB SHOWING RUTTING IN THE DIFFERENT LIFTS

APPENDIX B

STAGE I - SITE MATERIALS DATA AND SITE PROFILES

APPENDIX B

Stage I - Site Materials Data And Site Profiles

This appendix contains the site materials data and site profiles for Stage I of this study. Stage I involved measuring the cross-section profile and rut depths at specific sites on Alberta highways, coring these sites to obtain mix samples and to determine the actual structure, laboratory testing to determine the current in-place materials characteristics and the review of historical records to determine initial material and mix characteristics for the various sites.

This appendix presents a series of tables which give the mix data, aggregate gradings, asphalt rheology and measured rut depths for each site. The design and field data are from historical records kept by Alberta Transportation. The 1987 data is from the laboratory testing conducted on the specific sites cored for this investigation. The Laboratory results represent average values from all cores taken at a given site. The coring program generally involved extracting one core from each wheelpath, and one core from between the wheelpaths. Additional cores were taken at selected sites both to assure that there would be enough asphalt for testing and to examine the profile at the bottom of the subgrade.

The site profiles are shown in Figures B-1 to B-14. Each site was profiled with a survey level. The profiles were used to examine the shape of the rutted pavement. In most cases it can be seen that shoving of the mix has occurred in conjunction with the rutting. The site profiles also show the pavement structure as recorded by Alberta Transportation's Pavement Management System (PMS), the structures shown in the tables are from measurements of cores.

TABLE B-1 SITE MATERIALS DATA

SITE: 1 PROJECT: 22:28 KILOMETER: 27.69

PROJECT DESCRIPTION: Jct highway 13 to Jct highway 39

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,375		STABILITY (N):	10,050
	% AIR VOIDS:	3.8		FLOW (mm):	2.2
	% VMA:	13.6			
	% ASPHALT:	5.1			
FIELD -	DENSITY (kg/m³):	2,365	CORES -	DENSITY:	2,272
	% AIR VOIDS:	3.8		% AIR VOIDS:	7.6
	% VMA:	-		% COMPACTION:	95.9
	% ASPHALT:	5.3			
1987 -	DENSITY (kg/m³):	2,3[illegible]0		AGE: 5 years	
	% AIR VOIDS:	6.3			
	% VMA:	16.6			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		75	51.	33.	31.0	21.0	11.2	6.8
AS-BUILT	100.		80	57.	41.	31.0	25.0	13.0	7.5
1987	99.	90.	75	48.	33.	30.3	21.3	11.7	7.4

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	263	PENETRATION (dmm, 25 C):	106
ABS VISC (Pa.s, 60 C):	39.8	ABS VISC (Pa.s, 60 C):	167
		KIN VISC (mm²/s, 135 C):	340
STIFFNESS (kPa):	78	STIFFNESS (kPa):	343

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 200 mm
TOTAL BASE THICKNESS : 230 mm

RUT DEPTHS

AVERAGE: 0 mm
RANGE: [illegible] mm

TABLE B-2 SITE MATERIALS DATA

SITE: 2 PROJECT: 1A:02 KILOMETER: 10.14

PROJECT DESCRIPTION: Canmore to Jct highway 1X

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,462		STABILITY (N):	11,525
	% AIR VOIDS:	3.2		FLOW (mm):	2.6
	% VMA:	11.4			
	% ASPHALT:	4.2			
FIELD -	DENSITY (kg/m³):	2,474	CORES -	DENSITY:	2,344
	% AIR VOIDS:	2.4		% AIR VOIDS:	7.5
	% VMA:	-		% COMPACTION:	94.8
	% ASPHALT:	4.7			
1987 -	DENSITY (kg/m³):	-		AGE:	7 years
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		72	50.	34.	28.0	20.5	12.1	9.7
AS-BUILT		88.	75	52.	33.	28.0	22.0	16.0	10.9
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	168	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	67.0	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm²/s, 135 C):	-
STIFFNESS (kPa):	147	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 0.0 mm
RANGE: 0 - 0 mm

TABLE B-3 SITE MATERIALS DATA

SITE: 3 PROJECT: 2A:06 KILOMETER: 10.72

PROJECT DESCRIPTION: Aldersyde to Dewinton Interchange

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,350		STABILITY (N):	5,820
	% AIR VOIDS:	3.8		FLOW (mm):	2.4
	% VMA:	14.4			
	% ASPHALT:	5.2			
FIELD -	DENSITY (kg/m³):	2,383	CORES -	DENSITY:	2,271
	% AIR VOIDS:	2.2		% AIR VOIDS:	6.7
	% VMA:	12.8		% COMPACTION:	96.9
	% ASPHALT:	5.3			
1987 -	DENSITY (kg/m³):	2,352		AGE: 12 years	
	% AIR VOIDS:	4.2			
	% VMA:	13.8			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	89.	74	50.	30.	23.0	20.0	15.0	10.0
AS-BUILT	96.	82.	76	54.	29.	23.0	18.5	13.5	10.3
1987	99.	90.	78	53.	29.	25.0	20.0	15.4	11.3

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm,25 C):	317	PENETRATION (dmm,25 C):	156
ABS VISC (Pa.s,60 C):	30.1	ABS VISC (Pa.s,60 C):	78.8
		KIN VISC (mm²/s,135 C):	236
STIFFNESS (kPa):	49	STIFFNESS (kPa):	147

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 208 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 6.0 mm
RANGE: 5 - 7 mm

TABLE B-4 SITE MATERIALS DATA

SITE: 4 PROJECT: 24:02 KILOMETER: 4.63

PROJECT DESCRIPTION: Jct highway 23 to Bow River

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,347		STABILITY (N):	6,320
	% AIR VOIDS:	3.7		FLOW (mm):	2.6
	% VMA:	14.2			
	% ASPHALT:	5.4			
FIELD -	DENSITY (kg/m³):	2,373	CORES -	DENSITY:	2,258
	% AIR VOIDS:	2.4		% AIR VOIDS:	7.2
	% VMA:	13.5		% COMPACTION:	94.5
	% ASPHALT:	5.6			
1987 -	DENSITY (kg/m[illegible]	[illegible]372		AGE: 17 years	
	% AIR VOIDS:	2.5			
	% VMA:	13.5			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	90.	80.	72	51.	30.	26.0	20.0	14.0	7.9
AS-BUILT		75.	66	48.	32.	26.0	22.0	15.0	7.3
1987	94.	82.	69	46.	28.	25.3	19.3	12.1	7.9

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	266	PENETRATION (dmm, 25 C):	127
ABS VISC (Pa.s, 60 C):	32.9	ABS VISC (Pa.s, 60 C):	89.1
		KIN VISC (mm²/s, 135 C):	226
STIFFNESS (kPa):	78	STIFFNESS (kPa):	196

STRUCTURE		RUT DEPTHS
TOTAL ASPHALT PAVEMENT THICKNESS :	146 mm	AVERAGE: 7.5 mm
TOTAL BASE THICKNESS :	200 mm	RANGE: 6 - 9 mm

TABLE B-5 SITE MATERIALS DATA

SITE: 5 PROJECT: 24:02 KILOMETER: 7.95

PROJECT DESCRIPTION: Jct highway 23 to Bow River

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,347		STABILITY (N):	6,320
	% AIR VOIDS:	3.7		FLOW (mm):	2.6
	% VMA:	14.2			
	% ASPHALT:	5.4			
FIELD -	DENSITY (kg/m³):	2,373	CORES -	DENSITY:	2,258
	% AIR VOIDS:	2.4		% AIR VOIDS:	7.2
	% VMA:	13.5		% COMPACTION:	94.5
	% ASPHALT:	5.6			
1987 -	DENSITY (kg/m³):	2,376		AGE: 17 years	
	% AIR VOIDS:	1.8			
	% VMA:	13.7			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	90.	80.	72	51.	30.	26.0	20.0	14.0	7.9
AS-BUILT		75.	66	48.	32.	26.0	22.0	15.0	7.3
1987	93.	83.	71	47.	29.	25.0	19.0	11.6	7.7

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C):	266
ABS VISC (Pa.s, 60 C):	32.9
STIFFNESS (kPa):	78

ABSON

PENETRATION (dmm, 25 C):	105
ABS VISC (Pa.s, 60 C):	112
KIN VISC (mm²/s, 135 C):	259
STIFFNESS (kPa):	196

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 114 mm
TOTAL BASE THICKNESS : 308 mm

RUT DEPTHS

AVERAGE: 5.5 mm
RANGE: 5 - 6 mm

TABLE B-6 SITE MATERIALS DATA

SITE: 6 PROJECT: 21:12 KILOMETER: 34.40

PROJECT DESCRIPTION: Jct highway 1 to Jct highway 9

MIX DATA

DESIGN - DENSITY (kg/m³): 2,330
% AIR VOIDS: 3.4
% VMA: 14.7
% ASPHALT: 5.8

STABILITY (N): 6,000
FLOW (mm): 2.3

FIELD - DENSITY (kg/m³): 2,335
% AIR VOIDS: 3.4
% VMA: -
% ASPHALT: 5.8

CORES - DENSITY: 2,279
% AIR VOIDS: 5.8
% COMPACTION: 96.4

1987 - DENSITY (kg/m³): 2,343
% AIR VOIDS: 2.9
% VMA: 14.4

AGE: 17 years

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	91.	84	63.	39.	28.0	17.0	13.0	9.5
AS-BUILT	100.		80	61.	38.	28.0	19.0	13.0	8.1
1987	99.	92.	82	61.	39.	27.3	16.5	11.6	9.0

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 285
ABS VISC (Pa.s, 60 C): 30.0

STIFFNESS (kPa): 49

ABSON

PENETRATION (dmm, 25 C): 122
ABS VISC (Pa.s, 60 C): 85.8
KIN VISC (mm²/s, 135 C): 215
STIFFNESS (kPa): 196

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 100 mm
TOTAL BASE THICKNESS . 135 mm

RUT DEPTHS

AVERAGE: 3.8 mm
RANGE: 2 - 8 mm

TABLE B-7 SITE MATERIALS DATA

SITE: 7 PROJECT: 21:12 KILOMETER: 36.32

PROJECT DESCRIPTION: Jct highway 1 to Jct highway 9

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,330		STABILITY (N):	6,000
	% AIR VOIDS:	3.4		FLOW (mm):	2.3
	% VMA:	14.7			
	% ASPHALT:	5.8			
FIELD -	DENSITY (kg/m³):	2,335	CORES -	DENSITY:	2,279
	% AIR VOIDS:	3.4		% AIR VOIDS:	5.8
	% VMA:	-		% COMPACTION:	96.4
	% ASPHALT:	5.8			
1987 -	DENSITY (kg/m³):	2,321		AGE: 17 years	
	% AIR VOIDS:	3.9			
	% VMA:	15.2			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	91.	84	63.	39.	28.0	17.0	13.0	9.5
AS-BUILT	100.		80	61.	38.	28.0	19.0	13.0	8.1
1987	99.	93.	83	63.	41.	28.3	17.0	12.1	9.2

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	285	PENETRATION (dmm, 25 C):	115
ABS VISC (Pa.s, 60 C):	30.0	ABS VISC (Pa.s, 60 C):	104
		KIN VISC (mm²/s, 135 C):	236
STIFFNESS (kPa):	49	STIFFNESS (kPa):	196

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 131 mm
TOTAL BASE THICKNESS : 105 mm

RUT DEPTHS

AVERAGE: 6.3 mm
RANGE: 4 - 9 mm

TABLE B-8 SITE MATERIALS DATA

SITE: 8 PROJECT: 1:10 KILOMETER: 9.00

PROJECT DESCRIPTION: Calgary East City to Jct highway 9

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,332		STABILITY (N):	8,430
	% AIR VOIDS:	3.7		FLOW (mm):	2.0
	% VMA:	14.3			
	% ASPHALT:	5.7			
FIELD -	DENSITY (kg/m³):	2,358	CORES -	DENSITY:	2,227
	% AIR VOIDS:	2.6		% AIR VOIDS:	8.1
	% VMA:	13.4		% COMPACTION:	94.4
	% ASPHALT:	5.8			
1987 -	DENSITY (kg/m³):	2,342		AGE: 12 years	
	% AIR VOIDS:	3.2			
	% VMA:	13.6			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	87.	77	56.	35.	27.0	23.0	18.0	13.9
AS-BUILT	100.		78	56.	34.	27.0	21.0	16.0	12.2
1987	96.	85.	73	51.	32.	26.3	20.8	16.5	11.8

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 243
ABS VISC (Pa.s, 60 C): 29.1

STIFFNESS (kPa): 74

ABSON

PENETRATION (dmm, 25 C): 118
ABS VISC (Pa.s, 60 C): 79.7
KIN VISC (mm²/s, 135 C): 237
STIFFNESS (kPa): 275

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 241 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 6.0 mm
RANGE: 6 - 6 mm

TABLE B-9 SITE MATERIALS DATA

SITE: 9 PROJECT: 41A:02 KILOMETER: 6 87

PROJECT DESCRIPTION: Medicine Hat east city limit to Jct highway 41

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,345		STABILITY (N):	4,340
	% AIR VOIDS:	4.0		FLOW (mm):	2.4
	% VMA:	15.9			
	% ASPHALT:	5.9			
FIELD -	DENSITY (kg/m³):	2,330	CORES -	DENSITY:	2,259
	% AIR VOIDS:	5.9		% AIR VOIDS:	8.1
	% VMA:	-		% COMPACTION:	6.5
	% ASPHALT:	4.9			
1987 -	DENSITY (kg/m³):	-		AGE: 14 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	85.	75	56.	38.	23.0	14.0	10.0	7.2
AS-BUILT	100.		75	55.	35.	23.0	18.0	9.0	7.0
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 278
ABS VISC (Pa.s, 60 C): 25.5

STIFFNESS (kPa): 49

ABSON

PENETRATION (dmm, 25 C): -
ABS VISC (Pa.s, 60 C): -
KIN VISC (mm²/s, 135 C): -
STIFFNESS (kPa): 343

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 2.0 mm
RANGE: 1 - 4 mm

TABLE B-12 SITE MATERIALS DATA

SITE: 12 PROJECT: 887:04 KILOMETER: 45.34

PROJECT DESCRIPTION: Jct highway 61 to Jct highway 3

MIX DATA

DESIGN - DENSITY (kg/m^3): 2,382
% AIR VOIDS: 3.0
% VMA: 13.9
% ASPHALT: 5.4

STABILITY (N): 11,125
FLOW (mm): 2.5

FIELD - DENSITY (kg/m^3): 2,371
% A = VOIDS: 3.6
% VMA: -
% ASPHALT: 5.2

CORES - DENSITY: 2,254
% AIR VOIDS: -
% COMPACTION: 95.0

1987 - DENSITY (kg/m^3): 2,283
% AIR VOIDS: 7.8
% VMA: 17.2

AGE: 3 years

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	92.	82	63.	40.	31.5	23.5	15.3	11.0
AS-BUILT	100.	93.	85	67.	43.	31.5	25.0	17.0	12.4
1987	99.	93.	84	66.	40.	31.0	23.7	16.9	12.3

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm,25 C): 168
ABS VISC (Pa.s,60 C): 62.0

STIFFNESS (kPa): 68

ABSON

PENETRATION (dmm,25 C): 87
ABS VISC (Pa.s,60 C): 204
KIN VISC (mm^2/s,135 C): 374
STIFFNESS (kPa): 275

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 162 mm
TOTAL BASE THICKNESS : 90 mm

RUT DEPTHS

AVERAGE: 2.3 mm
RANGE: 0 - 4 mm

TABLE B-13 SITE MATERIALS DATA

SITE: 13 PROJECT: 529:04 KILOMETER: 2.94

PROJECT DESCRIPTION: Jct highway 23 to Jct SR 843

MIX DATA

DESIGN - DENSITY (kg/m^3): 2,376 — STABILITY (N): 9,125
% AIR VOIDS: 3.3 — FLOW (mm): 2.9
% VMA: 13.5
% ASPHALT: 5.3

FIELD - DENSITY (kg/m^3): 2,393 — CORES - DENSITY: 2,294
% AIR VOIDS: 2.8 — % AIR VOIDS: 7.1
% VMA: - — % COMPACTION: 95.8
% ASPHALT: 5.1

1987 - DENSITY (kg/m^3): 2,329 — AGE: 4 years
% AIR VOIDS: 5.8
% VMA: 15.0

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		84	61.	38.	27.0	19.0	12.2	8.6
AS-BUILT	99.		86	64.	39.	27.0	18.0	12.6	8.6
1987	99.	95.	88	65.	39.	29.0	19.8	14.2	10.5

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 208
ABS VISC (Pa.s, 60 C): 67.5
STIFFNESS (kPa): 98

ABSON

PENETRATION (dmm, 25 C): 91
ABS VISC (Pa.s, 60 C): 216
KIN VISC (mm^2/s, 135 C): 405
STIFFNESS (kPa): 206

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 160 mm
TOTAL BASE THICKNESS : 108 mm

RUT DEPTHS

AVERAGE: 3.3 mm
RANGE: 1 - 4 mm

TABLE B-14 SITE MATERIALS DATA

SITE: 14 PROJECT: 512:02 KILOMETER: 6.30

PROJECT DESCRIPTION: Lethbridge east city limit to Jct highway 3

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,332		STABILITY (N):	6,000
	% AIR VOIDS:	4.9		FLOW (mm):	2.2
	% VMA:	14.6			
	% ASPHALT:	5.4			
FIELD -	DENSITY (kg/m³):	2,358	CORES -	DENSITY:	2,286
	% AIR VOIDS:	4.0		% AIR VOIDS:	7.1
	% VMA:	-		% COMPACTION:	96.8
	% ASPHALT:	5.3			
1987 -	DENSITY (kg/m³):	2,362		AGE: 10 years	
	% AIR VOIDS:	4.3			
	% VMA:	13.3			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	85.	72	47.	26.	23.0	15.0	9.0	4.9
AS-BUILT	100.		77	50.	28.	23.0	16.0	10.0	6.1
1987	99.	92.	76	49.	28.	23.0	15.0	9.0	6.1

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION(dmm,25 C):	295	PENETRATION(dmm,25 C):	169
ABS VISC(Pa.s,60 C):	34.5	ABS VISC(Pa.s,60 C):	76.7
		KIN VISC(mm²/s,135 C):	248
STIFFNESS (kPa):	39	STIFFNESS (kPa):	96

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 165 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 7.3 mm
RANGE: 2 - 14 mm

TABLE B-15 SITE MATERIALS DATA

SITE: 15 PROJECT: 524:04 KILOMETER: 18.24

PROJECT DESCRIPTION: Jct highway 36 to South Saskatchewan River

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,377		STABILITY (N):	10,730
	% AIR VOIDS:	3.6		FLOW (mm):	2.4
	% VMA:	13.5			
	% ASPHALT:	5.4			
FIELD -	DENSITY (kg/m³):	2,368	CORES -	DENSITY:	2,263
	% AIR VOIDS:	3.6		% AIR VOIDS:	8.1
	% VMA:	-		% COMPACTION:	95.5
	% ASPHALT:	5.6			
1987 -	DENSITY (kg/m³):	-		AGE: 5 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		78	52.	28.	-	18.0	14.5	10.9
AS-BUILT	100.		78	52.	28.	-	18.0	15.0	10.9
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	266	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	43.5	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm²/s, 135 C):	-
STIFFNESS (kPa):	49	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -

TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 5.8 mm

RANGE: 3 - 12 mm

TABLE B-16 SITE MATERIALS DATA

SITE: 16 PROJECT: 507:02 KILOMETER: 29.26

PROJECT DESCRIPTION: Jct highway 3 to Jct highway 6

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,397		STABILITY (N):	13,950
	% AIR VOIDS:	3.2		FLOW (mm):	3.0
	% VMA:	13.1			
	% ASPHALT:	5.5			
FIELD -	DENSITY (kg/m³):	2,390	CORES -	DENSITY:	2,266
	% AIR VOIDS:	3.4		% AIR VOIDS:	8.5
	% VMA:	-		% COMPACTION:	94.8
	% ASPHALT:	5.8			
1987 -	DENSITY (kg/m³):	2,307		AGE: 4 years	
	% AIR VOIDS:	7.7			
	% VMA:	15.9			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		89	67.	41.	-	24.0	14.5	10.6
AS-BUILT	100.		92	71.	45.	-	27.0	17.0	10.7
1987	100.	97.	91	69.	42.	34.8	26.3	16.7	11.8

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm,25 C):	160	PENETRATION (dmm,25 C):	66
ABS VISC (Pa.s,60 C):	72.0	ABS VISC (Pa.s,60 C):	326
		KIN VISC (mm²/s,135 C):	422
STIFFNESS (kPa):	127	STIFFNESS (kPa):	686

STRUCTURE		RUT DEPTHS	
TOTAL ASPHALT PAVEMENT THICKNESS :	123 mm	AVERAGE:	3.8 mm
TOTAL BASE THICKNESS :	150 mm	RANGE:	2 - 6 mm

TABLE B-17 SITE MATERIALS DATA

SITE: 17 PROJECT: 16:12 KILOMETER: 21.89

PROJECT DESCRIPTION: WBL - Jct highway 22 to Jct highway 43

MIX DATA

DESIGN - DENSITY (kg/m³): 2,310
% AIR VOIDS: 4.1
% VMA: 15.0
% ASPHALT: 6.1

STABILITY (N): 7,650
FLOW (mm): 2.1

FIELD - DENSITY (kg/m³): 2,288
% AIR VOIDS: 4.7
% VMA: -
% ASPHALT: 6.5

CORES - DENSITY: 2,182
% AIR VOIDS: 9.1
% COMPACTION: 95.4

1987 - DENSITY (kg/m³): 2,305
% AIR VOIDS: 4.8
% VMA: 14.9

AGE: 4 years

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		79	56.	42.	-	22.0	10.8	7.3
AS-BUILT	100.		79	57.	42.	-	23.0	12.0	7.6
1987	99.	93.	83	56.	42.	35.8	24.5	13.0	7.8

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 166
ABS VISC (Pa.s, 60 C): 76.5
STIFFNESS (kPa): 78

ABSON

PENETRATION (dmm, 25 C): 82
ABS VISC (Pa.s, 60 C): 282
KIN VISC (mm²/s, 135 C): 456
STIFFNESS (kPa): 687

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 196 mm
TOTAL BASE THICKNESS : 415 mm

RUT DEPTHS

AVERAGE: 3.5 mm
RANGE: 3 - 4 mm

TABLE B-18 SITE MATERIALS DATA

SITE: 18 PROJECT: 22:30 KILOMETER: 49.88

PROJECT DESCRIPTION: Jct highway 39 to Jct highway 16

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,332		STABILITY (N):	11,650
	% AIR VOIDS:	3.8		FLOW (mm):	2.1
	% VMA:	14.1			
	% ASPHALT:	5.6			
FIELD -	DENSITY (kg/m^3):	2,347	CORES -	DENSITY:	2,221
	% AIR VOIDS:	3.0		% AIR VOIDS:	8.6
	% VMA:	-		% COMPACTION:	94.6
	% ASPHALT:	5.5			
1987 -	DENSITY (kg/m^3):	2,324		AGE: 5 years	
	% AIR VOIDS:	4.1			
	% VMA:	14.6			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		75	46.	35.	26.0	23.0	10.0	6.6
AS-BUILT	100.		80	52.	33.	26.0	20.0	12.0	7.0
1987	100.	95.	82	54.	37.	33.3	22.0	12.9	7.9

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION(dmm,25 C):	284	PENETRATION(dmm,25 C):	126
ABS VISC(Pa.s,60 C):	36.0	ABS VISC(Pa.s,60 C):	121
		KIN VISC(mm^2/s,135 C):	298
STIFFNESS (kPa):	59	STIFFNESS (kPa):	196

STRUCTURE		RUT DEPTHS
TOTAL ASPHALT PAVEMENT THICKNESS :	160 mm	AVERAGE: 2.3 mm
TOTAL BASE THICKNESS :	350 mm	RANGE: 0 - 6 mm

TABLE B-19 SITE MATERIALS DATA

SITE: 19 PROJECT: 22:30 KILOMETER: 19.86

PROJECT DESCRIPTION: Jct highway 39 to Jct highway 16

MIX DATA

DESIGN - DENSITY (kg/m³): 2,380 | STABILITY (N): 12,900
% AIR VOIDS: 3.5 | FLOW (mm): 2.3
% VMA: 13.4
% ASPHALT: 5.1

FIELD - DENSITY (kg/m³): 2,373 | CORES - DENSITY: 2,264
% AIR VOIDS: 3.5 | % AIR VOIDS: 7.8
% VMA: - | % COMPACTION: 95.4
% ASPHALT: 5.4

1987 - DENSITY (kg/m³): - | AGE: 6 years
% AIR VOIDS: -
% VMA: -

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		82	55.	37.	-	24.0	15.3	9.5
AS-BUILT	100.		84	60.	40.	-	25.0	17.0	10.5
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION(dmm,25 C): 311
ABS VISC(Pa.s,60 C): 34.9

STIFFNESS (kPa): 51

ABSON

PENETRATION(dmm,25 C): -
ABS VISC(Pa.s,60 C): -
KIN VISC(mm²/s,135 C): -
STIFFNESS (kPa): -

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 1.0 mm
RANGE: 0 - 2 mm

TABLE B-20 SITE MATERIALS DATA

SITE: 20 PROJECT: 11:12 KILOMETER: 9.56

PROJECT DESCRIPTION: Jct SR 766 to Jct highway 2A

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,332		STABILITY (N):	-
	% AIR VOIDS:	-		FLOW (mm):	-
	% VMA:	-			
	% ASPHALT:	6.1			
FIELD -	DENSITY (kg/m³):	2,332	CORES -	DENSITY:	2,215
	% AIR VOIDS:	-		% AIR VOIDS:	-
	% VMA:	-		% COMPACTION:	95.0
	% ASPHALT:	6.0			
1987 -	DENSITY (kg/m³):	-		AGE: 31 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	94.	85.	78	56.	33.	25.0	15.0	10.0	6.1
AS-BUILT		81.		57.	36.	25.0	18.0	11.0	7.3
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 249
ABS VISC (Pa.s, 60 C): 14.0

STIFFNESS (kPa): 98

ABSON

PENETRATION (dmm, 25 C): -
ABS VISC (Pa.s, 60 C): -
KIN VISC (mm²/s, 135 C): -
STIFFNESS (kPa): -

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 4.5 mm
RANGE: 2 - 8 mm

TABLE B-21 SITE MATERIALS DATA

SITE: 21 PROJECT: 11:10 KILOMETER: 14.67

PROJECT DESCRIPTION: Jct highway 11A to Jct SR 766

MIX DATA

DESIGN - DENSITY (kg/m³): 2,353
% AIR VOIDS: -
% VMA: -
% ASPHALT: 6.5

STABILITY (N): -
FLOW (mm): -

FIELD - DENSITY (kg/m³): 2,353
% AIR VOIDS: -
% VMA: -
% ASPHALT: 6.5

CORES - DENSITY: 2,265
% AIR VOIDS: -
% COMPACTION: 96.3

1987 - DENSITY (kg/m³): 2,376
% AIR VOIDS: -
% VMA: -

AGE: 30 years

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	90.	77.	65	47.	32.	-	23.0	14.0	7.0
AS-BUILT						-	-	-	-
1987	97.	89.	77	54.	35.	31.0	22.5	13.9	10.1

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 190
ABS VISC (Pa.s, 60 C): 30.0
STIFFNESS (kPa): 176

ABSON

PENETRATION (dmm, 25 C): 84
ABS VISC (Pa.s, 60 C): 289
KIN VISC (mm²/s, 135 C): 480
STIFFNESS (kPa): 471

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 175 mm
TOTAL BASE THICKNESS : 332 mm

RUT DEPTHS

AVERAGE: 7.5 mm
RANGE: 6 - 11 mm

TABLE B-22 SITE MATERIALS DATA

SITE: 22 PROJECT: 11:08 KILOMETER: 38.94

PROJECT DESCRIPTION: Forest Reserve to Jct highway 11A

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,353		STABILITY (N):	7,200
	% AIR VOIDS:	4.4		FLOW (mm):	2.6
	% VMA:	14.1			
	% ASPHALT:	4.7			
FIELD -	DENSITY (kg/m^3):	2,409	CORES -	DENSITY:	2,309
	% AIR VOIDS:	2.6		% AIR VOIDS:	6.7
	% VMA:	-		% COMPACTION:	95.9
	% ASPHALT:	4.8			
1987 -	DENSITY (kg/m^3):	-		AGE: 13 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	90.	81	55.	29.	25.0	17.0	13.0	8.8
AS-BUILT	100.		78	54.	32.	25.0	21.0	16.0	10.0
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	268	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	45.0	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm^2/s, 135 C):	-
STIFFNESS (kPa):	68	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -

TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 1.0 mm

RANGE: 0 - 2 mm

TABLE B-23 SITE MATERIALS DATA

SITE: 23 PROJEC 1:08 KILOMETER: 2S.27

PROJECT DESCRIPTION: Forest reserve to Jct highway 11A

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,380		STABILITY (N):	12,000
	% AIR VOIDS:	3.6		FLOW (mm):	2.4
	% VMA:	14.1			
	% ASPHALT:	5.7			
FIELD -	DENSITY (kg/m^3):	2,369	CORES -	DENSITY:	2,257
	% AIR VOIDS:	3.6		% AIR VOIDS:	8.3
	% VMA:	-		% COMPACTION:	95.3
	% ASPHALT:	5.8			
1987 -	DENSITY (kg/m^3):	-		AGE: 11 years	
	% AIR VOIDS	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	88.	77	55.	31.	26.0	17.0	12.0	6.5
AS-BUILT	100.		82	58.	34.	26.0	21.0	15.0	8.2
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	287	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	35.0	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm^2/s, 135 C):	-
STIFFNESS (kPa):	59	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -

TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 3.5 mm

RANGE: 2 - 6 mm

TABLE B-24 SITE MATERIALS DATA

SITE: 24 PROJECT: 11:06 KILOMETER: 18.43

PROJECT DESCRIPTION: North of Nordegg to forest reserve

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,340		STABILITY (N):	9,650
	% AIR VOIDS:	4.3		FLOW (mm):	3.0
	% VMA:	14.5			
	% ASPHALT:	6.2			
FIELD -	DENSITY (kg/m³):	2,360	CORES -	DENSITY:	2,277
	% AIR VOIDS:	3.7		% AIR VOIDS:	7.1
	% VMA:	-		% COMPACTION:	96.5
	% ASPHALT:	6.1			
1987 -	DENSITY (kg/m³):	-		AGE: 9 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	87.	75	51.	29.	21.0	17.0	13.0	10.1
AS-BUILT	100.		71	46.	26.	21.0	17.0	15.0	11.0
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	300	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	33.7	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm²/s, 135 C):	-
STIFFNESS (kPa):	57	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 1.5 mm
RANGE: 0 - 4 mm

TABLE B-25 SITE MATERIALS DATA

SITE: 25 PROJECT: 11A:02 KILOMETER: 3.18

PROJECT DESCRIPTION: Jct highway 11 to Rocky Mountain House

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,344		STABILITY (N):	.400
	% AIR VOIDS:	3.7		FLOW (mm):	2.3
	% VMA:	15.9			
	% ASPHALT:	5.8			
FIELD -	DENSITY (kg/m^3):	2,373	CORES -	DENSITY:	2,325
	% AIR VOIDS:	2.4		% AIR VOIDS:	4.4
	% VMA:	14.9		% COMPACTION:	98.5
	% ASPHALT:	5.9			
1987 -	DENSITY (kg/m^3):	2,403		AGE: 17 years	
	% AIR VOIDS:	1.9			
	% VMA:	13.3			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	90.	78	58.	38.	30.0	22.0	15.0	8.5
AS-BUILT	100.		80	60.	36.	30.0	24.0	17.0	8.6
1987	99.	93.	83	60.	41.	34.7	26.3	16.9	12.5

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION(dmm,25 C): 274
ABS VISC(Pa.s,60 C): 38.1
STIFFNESS (kPa): 58

ABSON

PENETRATION(dmm,25 C): 129
ABS VISC(Pa.s,60 C): 118
KIN VISC(mm^2/s,135 C): 276
STIFFNESS (kPa): 197

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 140 mm
TOTAL BASE THICKNESS : 124 mm

RUT DEPTHS

AVERAGE: 3.3 mm
RANGE: 0 - 6 mm

TABLE B-26 SITE MATERIALS DATA

SITE: 26 PROJECT: 2A:18 KILOMETER: 26.00

PROJECT DESCRIPTION: South of Red Deer to Lacombe

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,281		STABILITY (N):	5,500
	% AIR VOIDS:	4.1		FLOW (mm):	2.5
	% VMA:	16.2			
	% ASPHALT:	6.3			
FIELD -	DENSITY (kg/m³):	2,281	CORES -	DENSITY:	2,193
	% AIR VOIDS:	4.1		% AIR VOIDS:	-
	% VMA:	16.2		% COMPACTION:	96.2
	% ASPHALT:	-			
1987 -	DENSITY (kg/m³):	2,332		AGE: 29 years	
	% AIR VOIDS:	3.1			
	% VMA:	13.5			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	93.	84.	75	55.	31.	18.0	12.0	10.0	7.6
AS-BUILT		81.		56.	30.	18.0	13.0	9.0	7.0
1987	98.	90.	78	55.	34.	24.8	15.3	10.9	8.7

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	170	PENETRATION (dmm, 25 C):	71
ABS VISC (Pa.s, 60 C):	20.0	ABS VISC (Pa.s, 60 C):	180
		KIN VISC (mm²/s, 135 C):	302
STIFFNESS (kPa):	196	STIFFNESS (kPa):	589

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 144 mm

TOTAL BASE THICKNESS : 407 mm

RUT DEPTHS

AVERAGE: 7.8 mm

RANGE: 5 - 10 mm

TABLE B-27 SITE MATERIALS DATA

SITE: 27 PROJECT: 12:20 KILOMETER: 34.18

PROJECT DESCRIPTION: Jct highway 41 to the Saskatchewan boundary

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,331	STABILITY (N):	8,270
	% AIR VOIDS:	4.3	FLOW (mm):	1.8
	% VMA:	14.5		
	% ASPHALT:	6.2		
FIELD -	DENSITY (kg/m^3):	2,357.0	CORES - DENSITY:	2,212
	% AIR VOIDS:	3.2	% AIR VOIDS:	9.2
	% VMA:	-	% COMPACTION:	93.8
	% ASPHALT:	6.2		
1987 -	DENSITY (kg/m^3):	2,289.5	AGE:	9 years
	% AIR VOIDS:	6.3		
	% VMA:	15.8		

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	92.	82	64.	34.	26.0	17.0	11.0	6.7
AS-BUILT	100.		84	65.	34.	26.0	20.0	12.5	8.1
1987	99.	92.	85	64.	33.	25.5	18.5	11.1	6.9

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 182
ABS VISC (Pa.s, 60 C): 69.7

STIFFNESS (kPa): 98

ABSON

PENETRATION (dmm, 25 C): 76
ABS VISC (Pa.s, 60 C): 308
KIN VISC (mm^2/s, 135 C): 432
STIFFNESS (kPa): 392

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 135 mm
TOTAL BASE THICKNESS : 166 mm

RUT DEPTHS

AVERAGE: 3.0 mm
RANGE: 0 - 7 mm

TABLE B-28 SITE MATERIALS DATA

SITE: 28 PROJECT: 12:20 KILOMETER: 7.95

PROJECT DESCRIPTION: Jct highway 41 to the Saskatchewan boundary

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,337		STABILITY (N):	4,220
	% AIR VOIDS:	3.9		FLOW (mm):	2.3
	% VMA:	14.7			
	% ASPHALT:	5.6			
FIELD -	DENSITY (kg/m^3):	2,368	CORES -	DENSITY:	2,220
	% AIR VOIDS:	2.8		% AIR VOIDS:	8.8
	% VMA:	-		% COMPACTION:	93.8
	% ASPHALT:	5.7			
1987 -	DENSITY (kg/m^3):	2,344		AGE: 13 years	
	% AIR VOIDS:	4.5			
	% VMA:	13.5			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	92.	83	63.	34.	25.0	18.0	13.0	7.3
AS-BUILT	100.		85	66.	34.	25.0	19.0	13.0	7.0
1987	100.	96.	89	66.	35.	27.0	19.0	10.6	6.6

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	270	PENETRATION (dmm, 25 C):	75
ABS VISC (Pa.s, 60 C):	44.8	ABS VISC (Pa.s, 60 C):	322
		KIN VISC (mm^2/s, 135 C):	452
STIFFNESS (kPa):	49	STIFFNESS (kPa):	490

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 102 mm
TOTAL BASE THICKNESS : 239 mm

RUT DEPTHS

AVERAGE: 2.5 mm
RANGE: 0 - 7 mm

TABLE B-29 SITE MATERIALS DATA

SITE: 29 PROJECT: 9:14 KILOMETER: 10.04

PROJECT DESCRIPTION: Jct highway 41 to the Saskatchewan boundary

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,300		STABILITY (N):	4,400
	% AIR VOIDS:	4.0		FLOW (mm):	1.8
	% VMA:	15.7			
	% ASPHALT:	6.0			
FIELD -	DENSITY (kg/m³):	2,330	CORES -	DENSITY:	2,264
	% AIR VOIDS:	-		% AIR VOIDS:	5.6
	% VMA:	-		% COMPACTION:	98.4
	% ASPHALT:	5.8			
1987 -	DENSITY (kg/m³):	-		AGE: 20 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	90.	79	58.	40.	18.0	10.0	9.0	6.6
AS-BUILT	100.		80	60.	28.	18.0	12.0	8.0	5.7
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	248	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	42.3	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm²/s, 135 C):	-
STIFFNESS (kPa):	59	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : -

TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 2.8 mm

RANGE: 0 - 6 mm

TABLE B-30 SITE MATERIALS DATA

SITE: 30 PROJECT: 49:02 KILOMETER: 1.40

PROJECT DESCRIPTION: B.C. boundary to Park Road 1:2

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,384		STABILITY (N):	8,500
	% AIR VOIDS:	3.3		FLOW (mm):	2.1
	% VMA:	14.2			
	% ASPHALT:	5.4			
FIELD -	DENSITY (kg/m³):	2,362	CORES -	DENSITY:	2,302
	% AIR VOIDS: 4.1	4.1		% AIR VOIDS:	6.3
	% VMA:	25.1		% COMPACTION:	97.5
	% ASPHALT:	5.5			
1987 -	DENSITY (kg/m³):	2,313		AGE: 6 years	
	% AIR VOIDS:	6.8			
	% VMA:	16.4			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.		82	56.	32.	-	16.0	8.8	6.4
AS-BUILT	100.		81	51.	31.	-	15.0	9.0	6.6
1987	99.	92.	78	50.	31.	26.0	17.5	10.2	7.3

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	278	PENETRATION (dmm, 25 C):	99
ABS VISC (Pa.s, 60 C):	39.7	ABS VISC (Pa.s, 60 C):	184
		KIN VISC (mm²/s, 135 C):	345
STIFFNESS (kPa):	89	STIFFNESS (kPa):	510

STRUCTURE

TOTAL ASPHALT PAVEMENT TH'CKNESS : 121 mm

TOTAL BASE THICKNESS : 140 mm

RUT DEPTHS

AVERAGE: 5.0 mm

RANGE: 3 - 6 mm

TABLE B-31 SITE MATERIALS DATA

SITE: 31 PROJECT: 49:04 KILOMETER: 18.39

PROJECT DESCRIPTION: Park Road 112 to Jct highway 2

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,356		STABILITY (N):	3,505
	% AIR VOIDS:	3.8		FLOW (mm):	1.9
	% VMA:	14.0			
	% ASPHALT:	5.0			
FIELD -	DENSITY (kg/m³):	2,356	CORES -	DENSITY:	2,295
	% AIR VOIDS:	-		% AIR VOIDS:	6.9
	% VMA:	-		% COMPACTION:	97.4
	% ASPHALT:	4.6			
1987 -	DENSITY (kg/m³):	2,365		AGE: 18 years	
	% AIR VOIDS:	3.7			
	% VMA:	13.7			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	88.		64	42.	22.	17.5	10.0	8.0	5.7
AS-BUILT	100.	70.	75	49.	26.	17.5	12.5	9.0	5.6
1987	94.	85.	74	50.	28.	21.0	14.2	9.6	7.1

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	238	PENETRATION (dmm, 25 C):	91
ABS VISC (Pa.s, 60 C):	29.8	ABS VISC (Pa.s, 60 C):	143
		KIN VISC (mm²/s, 135 C):	290
STIFFNESS (kPa):	98	STIFFNESS (kPa):	490

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 187 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 3.8 mm
RANGE: 0 - 8 mm

TABLE B-32 SITE MATERIALS DATA

SITE: 32 PROJECT: 35:12 KILOMETER: 42.18

PROJECT DESCRIPTION: Keg River to North boundary of metis colony

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,329		STABILITY (N):	4,000
	% AIR VOIDS:	4.6		FLOW (mm):	1.7
	% VMA:	15.4			
	% ASPHALT:	5.3			
FIELD -	DENSITY (kg/m³):	2,342	CORES -	DENSITY:	2,280
	% AIR VOIDS:	3.8		% AIR VOIDS:	6.5
	% VMA:	-		% COMPACTION:	97.3
	% ASPHALT:	5.3			
1987 -	DENSITY (kg/m³):	2,342		AGE: 12 years	
	% AIR VOIDS:	5.1			
	% VMA:	14.5			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	88.	77	60.	38.	23.0	12.0	7.5	5.1
AS-BUILT	99.		74	55.	36.	23.0	14.0	10.0	5.6
1987	99.	8	76	56.	41.	30.8	16.5	9.0	6.5

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION(dmm,25 C):	286
ABS VISC(Pa.s,60 C):	23.7
STIFFNESS (kPa):	68

ABSON

PENETRATION(dmm,25 C):	76
ABS VISC(Pa.s,60 C):	142
KIN VISC(mm²/s,135 C):	273
STIFFNESS (kPa):	785

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 156 mm
TOTAL BASE THICKNESS : 165 mm

RUT DEPTHS

AVERAGE: 4.0 mm
RANGE: 2 - 7 mm

TABLE B-33 SITE MATERIALS DATA

SITE: 33 PROJECT: 35:16 KILOMETER: 18.20

PROJECT DESCRIPTION: Jct highway 58 to Meander river

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,316		STABILITY (N):	5,300
	% AIR VOIDS:	6.5		FLOW (mm):	2.6
	% VMA:	17.3			
	% ASPHALT:	6.3			
FIELD -	DENSITY (kg/m³):	2,381	CORES -	DENSITY:	2,292
	% AIR VOIDS:	4.1		% AIR VOIDS:	7.6
	% VMA:	14.5		% COMPACTION:	96.3
	% ASPHALT:	5.9			
1987 -	DENSITY (kg/m³):	2,308		AGE: 10 years	
	% AIR VOIDS:	7.3			
	% VMA:	17.3			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	87.	75	50.	23.	17.0	10.0	8.0	6.1
AS-BUILT	100.	90.	78	52.	24.	17.0	12.0	9.0	8.0
1987	98.	90.	79	54.	27.	18.0	13.8	11.0	9.1

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	222	PENETRATION (dmm, 25 C):	109
ABS VISC (Pa.s, 60 C):	34.6	ABS VISC (Pa.s, 60 C):	91.4
		KIN VISC (mm²/s, 135 C):	235
STIFFNESS (kPa):	167	STIFFNESS (kPa):	294

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 192 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 2.8 mm
RANGE: 1 - 4 mm

TABLE B-34 SITE MATERIALS DATA

SITE: 34 PROJECT: 881:12 KILOMETER: 11.19

PROJECT DESCRIPTION: North Saskatchewan River to Jct highway 28

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,313		STABILITY (N):	7,810
	% AIR VOIDS:	4.4		FLOW (mm):	1.5
	% VMA:	14.5			
	% ASPHALT:	5.3			
FIELD -	DENSITY (kg/m³):	2,340	CORES -	DENSITY:	2,166
	% AIR VOIDS:	2.7		% AIR VOIDS:	9.9
	% VMA:	-		% COMPACTION:	92.5
	% ASPHALT:	5.6			
1987 -	DENSITY (kg/m³):	-		AGE: 9 years	
	% AIR VOIDS:	-			
	% VMA:	-			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	93.	87	70.	33.	19.0	14.0	11.5	9.4
AS-BUILT	100.		86	65.	31.	19.0	13.0	9.0	7.9
1987						-	-	-	-

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	268	PENETRATION (dmm, 25 C):	-
ABS VISC (Pa.s, 60 C):	43.8	ABS VISC (Pa.s, 60 C):	-
		KIN VISC (mm²/s, 135 C):	-
STIFFNESS (kPa):	49	STIFFNESS (kPa):	-

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 156 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 5.0 mm
RANGE: 0 - 10 mm

TABLE B-35 SITE MATERIALS DATA

SITE: 35 PROJECT: 881:12 KILOMETER: 9.70

PROJECT DESCRIPTION: North Saskatchewan River to Jct highway 28

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,326	STABILITY (N):	6,150
	% AIR VOIDS:	4.3	FLOW (mm):	1.
	% VMA:	15.9		
	% ASPHALT:	5.9		
FIELD -	DENSITY (kg/m³):	2,325	CORES - DENSITY:	2,208
	% AIR VOIDS:	4.0	% AIR VOIDS:	8.9
	% VMA:	-	% COMPACTION:	94.9
	% ASPHALT:	6.0		
1987 -	DENSITY (kg/m³):	2,288	AGE: 7 years	
	% AIR VOIDS:	5.9		
	% VMA:	17.1		

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	·	315	160	80
DESIGN	100.		81	64.	32.	22.0	15.0	11.4	9.0
AS-BUILT	99.	87.	84	65.	37.	22.0	16.0	13.0	9.1
1987	97.	90.	83	66.	36.	23.5	13.5	9.6	7.4

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	270	PENETRATION (dmm, 25 C):	119
ABS VISC (Pa.s, 60 C):	42.0	ABS VISC (Pa.s, 60 C):	162
		KIN VISC (mm²/s, 135 C):	352
STIFFNESS (kPa):	49	STIFFNESS (kPa):	392

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 137 mm
TOTAL BASE THICKNESS : 120 mm

RUT DEPTHS

AVERAGE: 4.0 mm
RANGE: 1 - 6 mm

TABLE B-36 SITE MATERIALS DATA

SITE: 36 PROJECT: 2:38 KILOMETER: 22.59

PROJECT DESCRIPTION: Jct highway 18 to west of Rochester

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,307	STABILITY (N):	4,600
	% AIR VOIDS:	3.9	FLOW (mm):	2.0
	% VMA:	15.8		
	% ASPHALT:	-		
FIELD -	DENSITY (kg/m³):	2,307	CORES - DENSITY:	2,229
	% AIR VOIDS:	-	% AIR VOIDS:	7.5
	% VMA:	-	% COMPACTION:	96.7
	% ASPHALT:	6.1		
1987 -	DENSITY (kg/m³):	2,344	AGE: 21 years	
	% AIR VOIDS:	2.6		
	% VMA:	14.6		

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	88.	78	56.	34.	21.0	12.0	9.0	7.2
AS-BUILT	100.		84	62.	46.	21.0	12.0	8.0	5.4
1987	99.	94.	85	63.	35.	23.0	14.5	9.5	6.8

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 280
ABS VISC (Pa.s, 60 C): 40.1

STIFFNESS (kPa): 59

ABSON

PENETRATION (dmm, 25 C): 34
ABS VISC (Pa.s, 60 C): 161
KIN VISC (mm²/s, 135 C): 357
STIFFNESS (kPa): 392

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 117 mm
TOTAL BASE THICKNESS : 147 mm

RUT DEPTHS

AVERAGE: 5.8 mm
RANGE: 2 - 9 mm

TABLE B-37 SITE MATERIALS DATA

SITE: 37 PROJECT: 2:40 KILOMETER: 38.58

PROJECT DESCRIPTION: West of Rochester to Athabasca

MIX DATA

DESIGN - DENSITY (kg/m³): 2,305 | STABILITY (N): 6,800
% AIR VOIDS: 4.3 | FLOW (mm): 2.2
% VMA: 16.9
% ASPHALT: 6.4

FIELD - DENSITY (kg/m³): 2,271 | CORES - DENSITY: 2,232
% AIR VOIDS: - | % AIR VOIDS: 6.8
% VMA: - | % COMPACTION: 98.0
% ASPHALT: 6.6

1987 - DENSITY (kg/m³): 2,296 | AGE: 19 years
% AIR VOIDS: 4.2
% VMA: 17.5

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	90.	77	54.	37.	25.0	28.0	19.0	10.0
AS-BUILT	100.		81	60.	38.	25.0	18.0	12.0	8.0
1987	97.	89.	79	62.	42.	32.5	21.0	14.3	10.8

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION(dmm,25 C): 258
ABS VISC(Pa.s,60 C): 29.9
STIFFNESS (kPa): 78

ABSON

PENETRATION(dmm,25 C): 130
ABS VISC(Pa.s,60 C): 82.2
KIN VISC(mm²/s,135 C): 271
STIFFNESS (kPa): 170

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 103 mm
TOTAL BASE THICKNESS : 115 mm

RUT DEPTHS

AVERAGE: 3.5 mm
RANGE: 0 - 5 mm

TABLE B-38 SITE MATERIALS DATA

SITE: 38 PROJECT: 63:01 KILOMETER: 37.39

PROJECT DESCRIPTION: North of Newbrook to Jct highway 55

MIX DATA

DESIGN -	DENSITY (kg/m^3):	2,311		STABILITY (N):	6,200
	% AIR VOIDS:	4.5		FLOW (mm):	2.4
	% VMA:	17.2			
	% ASPHALT:	6.6			
FIELD -	DENSITY (kg/m^3):	2,373	CORES	DENSITY:	2,242
	% AIR VOIDS:	2.6		% AIR VOIDS:	8.0
	% VMA:	-		% COMPACTION:	97.0
	% ASPHALT:	6.2			
1987 -	DENSITY (kg/m^3):	2,328		AGE: 16 years	
	% AIR VOIDS:	3.6			
	% VMA:	16.8			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	93.	84	63.	38.	28.0	22.0	15.0	10.3
AS-BUILT	100.		83	62.	39.	28.0	18.0	13.0	8.5
1987	100.	96.	89	68.	45.	34.3	23.7	16.9	12.7

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 267
ABS VISC (Pa.s, 60 C): 43.1

STIFFNESS (kPa): 59

ABSON

PENETRATION (dmm, 25 C): 177
ABS VISC (Pa.s, 60 C): 80.0
KIN VISC (mm^2/s, 135 C): 262
STIFFNESS (kPa): 137

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 73 mm
TOTAL BASE THICKNESS : 223 mm

RUT DEPTHS

AVERAGE: 7.0 mm
RANGE: 4 - 10 mm

TABLE B-39 SITE MATERIALS DATA

SITE: 39 PROJECT: 2:32 KILOMETER: 2.55

PROJECT DESCRIPTION: SBL - Jct highway 39 to Edmonton south city limit

MIX

DESIGN - DENSITY (kg/m³): 2,322
% AIR VOIDS: 4.5
% VMA: 14.6
% ASPHALT: 5.9

STABILITY (N): 9,600
FLOW (mm): 2.1

FIELD - DENSITY (kg/m³): 2,352
% AIR VOIDS: 3.2
% VMA: 13.5
% ASPHALT: 5.9

CORES - DENSITY: 2,270
% AIR VOIDS: 6.6
% COMPACTION: 96.5

1987 - DENSITY (kg/m³): 2,326
% AIR VOIDS: 4.5
% VMA: 14.4

AGE: 8 years

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	95.	83.	71	56.	38.	32.5	26.0	15.5	10.1
AS-BUILT		92.	86	67.	41.	32.5	24.5	16.0	10.0
1987	97.	88.	76	55.	40.	35.8	24.0	14.5	9.9

ASPHALT RHEOLOGY

ORIGINAL

PENETRATION (dmm, 25 C): 230
ABS VISC (Pa.s, 60 C): 54.5

STIFFNESS (kPa): 43

ABSON

PENETRATION (dmm, 25 C): 194
ABS VISC (Pa.s, 60 C): 84.6
KIN VISC (mm^2/s, 135 C): 264
STIFFNESS (kPa): 180

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 310 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 28.5 mm
RANGE: 28 - 29 mm

TABLE B-40 SITE MATERIALS DATA

SITE: 40 PROJECT: 520:02 KILOMETER: 44.81

PROJECT DESCRIPTION: Jct highway 22 to Jct highway 2

MIX DATA

DESIGN -	DENSITY (kg/m³):	2,315		STABILITY (N):	4,870
	% AIR VOIDS:	4.0		FLOW (mm):	2.4
	% VMA:	15.2			
	% ASPHALT:	8			
FIELD -	DENSITY (kg/m³):	2,369	CORES -	DENSITY:	2,234
	% AIR VOIDS:	1.9		% AIR VOIDS:	7.0
	% VMA:	-		% COMPACTION:	94.3
	% ASPHALT:	-.7			
1987 -	DENSITY (kg/m³):	2,211		AGE: 12 years	
	% AIR VOIDS:	9.2			
	% VMA:	18.3			

AGGREGATE

SIEVE	16000	12500	10000	5000	1250	630	315	160	80
DESIGN	100.	87.	72	47.	26.	24.0	14.0	11.7	9.5
AS-BUILT	100.		83	62.	33.	24.0	18.0	13.0	11.4
1987	99.	95.	86	61.	33.	25.7	20.0	15.7	12.1

ASPHALT RHEOLOGY

ORIGINAL		ABSON	
PENETRATION (dmm, 25 C):	270	PENETRATION (dmm, 25 C):	61
ABS VISC (Pa.s, 60 C):	44.1	ABS VISC (Pa.s, 60 C):	471
		KIN VISC (mm²/s, 135 C):	532
STIFFNESS (kPa):	49	STIFFNESS (kPa):	667

STRUCTURE

TOTAL ASPHALT PAVEMENT THICKNESS : 173 mm
TOTAL BASE THICKNESS : -

RUT DEPTHS

AVERAGE: 3.0 mm
RANGE: 0 - 9 mm

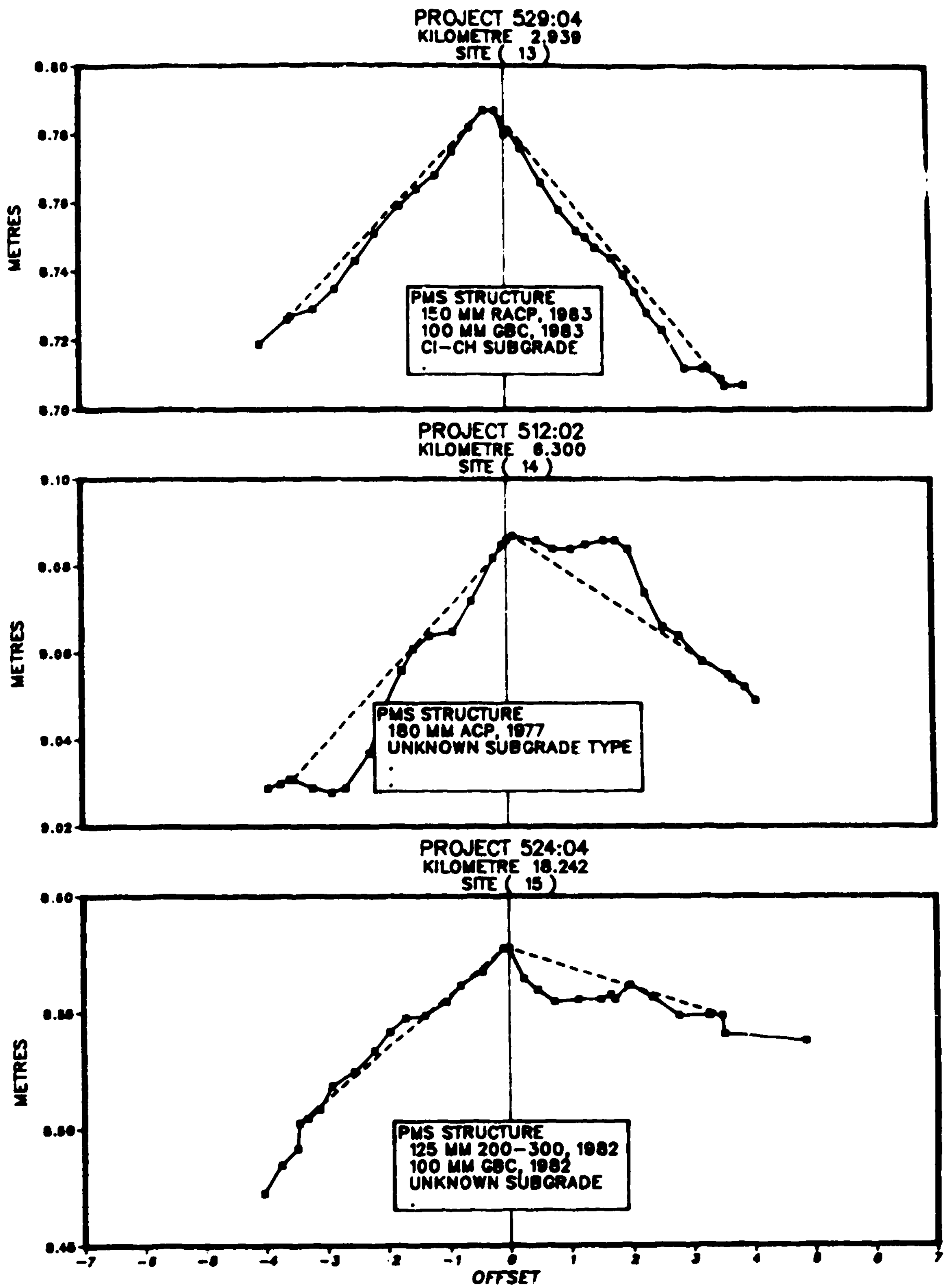

FIGURE B-1 SITE PROFILES (1 - 3)

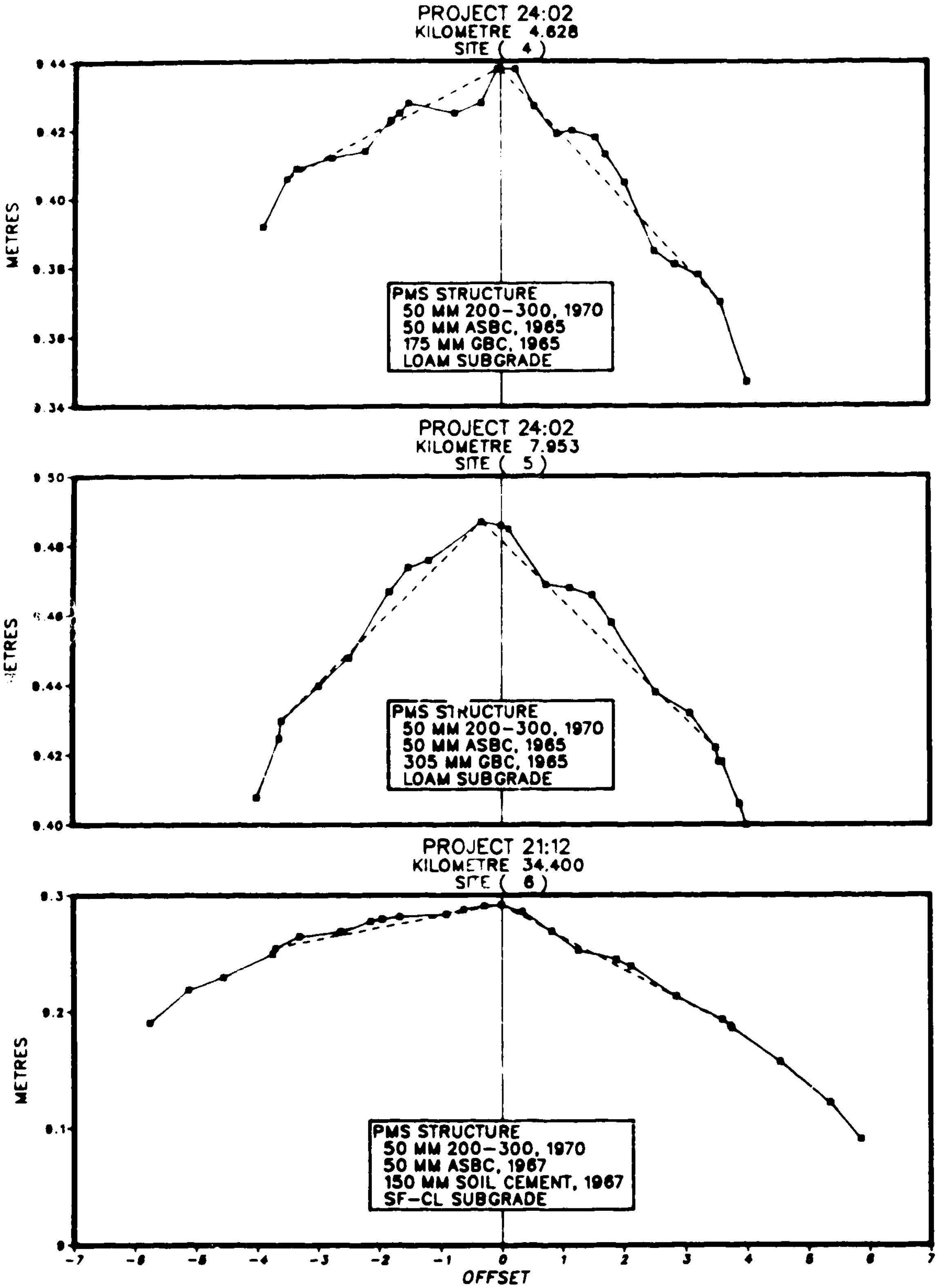

FIGURE B-2 SITE PROFILES (4 - 6)

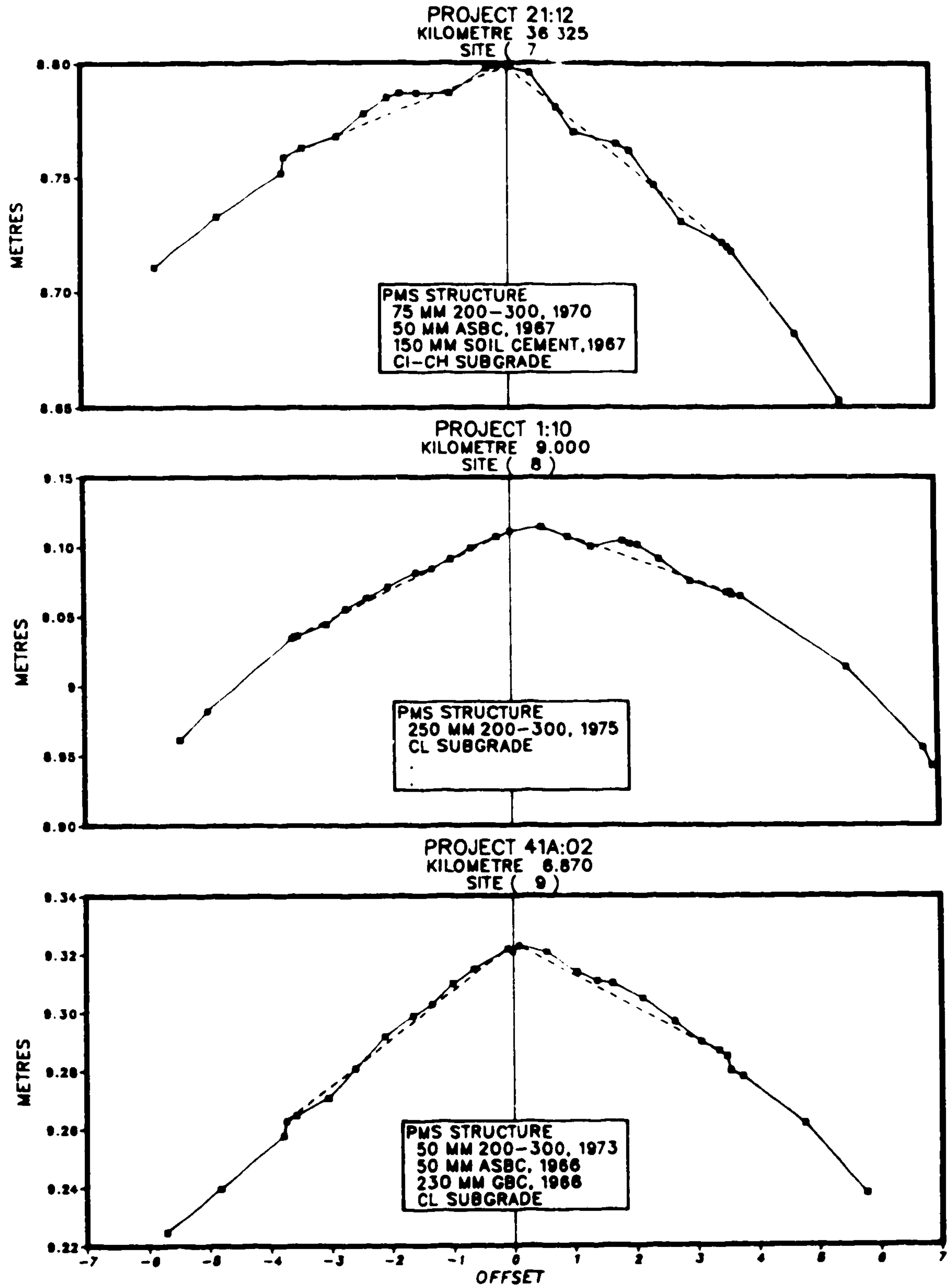

FIGURE B-3 SITE PROFILES (7 - 9)

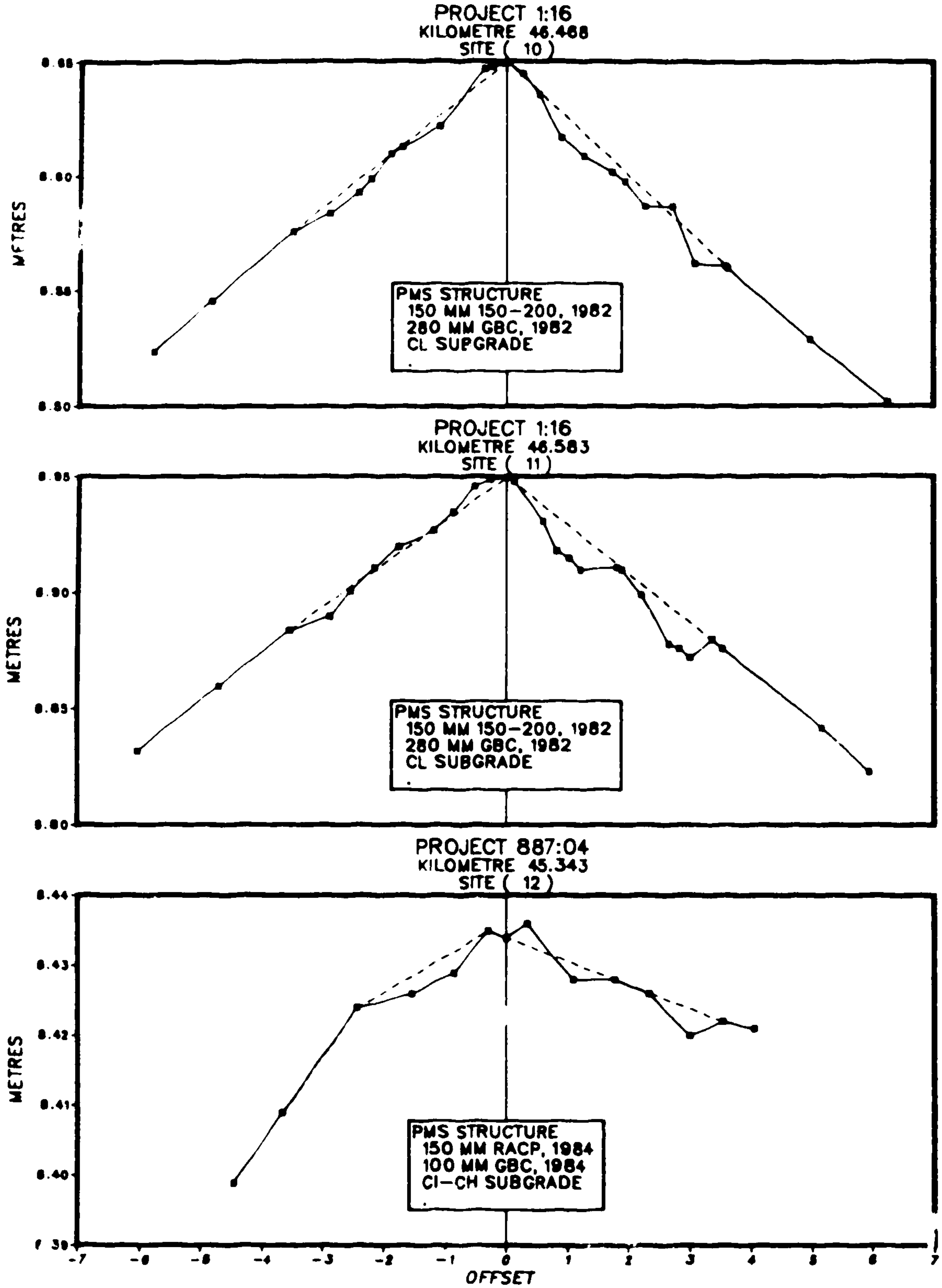

FIGURE B-4 SITE PROFILES (10 - 12)

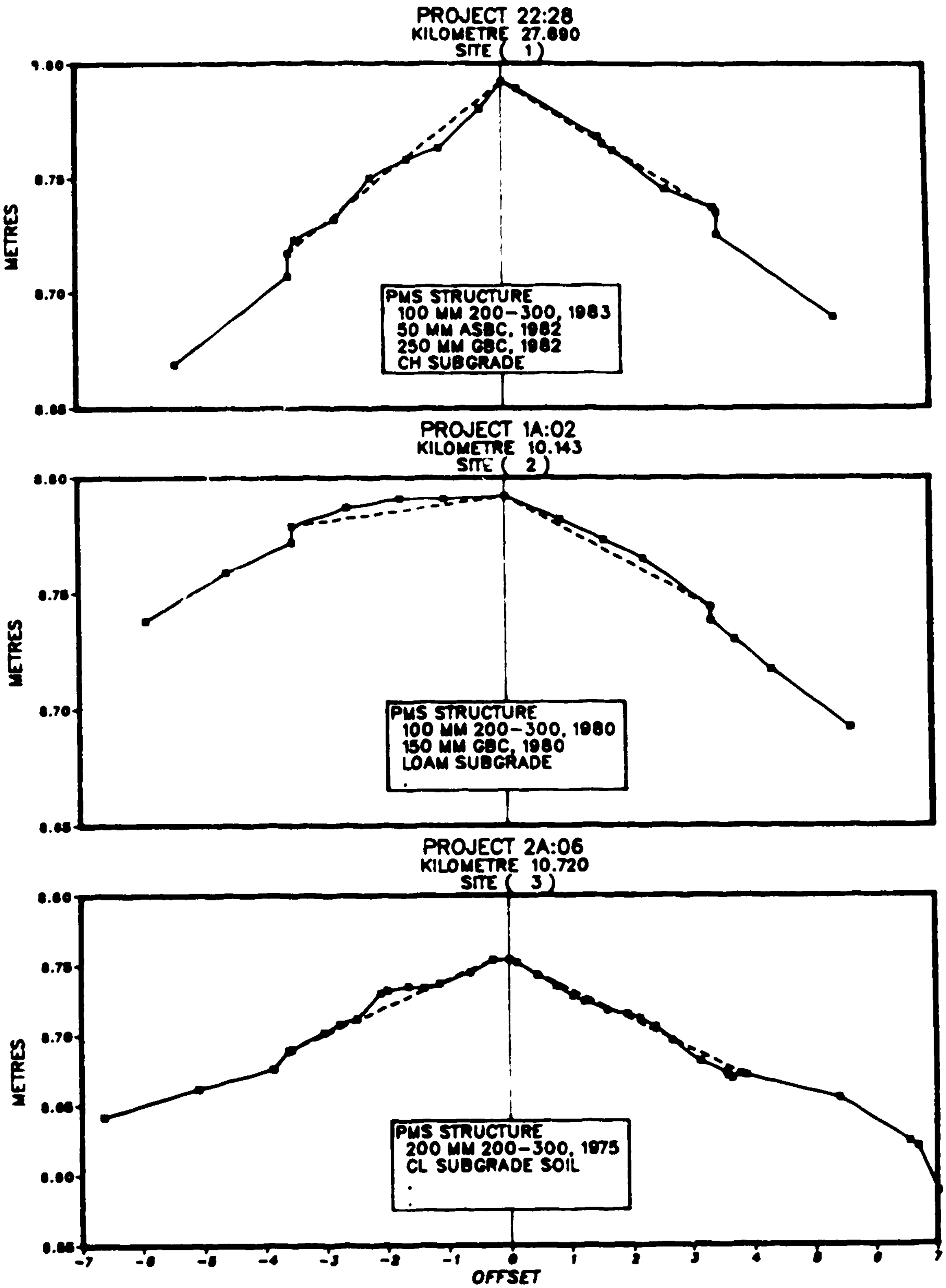

FIGURE B-5 S _ PROFILES (13 - 15)

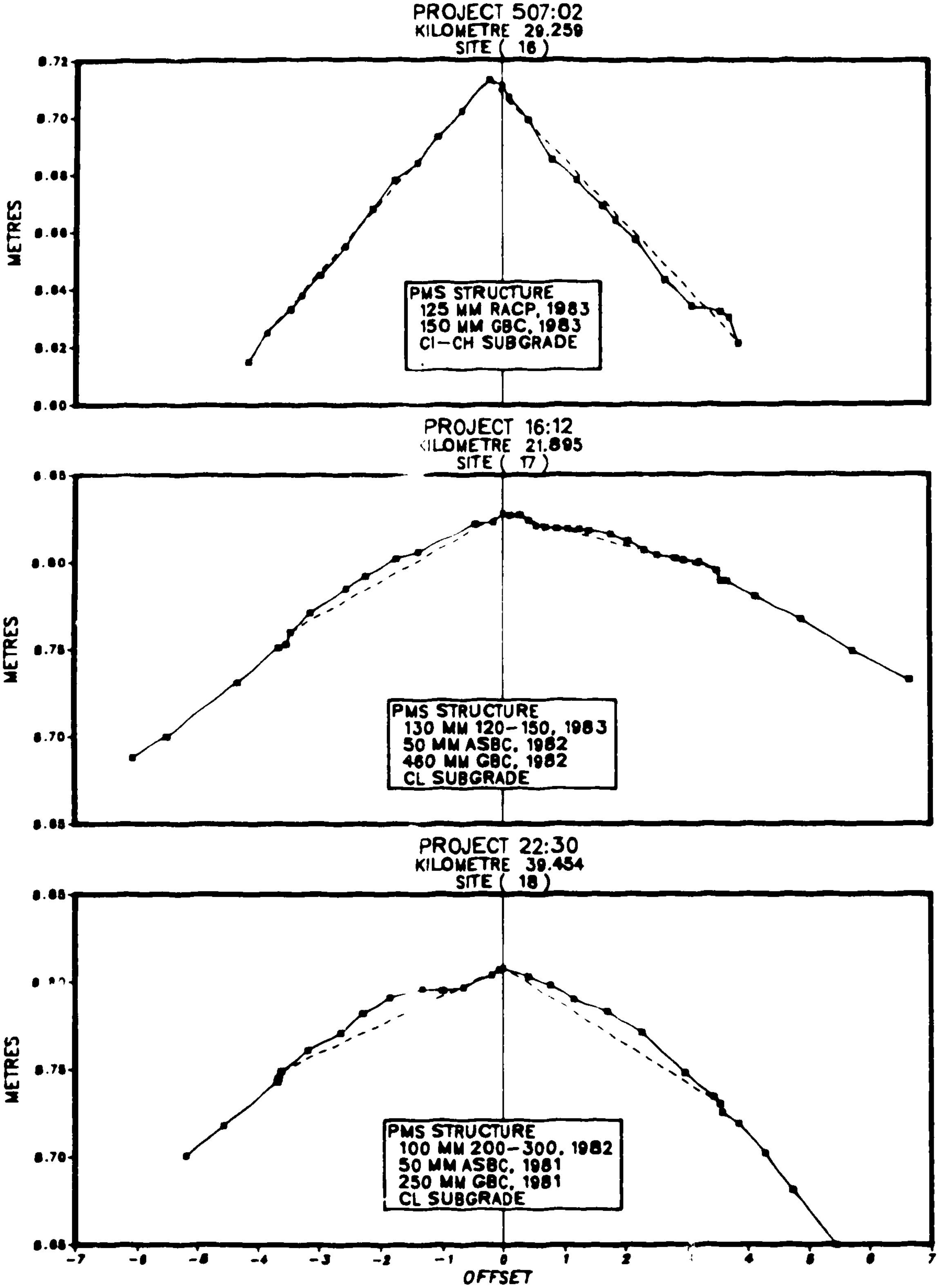

FIGURE B-6 SITE PROFILES (16 - 18)

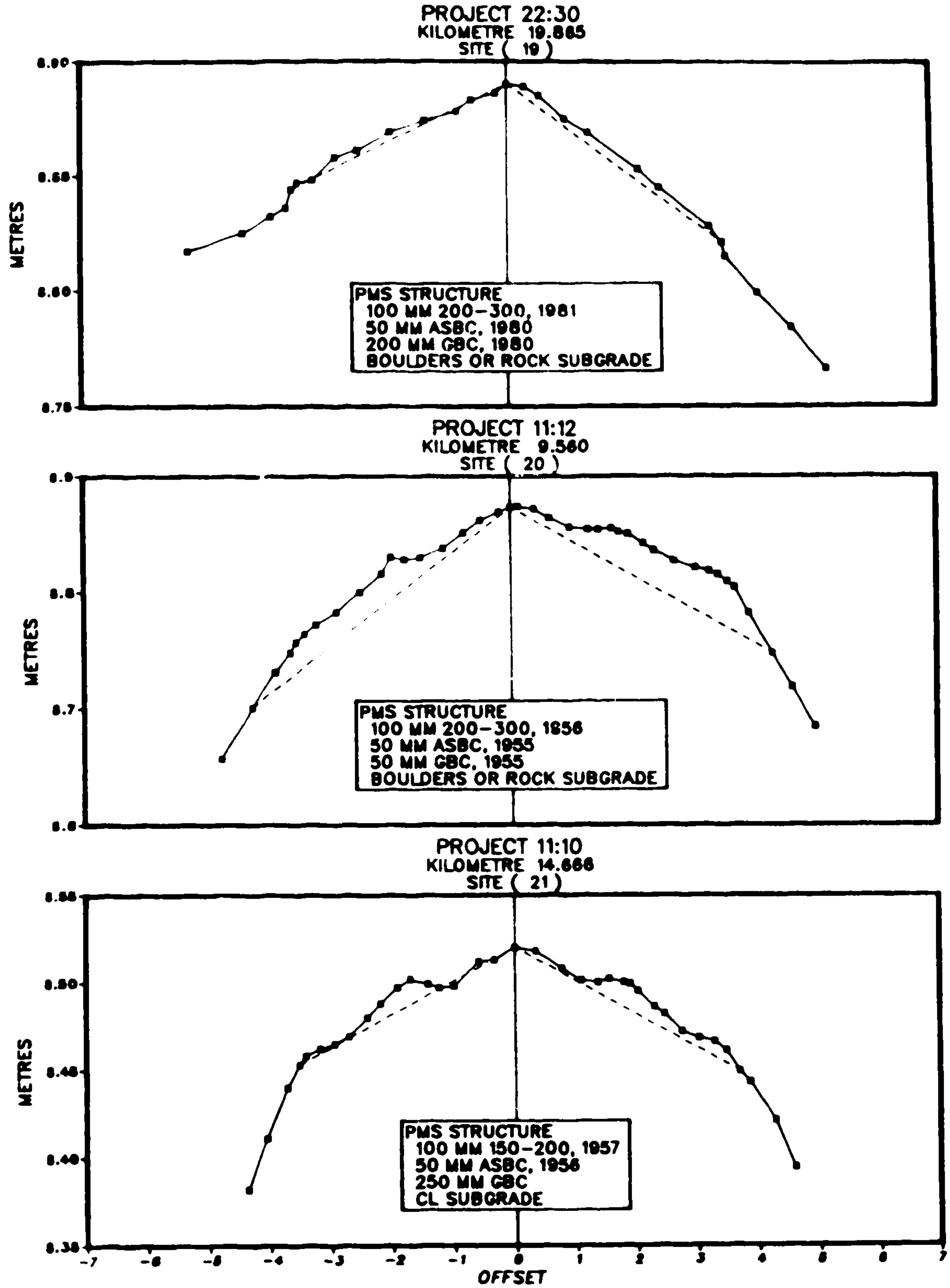

FIGURE B-7 SITE PROFILES (19 - 21)

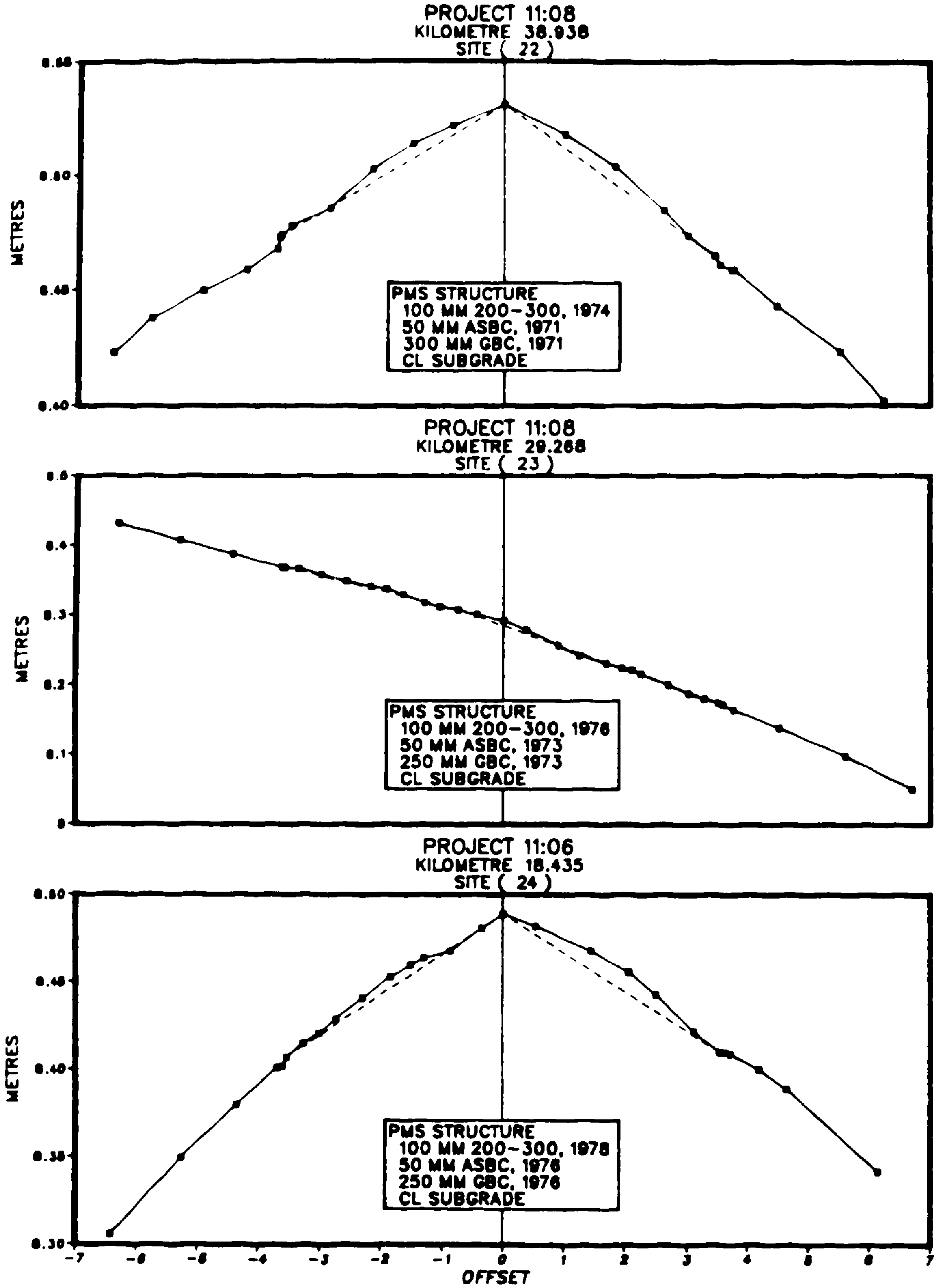

FIGURE B-8 SITE PROFILES (22 - 24)

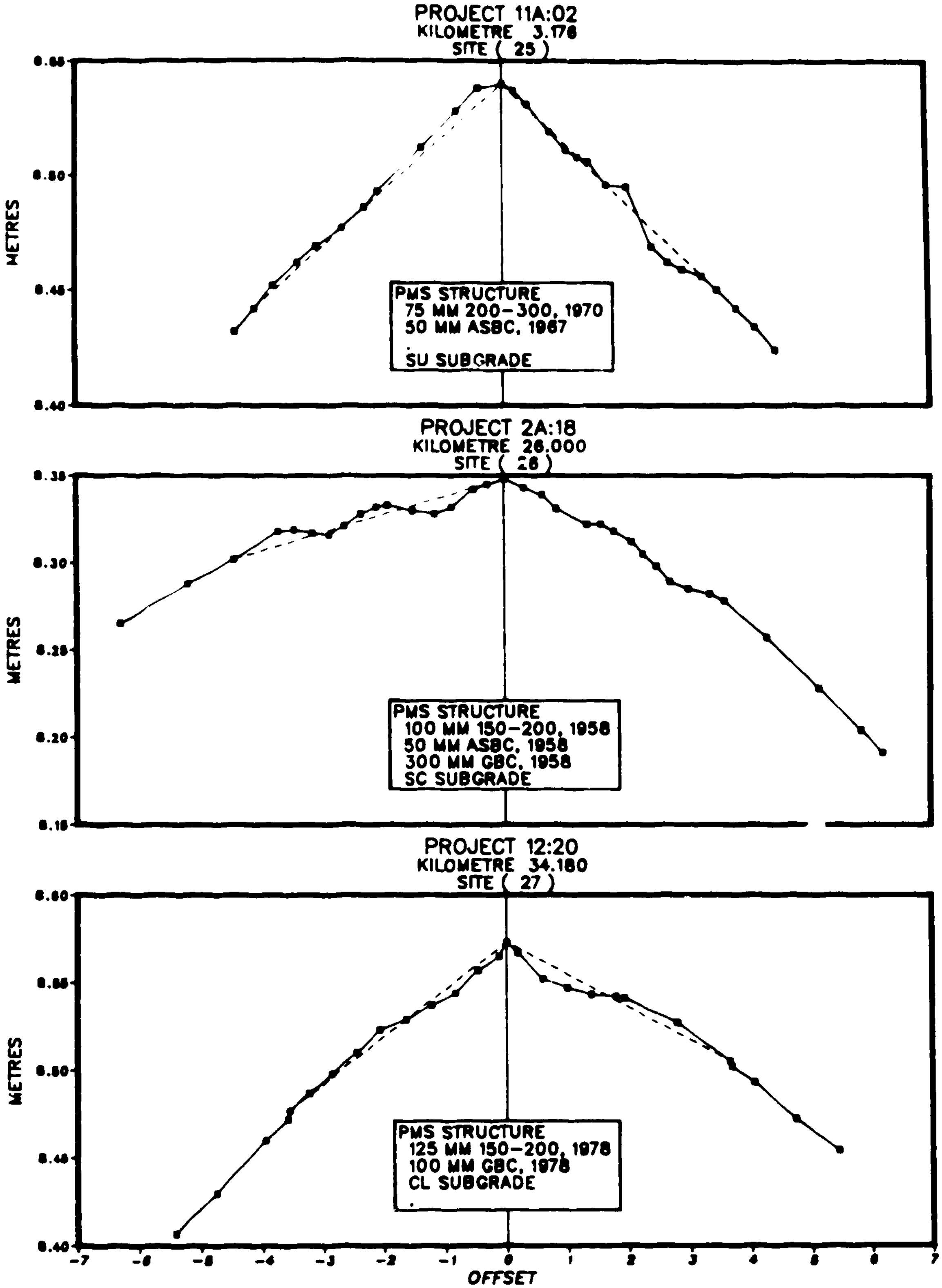

FIGURE B-9 SITE PROFILES (25 - 27)

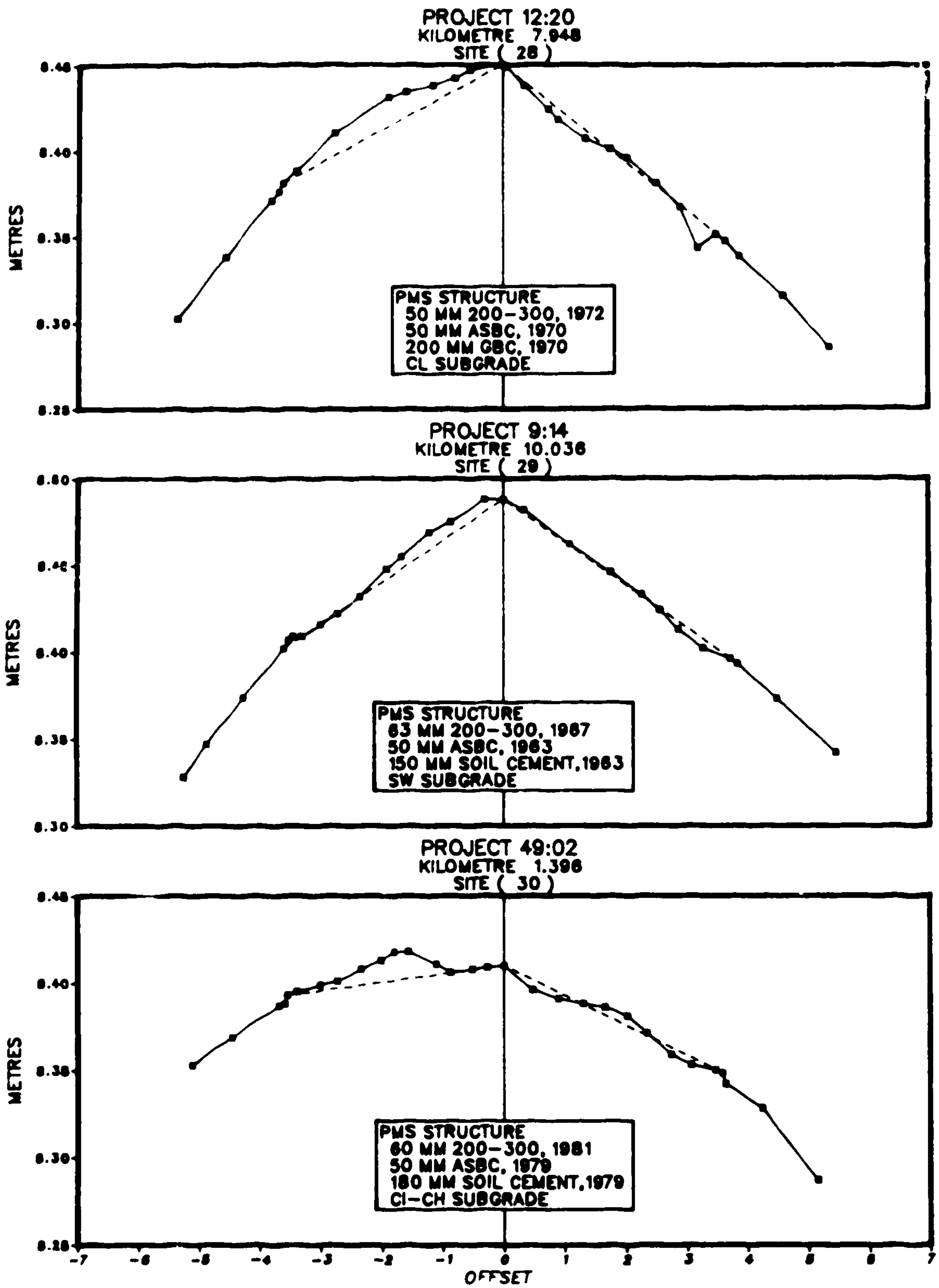

FIGURE B-10 SITE PROFILES (28 - 30)

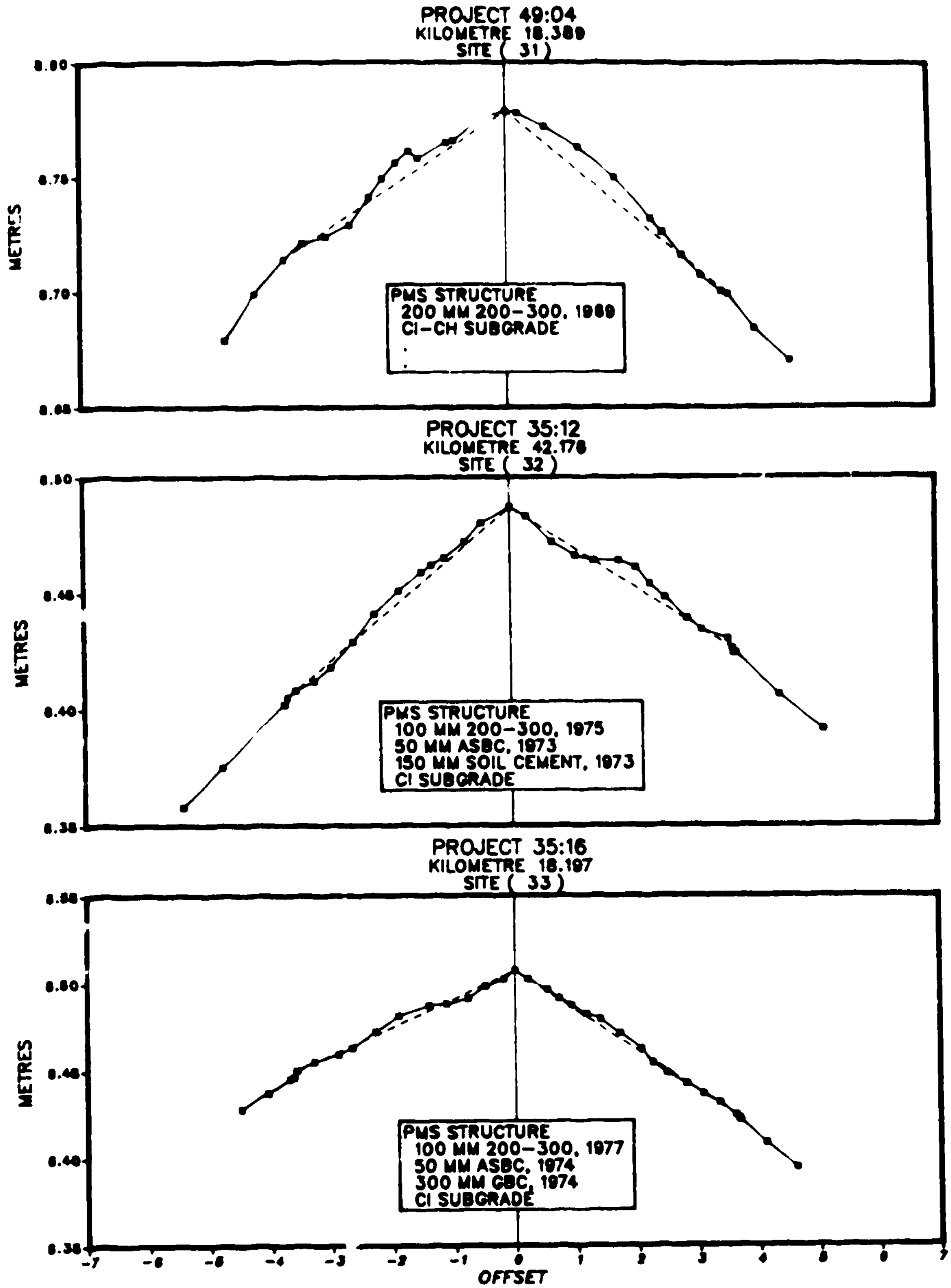

FIGURE B-11 SITE PROFILES (31 - 33)

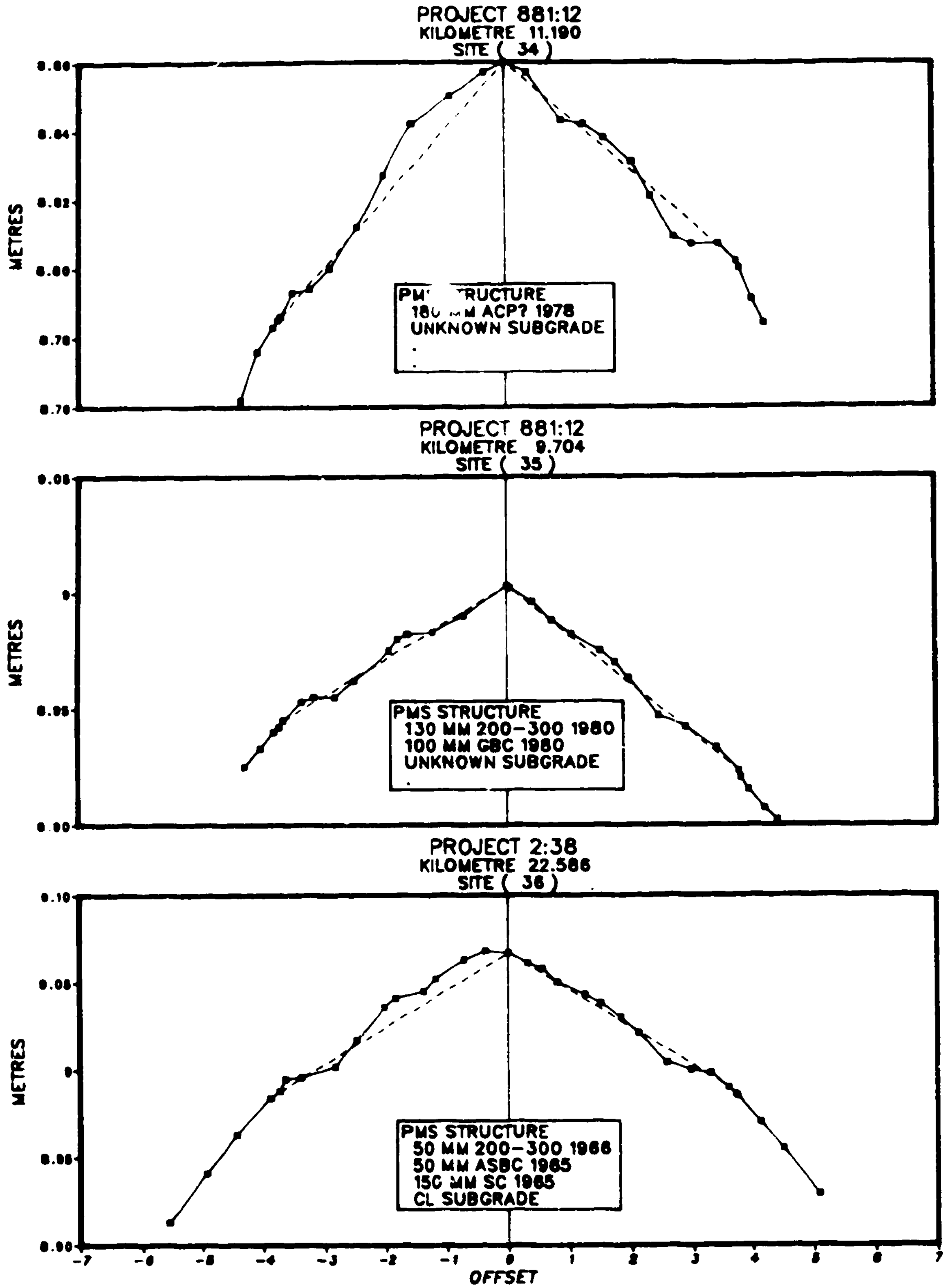

FIGURE B-12 SITE PROFILES (34 - 36)

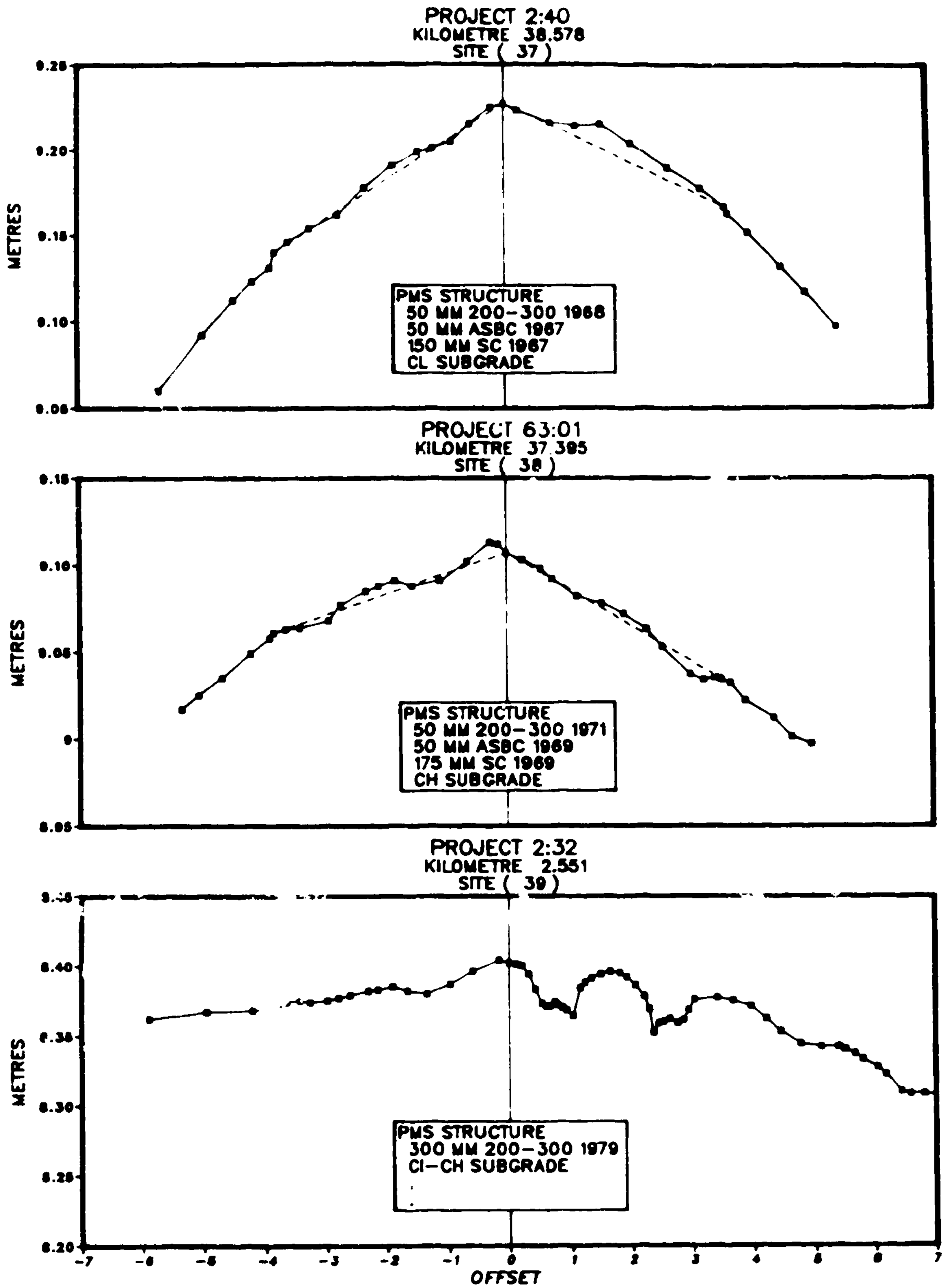

FIGURE B-13 SITE PROFILES (37 - 39)

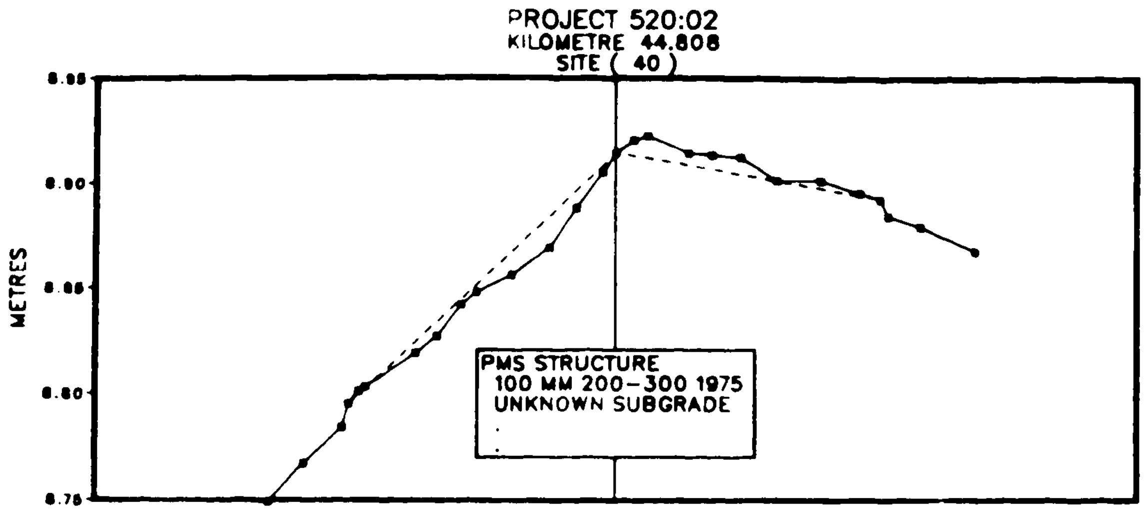

FIGURE B-14 SITE PROFILE (40)

APPENDIX C

TRIAXIAL TESTING PROCEDURE

APPENDIX C
TRIAXIAL TESTING PROCEDURE

Introduction

In order to determine the permanent deformation of the mix samples examined in this study, a repeated load triaxial test apparatus connected to a dynamic recording system was used. The testing equipment at the University of Alberta was initially set up for the type of testing conducted in this research by Hadipour[15]. A detailed explanation of the equipment and software used is given in Appendix D of that dissertation. This Appendix provides an overview of the testing procedure and data collection without attempting to detail the equipment and software specifications.

Test Equipment

A schematic representation of the testing equipment set up is shown in Figure C-1.

The applied load was measured by load cells (B) and the displacement of the sample under the applied load was measured by LVDT's (C). The confining pressure was measured by a pressure transducer (H). These three sets of measuring devices were calibrated prior to the beginning of the test program to assure accurate results could be obtained. Figure C-2 presents the calibrations for each of the load cells and LVDT's.

The load is applied via pneumatic cylinders to the loading bar which holds the load cell. The load cell acts directly on the loading ram of the triaxial cell. The loading frequency is controlled by an electric timing unit which operates electric solenoid valves which in turn control the air flow to the pneumatic cylinders. The timing unit is set up to provide a 0.5 second on and 0.5 second off time. The actual load felt by the specimen, as measured by the load cell, is shown in Figure C-3. This loading does not appear as a square wave as the supplied voltage signal does, but rather shows a buildup of the load over about the first 0.2 seconds followed by about 0.3 seconds of constant load before dropping off.

During testing a computer system logged the applied load and displacement at a minimum of 1 sec intervals. Readings were recorded for each sample for each of the first 1000 seconds, then at 100 second intervals until 10000 readings and then 1000 second intervals until the end of the test, up to a maximum of 100000 readings. Because the recording equipment is only capable of recording a maximum of one reading per second, the test start-up has to be staggered. The first test cell is started, after 1000 readings of the first cell the second cell is started, after 1000 readings had been made on the second cell the third cell is started.

The data is logged into the data collection program in binary form. In order to translate the binary data into meaningful deformation data a Lotus Macro program used which used the calibration factors determined for the LVDT's to calculate the change in thickness of each sample and thus determine the percent strain values.

Testing Procedure

The following describes the steps required for conducting the repeated load triaxial testing. Prior to testing, each sample must be inspected to assure smooth end surfaces to facilitate uniform loading.

1. Place the sample on the base platen of the loading cell and enclose the sample, base platen and load cap in a double rubber membrane. Secure the membranes with rubber o-rings at the base platen and loading cap.

2. Install the top portion of the triaxial cell and ensure that their is vertical alignment with the loading ram to prevent any eccentric loading.

3. Place the triaxial cell into the appropri... tes... y and fill the the cell with water. Connect the heat c... s... system to the connectors for the triaxial cell heating coil. (Time is r... for ... e sample temperature to reach the test temperature. For this ... as used for the 35° tests and 4h for the 45° tests).

4. Use a 'U' shap... , set the axial load. Remove the loading bar and place the load ... ctly on the triaxial loading ram.

5. Connect the LV... the mounting bracket and adjust it until it is within the voltage rang... ibrated for.

6. Apply the confining pressure, check that the correct voltage is measured by the pressure transducer. Bleed any air out of the triaxial cell.

7. Once all the cells have been set up and temperature equilibrium reached, initialize the computer program, providing the current test name, and start the test in bay 1. Start ba... 2 and 3 as 1000 readings have been taken in the adjacent cells.

The above procedure assumes that the test equipment is already set up at the proper loading frequency (0.5 sec on/off). The electri... timing unit (R) controls the loading frequency.

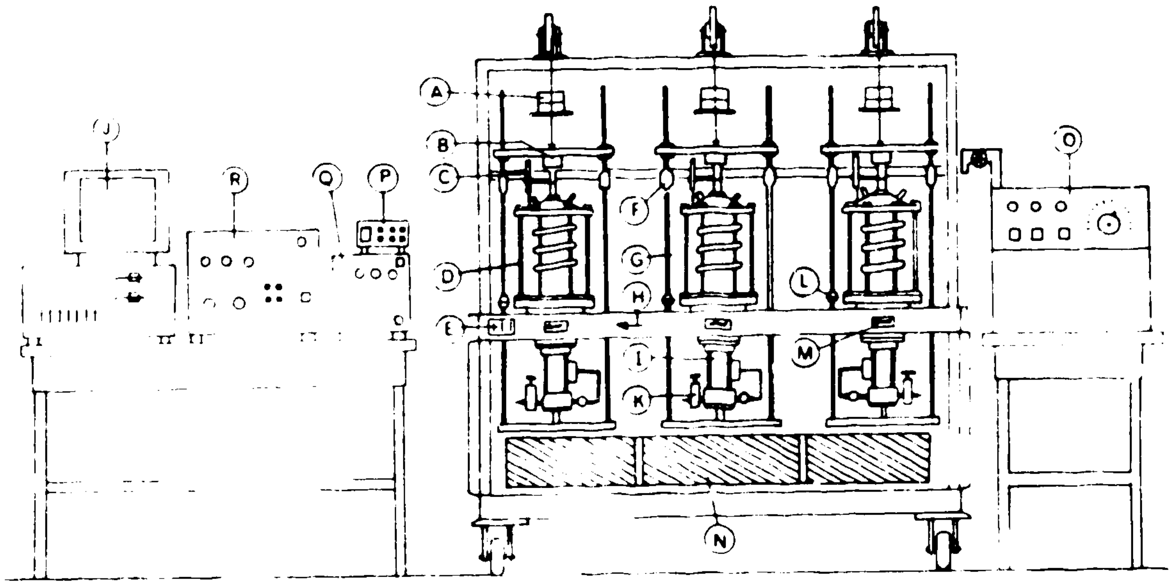

Figure C-1 Schematic of the Repeated Loading Triaxial Test Equipment

A. Counter balance weights
B. Load cell
C. LVDT
D. Triaxial cell
E. Loading switches
F. Guide frame
G. Loading yoke
H. Pressure transducer
I. Air pressure loading cylinder
J. Computer and data logger
K. Air pressure regulators

L. [illegible] shut-off switches
M. [illegible] counter
N. Air pressure tank
O. Temperature control bath
P. Voltmeter
Q. Electric timing unit
R. Signal conditioner

(Not shown)
Solenoid valves
Air pressure gauges

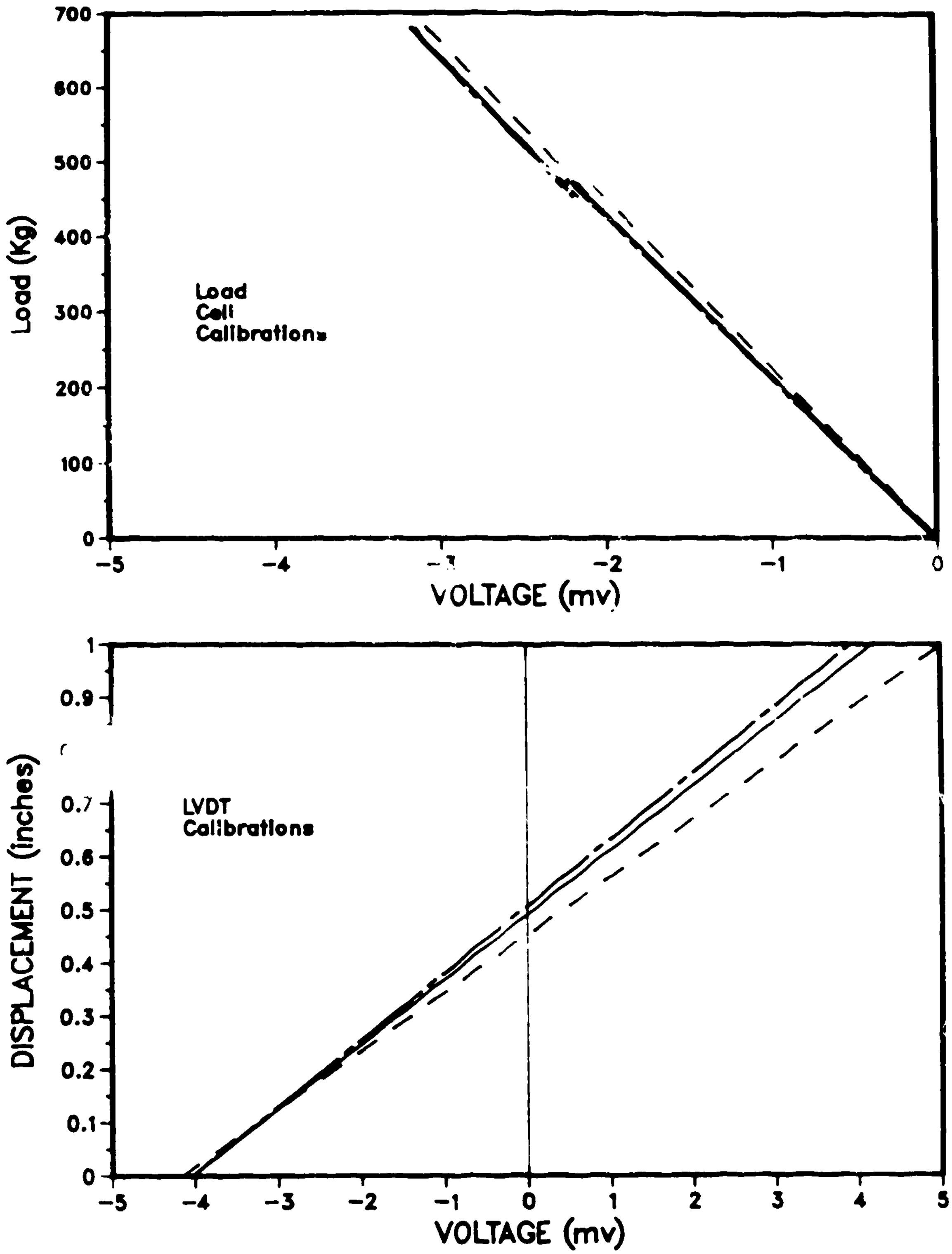

FIGURE C-2 CALIBRATION CURVES

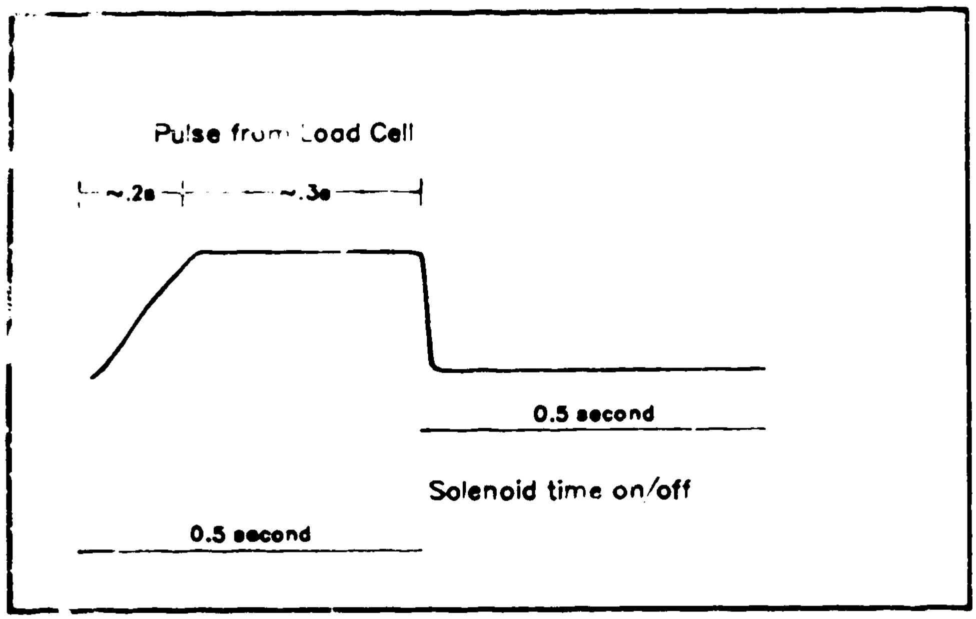

FIGURE C-3 LOAD PULSE TRACE

APPENDIX D
FIELD FORMED TEST SPECIMENS

APPENI \ D
FIELD FORMED TEST SPECIMENS

Introduction

Testing programs are generally conducted using laboratory prepared samples in order to assure consistency and eliminate variations associated with field production; as well as the logistics of obtaining field mix samples. In an effort to better relate the results of the testing program to field conditions, test specimens were formed in the field using plant produced mix and a hand operated kneading compactor. The objective was to compare the test results of the laboratory and field prepared samples to determine how representative the testing program is of the actual mix placed.

Field specimens were formed with four mixes from the two projects utilized in this research. The samples include recycled and virgin mixes and the design data for these mixes is given in Table D-1.

Sample Preparr .on

The sample preparation for the field formed specimens followed the procedure reported by Hadipour[15] which har been developed for the sample preparation in that study. Similar procedure .vere used for both the field and laboratory formed samples, except that diffe ent kneading compaction equipment was used and the pressure and number of blows ltered for the laboratory samples to result in more uniform densitie procedure requires 4080 g of mix which is compacted into split mol . mm in height by 102 mm diameter. The first 2720 g of mix was compacted using 150 blows at a compactive effort of 2.07 MPa; the final 1360 g was then given 110 blows at 3.45 MPa pressure. The specimen was then leveled with six blows of a marshall hammer. Following the formation of the field samples this procedure was found to result in too high of densities, however the samples were used as is for this work.

The field kneading equipment is shown in Figure D-1. The apparatus is equipped with a standard California kneading compactor foot complete with a heating element. The equipment was calibrated using a proving ring which yielded the calibration curve shown in Figure D-2. The moment applied to develop the compactive force was measured using a standard industrial type torque wrench.

Test Results

The strain values for the four field sample series are given in Table D-2. The relative magnitude of the strains between series is fairly constant, though at the 35° test temperature the 'FB' series strains more than the 'FD' series, which is opposite to the other two test temperatures. The test results are shown graphically in Figure D-3.

Analysis

The strair data for the field prepared samples were analyzed subsequent to that of the labcra y prepared samples reported in Chapter 6. The data for the field prepared samples were fit to a model form similar to the models presented in Chapter 6, but the temperatures was found to significantly improve the model and was therefore included as an additional term. The best fit model was determined as:

$$\begin{aligned}\text{Log}(\epsilon) = & -2.0591 + 0.1000 * \text{Log}(N) \\ & + 0.0431 * \text{Log}(N) * \text{Log}(\text{Abson Stiffness}) \\ & + 0.0248 * \text{Log}(N) * \text{Log}(\text{Abson Stiffness}) * \text{Log}(\% \text{ RAP}) \\ & + 0.0402 * \text{Temperature}\end{aligned}$$

where ϵ = permanent strain (%)
N = number of load applications
Abson Stiffness = calculated stiffness of Abson recovered binder
% RAP = percentage of reclaimed asphalt pavement in mix
Temperature = test temperature (°C)

This model has an r^2 of 0.7906 and does not show any lack-of-fit (probability ≤ 0.05). The ANOVA for this model is presented in Table D-3.

For the model developed here the quantity of RAP in the mix was determined to be a contributing variable, but opposite in sign compared to the relationships developed for the Laboratory prepared samples. For the field samples it can be seen that an increase in RAP content corresponds to increases in the predicted strains. Also it is interesting to note that both interaction terms have positive signs. This seems to suggest increased strain with increased binder stiffness, however the temperature term also reflects the the stiffness of the binder and results in a net reduction in the strain value predicted for various binder stiffnesses at the given temperatures. As well the binders are all of similar resultant asphalt grades which may have influenced the model.

Summary

Because the field formed samples were formed with such low air voids, direct comparisons of behaviour to the Laboratory prepared samples was not done. However two significant observations were made. Firstly, the sign of the % RAP correlation to the measured strain shows that the % RAP is not necessarily a causation variable and in fact may only reflect the resultant rheology of the binder. As well, the form of the best fit model, developed for the field samples, is very similar to the model form developed in Chapter VI. This shows that Laboratory prepared samples do offer an insight as to the behaviour of plant mixed asphalt concrete pavement, at least under laboratory conditions.

TABLE D-1 FIELD SPECIMEN CHARACTERISTICS
(Part 1)

SAMPLE	TYPE	MST	DENSITY	L (mm)	AIR VOIDS	AGGREGATE SOURCE	RAP SOURCE	DESIGN RHEOLOGY PEN	ABS VISC	KIN VISC	ABSON RHEOLOGY PEN	ABS VISC	KIN VISC
FA-1	R/V=25/75	2255BR1	2443.8	197.9	1.1	Goose Lake	SP 'C'	160	85	275			
FA-2	R/V=25/75	2255BR1	2445.7	198.7	1.0	Goose Lake	SP 'C'						
FA-3	R/V=25/75	2255BR1	2439.6	198.2	1.2	Goose Lake	SP 'C'						
FA-4	R/V=25/75	2255BR1	2439.6	198.8	1.2	Goose Lake	SP 'C'						
FA-5	R/V=25/75	2255BR1	2444.1	196.4	1.1	Goose Lake	SP 'C'				123	135.1	345.
FA-6	R/V=25/75	2255BR1	2442.0	200.6	1.1	Goose Lake	SP 'C'						
FA-7	R/V=25/75	2255BR1	2441.9	198.9	1.1	Goose Lake	SP 'C'						
FA-8	R/V=25/75	2255BR1	2437.3	198.8	1.3	Goose Lake	SP 'C'				123	142.8	350.
FA-9	R/V=25/75	2255BR1	2446.8	199.2	1.0	Goose Lake	SP 'C'						
FB-1	VIRGIN	2231B	2418.2	199.7	2.1	Goose Lake	-	187	73	250			
FB-2	VIRGIN	2231B	2437.4	199.5	1.4	Goose Lake	-				85	310.3	488.0
FB-3	VIRGIN	2231B	2422.5	201.3	2.0	Goose Lake	-						
FB-4	VIRGIN	2231B	2440.3	198.9	1.3	Goose Lake	-						
FB-5	VIRGIN	2231B	2416.6	200.6	2.2	Goose Lake	-				107	199.2	405.0
FB-6	VIRGIN	2231B	2440.2	199.8	1.3	Goose Lake	-				115	175.5	383.0
FB-7	VIRGIN	2231B	2417.8	199.7	2.1	Goose Lake	-						
FB-8	VIRGIN	2231B	2418.8	200.5	2.1	Goose Lake	-				85	276.6	457.0
FB-9	VIRGIN	2231B	2418.2	199.7	2.1	Goose Lake	-						

TABLE D-1 FIELD SPECIMEN CHARACTERISTICS
(Part 2)

SAMPLE	TYPE	MST	DENSITY	L (mm)	AIR VOIDS	AGGREGATE SOURCE	RAP SOURCE	DESIGN RHEOLOGY PEN	DESIGN RHEOLOGY ABS VISC	DESIGN RHEOLOGY KIN VISC	ABSON RHEOLOGY PEN	ABSON RHEOLOGY ABS VISC	ABSON RHEOLOGY KIN VISC
FD-1	R/V=25/75	2223B	2449.7	197.6	1.2	Goose Lake	SP 'A'	210	46	225			
FC-2	R/V=25/75	2223B	2437.2	198.8	1.7	Goose Lake	SP 'A'				88	227.3	420.
FC-3	R/V=25/75	2223B	2436.5	199.7	1.7	Goose Lake	SP 'A'						
FC-4	R/V=25/75	2223B	2442.5	202.8	1.4	Goose Lake	SP 'A'				95	277.9	427.
FC-5	R/V=25/75	2223B	2441.9	200.8	1.5	Goose Lake	SP 'A'						
FC-6	R/V=25/75	2223B	2431.9	201.4	1.9	Goose Lake	SP 'A'				90	225.0	425.
FC-7	R/V=25/75	2223B	2450.1	196.9	1.1	Goose Lake	SP 'A'						
FC-8	R/V=25/75	2223B	2448.7	199.7	1.2	Goose Lake	SP 'A'				95	220.4	421.
FC-9	R/V=25/75	2223B	2436.5	200.4	1.7	Goose Lake	SP 'A'						
FD-1	R/V=15/85	2274BR1	2393.0	201.9	1.1	Blackfalds	HW 2:26	170	80	245	118	165.7	376.0
FD-2	R/V=15/85	2274BR1	2394.3	202.0	1.0	Blackfalds	HW 2:26						
FD-3	R/V=15/85	2274BR1	2393.7	199.7	1.0	Blackfalds	HW 2:26						
FD-4	R/V=15/85	2274BR1	2368.0	201.3	2.1	Blackfalds	HW 2:26				90	255.7	446.
FD-5	R/V=15/85	2274BR1	2366.1	202.7	2.2	Blackfalds	HW 2:26						
FD-6	R/V=15/85	2274BR1	2371.4	200.5	2.0	Blackfalds	HW 2:26				84	345.4	520.
FD-7	R/V=15/85	2274BR1	2369.5	201.1	2.0	Blackfalds	HW 2:26				75	406.8	532.
FD-8	R/V=15/85	2274BR1	2374.9	200.9	1.8	Blackfalds	HW 2:26						
FD-9	R/V=15/85	2274BR1	2372.9	201.3	1.9	Blackfalds	HW 2:26						

TABLE D-2 PERCENT PERMANENT STRAIN - FIELD SERIES

Temperature	N	FA	FB	FC	FD
25	10	0.181 (0.12)	0.130 (0.07)	0.152 (0.08)	0.213 (0.10)
	50	0.407 (0.17)	0.258 (0.12)	0.353 (0.11)	0.412 (0.15)
	100	0.526 (0.19)	0.324 (0.15)	0.460 (0.13)	0.503 (0.17)
	1000	1.077 (0.27)	0.580 (0.26)	0.919 (0.26)	0.891 (0.31)
	5000	1.621 (0.36)	0.788 (0.40)	1.349 (0.41)	1.201 (0.42)
	10000	1.889 (0.42)	0.887 (0.46)	1.563 (0.49)	1.345 (0.46)
	20000	2.181 (0.47)	0.990 (0.52)	1.786 (0.60)	1.493 (0.50)
	30000	2.353 (0.52)	1.054 (0.55)	1.926 (0.68)	1.582 (0.54)
	40000	2.489 (0.54)	1.100 (0.58)	2.032 (0.76)	1.645 (0.56)
	50000	2.598 (0.55)	1.133 (0.60)	2.115 (0.82)	1.691 (0.58)
	60000	2.687 (0.57)	1.162 (0.62)	2.186 (0.87)	1.735 (0.59)
	70000	2.763 (0.58)	1.187 (0.63)	2.248 (0.92)	1.772 (0.60)
35	10	0.314 (0.05)	0.223 (0.07)	0.184 (0.10)	0.174 (0.05)
	50	0.634 (0.09)	0.393 (0.09)	0.288 (0.34)	0.336 (0.06)
	100	0.805 (0.11)	0.474 (0.10)	0.399 (0.40)	0.376 (0.05)
	1000	1.706 (0.19)	0.824 (0.21)	1.055 (0.42)	0.672 (0.04)
	5000	2.834 (0.16)	1.207 (0.37)	1.644 (0.46)	0.978 (0.01)
	10000	3.484 (0.10)	1.420 (0.49)	1.970 (0.51)	1.137 (0.05)
	20000	4.235 (0.10)	1.631 (0.56)	2.328 (0.57)	1.349 (0.09)
	30000	4.729 (0.16)	1.809 (0.67)	2.546 (0.60)	1.509 (0.10)
	40000	5.121 (0.26)	1.966 (0.78)	2.699 (0.64)	1.633 (0.11)
	50000	5.457 (0.38)	2.117 (0.90)	2.831 (0.67)	1.748 (0.12)
	60000	5.786 (0.53)	2.270 (1.03)	2.943 (0.70)	1.869 (0.13)
	70000	6.109 (0.70)	2.434 (1.17)	3.047 (0.74)	1.984 (0.16)
45	10	0.586 (0.23)	0.308 (0.08)	0.420 (0.03)	0.336 (0.10)
	50	1.064 (0.27)	0.592 (0.21)	0.852 (0.06)	0.643 (0.23)
	100	1.330 (0.30)	0.740 (0.28)	1.100 (0.09)	0.805 (0.30)
	1000	2.630 (0.35)	1.371 (0.57)	2.289 (0.40)	1.570 (0.57)
	5000	4.243 (0.41)	1.991 (0.81)	3.691 (0.97)	2.478 (0.81)
	10000	5.147 (0.49)	2.334 (0.95)	4.480 (1.26)	3.079 (0.97)
	20000	6.220 (0.50)	2.764 (1.12)	5.439 (1.58)	3.941 (1.21)
	30000	7.015 (0.48)	3.099 (1.23)	6.161 (1.74)	4.725 (1.37)
	40000	7.715 (0.42)	3.402 (1.34)	6.835 (1.86)	5.500 (1.42)
	50000	8.277 (0.23)	3.705 (1.45)	7.446 (2.00)	6.231 (1.38)
	60000	8.750 (0.52)	4.009 (1.56)	8.697 (2.55)	--
	70000	9.674 (1.05)	- -	11.294 (-)	- -

Bracketed values are standard deviation for the average strain values

TABLE D-3 STATISTICS FOR FIELD SAMPLE MODEL

INTERCEPT	-2.05915
MULTIPLE CORRELATION	.88920
STD. ERROR OF ESTIMATE	.13628
R-SQUARED	.79067

VARIABLE NO.	MEAN	STD. DEV.	CORRELATION X VS Y	REGRESSION COEFF.	STD. ERROR OF REG.COEF.	COMPUTED T VALUE
1	4.100	.4862	.3711	.10004067	.0205	4.8781
2	8.619	2.7069	-.3332	.04309155	.0065	6.6640
3	8.295	5.8606	.2495	.02487444	.0013	19.0025
4	35.080	8.4489	.6439	.04020077	.0019	20.8317
DEPENDENT						
5	.339	.2963				

ANALYSIS OF VARIANCE FOR THE REGRESSION

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARES
ATTRIBUTABLE TO REGRESSION	4	25.9540	6.4885
DEVIATION FROM REGRESSION	370	6.8713	.0186
TOTAL	374	32.8253	
Lack of Fit	140	2.4661	0.0176
Pure Error	231	4.4051	0.0190
			F value = 0.92

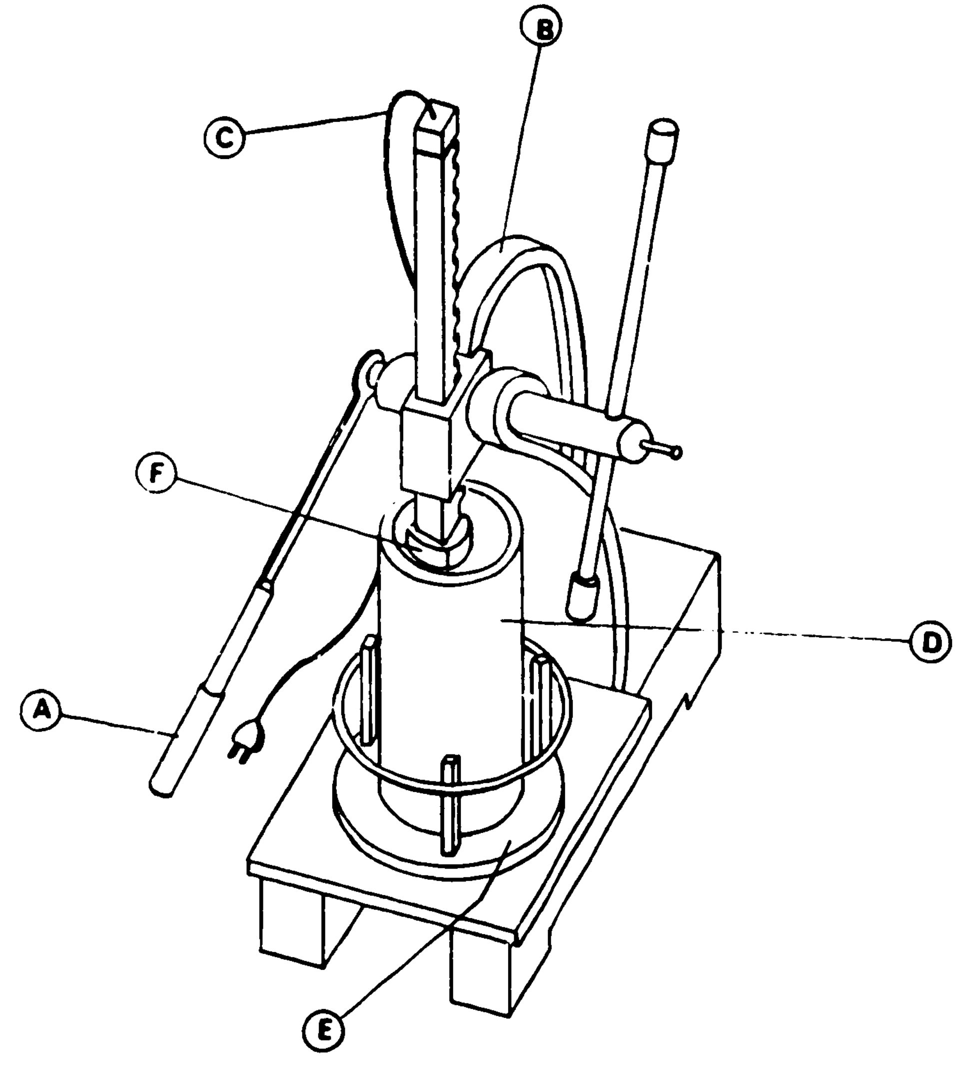

FIGURE D-1 FIELD KNEADING COMPACTOR

A. Torque Wrench
B. Press Frame
C. Heater Cord
D. Split Mold
E. Swivel Base
F. Heated Compaction Foot

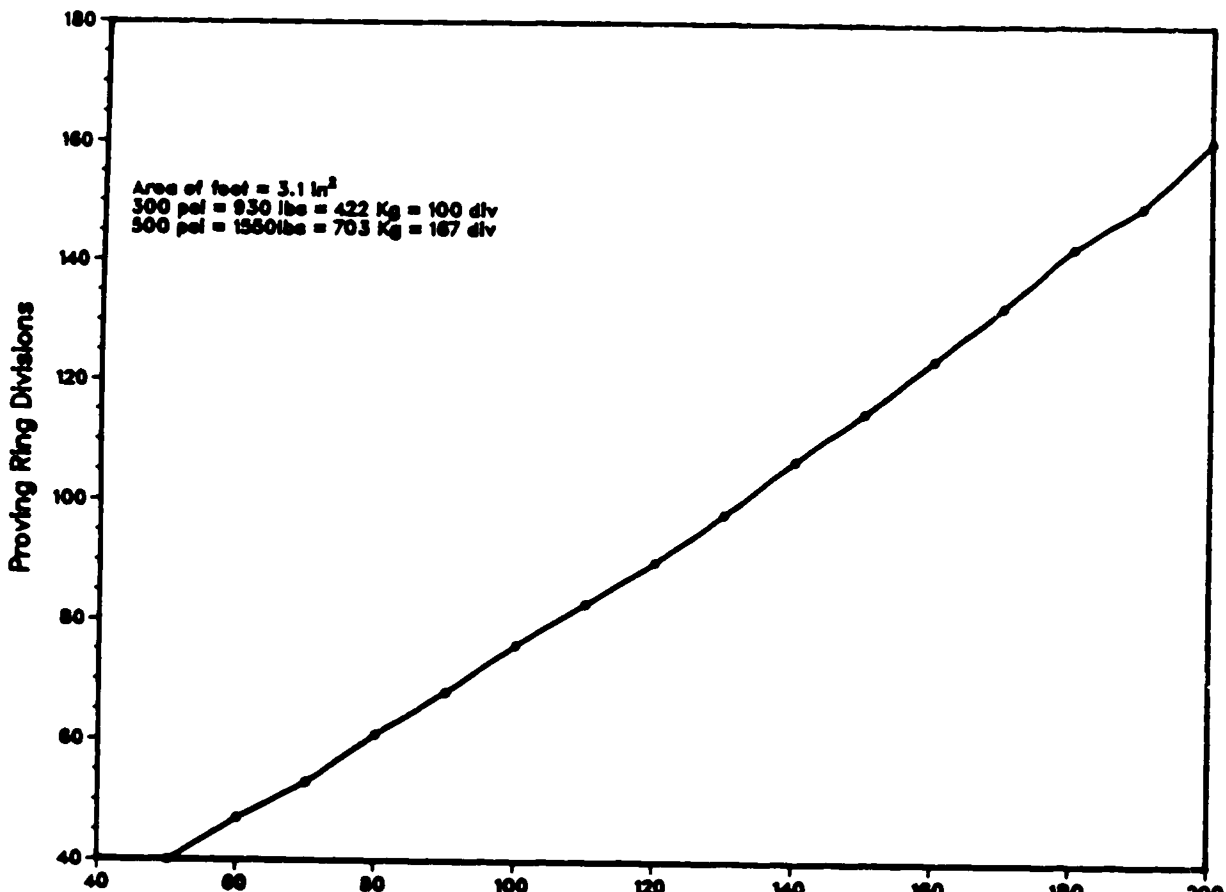

FIGURE D-2 CALIBRATION CURVE FOR FIELD KNEADING COMPACTOR

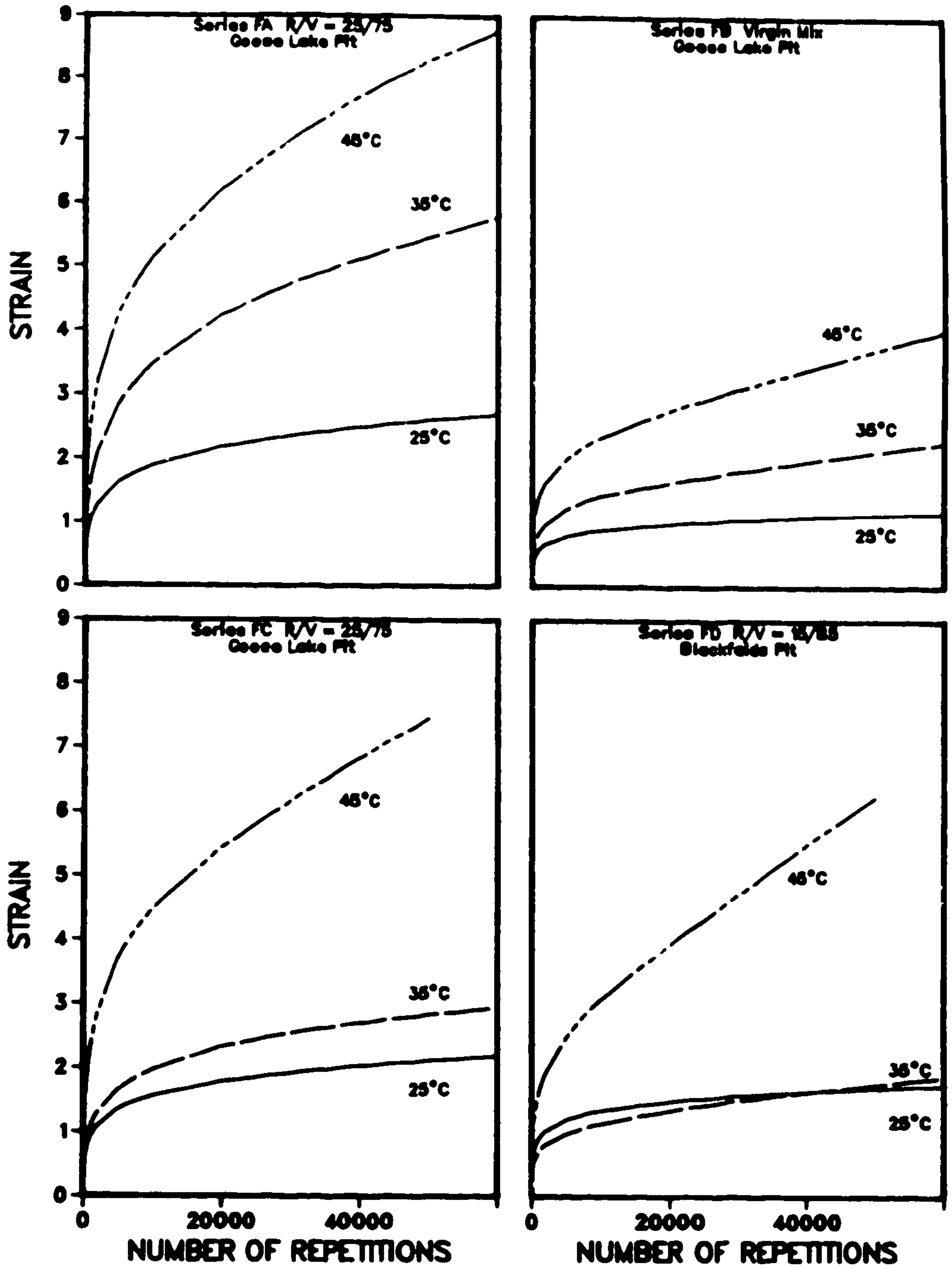

FIGURE D-3 PERMANENT DEFORMATION CURVES FOR FIELD MIXES

APPENDIX E
DATA HANDLING

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DATA HANDLING

Introduction

The amount of data collected for both the repeated load and field stages of this project were voluminous and thus lent itself to computer applications. The Database management program chosen to handle the large amount of information collected was FOCUS, a software product marketed by Information Builders of New York. The FOCUS system utilized (release 5.5) operated under TSO/VMS on the Government of Alberta's IBM 370 type mainframe computer.

The mainframe FOCUS system was used to maintain the databases utilized for this project and to perform manipulations on large amounts of data, as well as acting as an interface medium for generating plots using data from within the databases, and finally to conduct the statistical analysis.

Database Design

Two separate databases were used for this investigation, one for each of the two stages. Figures IV-7 and VI-1 show the rational design of the file hierarchy and the data fields for each segment. For the field study, data was inputted into the database using specifically designed CRT screens which facilitated both the entry and editing of data. For the Laboratory testing portion of the study the data was collected by the data logging program, and reduced using a LOTUS 123 macro. (Appendix B describes the data collection for the Laboratory testing). The reduced data was then downloaded into mainframe TSO files and subsequently read into the FOCUS database using a FOCUS fixform modify program. The materials data for the Laboratory portion of the study was loaded into the database using entry screens as for the field study.

Data Manipulation

Once the database has been created, FOCUS provides powerful syntax for further manipulations. FOCUS is a fourth generation language (4GL) and as such accepts simple english type commands to select and sort data. FOCUS command files, or focexec's, allow numerous subsets of selected data to be created which can subsequently be used for either creating report quality graphics, as presented within this report, or for statistical analysis of specific groups of data. The analysis of the Laboratory test data used numerous data subsets created this way to look at the different sample series individually.

Statistical Tools

The statistical analysis facilities available under FOCUS offer a number of procedures for calculating various statistics. The procedures utilized for this study were Corre (correlation matrix), Stepr (stepwise multiple regression), and Multr (multiple regression).

The Corre routine calculates the individual correlation coefficients between all of the variables selected. As well, the calculated means and standard deviations are given in the output. This routine proved useful as an initial step in identifying the significant parameters; and also allowed the interactions between variables to be examined. The results from each corre analysis directly influenced the development of various model forms which were then analyzed further.

The FOCUS Stepr routine uses a forward selection procedure[26] to select the independant variable entered into the model at each step of the regression analysis. The program allows for the choice of any of the database fields for dependant and independant variables. Each step then attempts to reduce the sum of squares the greatest amount, thus selecting the variable with the next highest partial correlation to the dependant variable. As well the user can force specific variables to be included as part of the model; th[illegible] for this study to examine specific models which were not selected by the [illegible]edure. The output from the stepr routine includes the selected model, the [illegible]ssion coeffici[illegible] the F value and t values for the individual coefficiants. The F value is [illegible] as the ratio of the average sum of squares attributable to the regression, to the mean square deviations from the regression and indicates the overall significance of the regression equation. The t value is the ratio of the regression coefficient to the standard error of the regression, and indicates the significance of individual coeficients.

The Multr program performs regression analysis for linear models. The Stepr program was used to determine the model final form and then Multr was used to produce the final statistics as presented in Chapters IV and VI.