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A SYNTHESIS OF CEMENT-TREATED BASE PRACTICE FOR HIGHWAYS IN ALBERTA

by

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ABSTRACT

In the last twenty-five years, cement-treated base has been extensively used in Alberta as replacement for conventional granular base course material in flexible highway pavements. With this increased use, a bulk of information has come to exist in the province on the material. This study was primarily directed at compiling the available information on cement-treated base practice for highway pavements in the province, in anticipation of a Roads and Transportation Association of Canada attempt to synthesize Canadian practice and performance, and also provide a needed reference for local practice.

A preceding in-depth literature review on the general cement-treated base practice worldwide, covered the broad aspects of its properties, characteristics, different methods of production and use. The review indicated how this material, composed mainly of substandard aggregates or soils, has proven to be a very stable pavement material in countries with diverse soils, climates and traffic conditions, and has been used extensively in road construction.

Against this background, the specifics of the mix design, structural design and construction procedures in Alberta, as practiced by Alberta Transportation, were presented. In addition, performance evaluations of some typical cement-treated base pavements in the province were undertaken to determine whether the design and construction

procedures used have given satisfactory pavements. From the limited data available it was apparent that, provided the correct practices and specifications are followed, cement-treated base performs adequately in all the types of flexible pavements common in the province. A theoretical economic analysis of some typical cement-treated base pavements and their equivalent conventional granular base course pavements also revealed that, in some instances, with specific reference to the general disposition of aggregate resources in the province, cement-treated base may indeed be the more economic alternative of the two.

On the basis of this study, it was concluded that cement-treated base is a viable base course material for flexible pavements comparable to the other base course materials frequently used in Alberta for highway construction. Not only does it provide an alternative to the so-called conventional granular base course materials, but also has in its own right the engineering properties, characteristics, performance capabilities and economic attributes which make it a bona fide pavement construction material. Consequently, its use should be encouraged and more scientific research work conducted to unearth the possibly numerous potentials of the material, yet to be discovered and utilized.

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Table of Contents

List of Tables

List of Figures

Figure | Page

List of Plates

1. INTRODUCTION

1.1 Background

Good quality material availability is one very important aspect of highway pavement design and construction. Not only are quality aggregate sources required but also the distance aggregates have to be transported is directly related to the final cost of a pavement. Consequently, with the continuing depletion of good quality aggregate sources, an increasing awareness of the need to conserve part of the available resources for future use, and the clear economic advantage of using locally available materials, it has become increasingly necessary to upgrade relatively inferior quality materials to make them suitable for use in road construction.

Generally, a number of stabilization methods are employed to enhance the engineering properties of these marginal quality aggregates, to standards that compare favorably with those of conventional high quality aggregate materials. A particular stabilized material which offers one of the best forms of pavement construction material is cement treated aggregate or sand. This material, described as cement-treated base and also commonly known as soil-cement, has been successfully used as a base course material for flexible and rigid pavements of highways and airfields; surfaced parking lots; surfaced shoulders of highways and airfields; and has been applied in many more other areas.

In the province of Alberta, Canada, cement-treated base has been used as a structural component of pavements to quite an extent. Lack of readily available good quality aggregates in some parts of the province has required the use of marginal quality aggregates in road construction. According to Dacyszyn (1961), this culminated in the construction of a short experimental pavement in 1953 on Highway 18, approximately 80 kilometers north of the provincial capital, Edmonton, to investigate the qualities of cement treated materials as an alternative for base course construction. As a result of material problems, however, this project was not entirely successful, and there was a lull in construction with cement treated materials in the province until 1959. In that year, as Dacyszyn (1961) reports, the construction of a total of 76.91 km (47.74 miles) of cement-treated base pavements marked the beginning of serious construction of that kind of pavement in Alberta.

By 1972, as Shields *et al.* (1975) indicate, approximately 1 850 equivalent two-lane kilometers of cement-treated base or soil-cement pavements, representing about 21% of the paved main highway system had been built in Alberta. This increased to about 3 000 equivalent two-lane kilometers by the end of 1985, representing about 22 % of the much increased paved main highway system. As a result, there has been considerable experience and expertise associated with cement-treated base in the 25 years of its use, and quite a wealth of information is available in the province on the construction and performance of highways with this material as base course in the pavement structure.

1.2 Purpose and Scope of Thesis

In light of the above, the usefulness of a synthesis of cement-treated base (CTB) practice for highways in Alberta has been realized. The general objective of this study is to satisfy this need. Information and documentation on the major aspects associated with the provision of CTB pavements in the province have been compiled to give an exhaustive account of the CTB practice in Alberta. Specifically, the main objectives of the study are to:

(a) review the literature on the development of CTB as a base course construction material;
(b) document the mixture design method for CTB in Alberta;
(c) present the structural design method for CTB pavements in the province;
(d) document CTB construction practices in Alberta;
(e) evaluate the performance of CTB pavements built in Alberta; and
(f) discuss the economic factors involved in the use of CTB, especially in comparison to the conventional granular base course material it often replaces.

As already indicated, the emphasis of the study is on the use of CTB as base course material for highway pavements. Its use as a construction material for parking lots, residential streets, airfield pavements, and others are not of major concern here.

1.3 General Organization of Thesis

CHAPTER 2 is a literature review on the general development of CTB as a pavement construction material, and is intended as a perspective introduction to the subsequent chapters on CTB practice in Alberta.

CHAPTER 3 looks at the CTB mixture design practice in Alberta. The method is a variation of the standard ASTM and AASHTO methods involving freeze-thaw, wet-dry and unconfined compressive strength tests, and requires an appreciable length of time. Consequently,short-cut methods suggested by some workers to minimize the time are also presented.

In CHAPTER 4, an account is given of the structural design practice in the province. Comparisons are made between the thicknesses arrived at using this design method and those obtained by other practices.

CHAPTER 5 documents the construction practices in Alberta. Of considerable interest are the quality control measures which accompany construction.

An attempt is made in CHAPTER 6 to evaluate the performance of CTB pavements in the over a quarter of a century of their serious use in the province. Basically, data collected by Alberta Transportation as part of an extensive inventory data base on the paved primary highway system in the province, and computerized by the Alberta Research Council (ARC), forms the basis of this evaluation.

CHAPTER 7 is a discussion on the economic factors involved in the usage of CTB in pavements. Comparisons are made on the basis of life-cycle costs between pavements with CTB as base course material and those with conventional granular base course material.

CHAPTER 8 ties the preceding chapters together and summarizes the general conclusions arrived at from the study. Recommendations are made for future research work which will contribute to the maximum economic utilization of CTB for pavement construction in Alberta.

2. CEMENT-TREATED BASE AS A PAVEMENT CONSTRUCTION MATERIAL

2.1 Introduction

Cement-treated base, possessing qualities of its own quite different from those of its constituent materials, is a widely used pavement construction material. Together with the other types of cement stabilized materials, they are the most frequently used form of stabilized road construction material next to those obtained by the mechanical stabilization of soils.

Primarily, cement-treated base has been used as a base course material for both flexible and rigid pavements, of highways and airfields, as replacement for unavailable conventional granular base course material. In the over 50 years of its serious engineering use, an intial skepticism that existed in the early years has been replaced by a recognition of the potential of cement-treated base as a useful pavement material. Capable of attaining high standards of the various engineering properties and characteristics, cement-treated base has provided commendable levels of service in many instances, with an added advantage of economy of construction.

Mixture design methods have mainly been by durability and/or strength based laboratory tests, and although structural designs have mostly been restricted to the selection of a thickness based on past experience, a majority of the cement-treated base pavements built have performed quite well under varying traffic and environmental conditions. Construction is either by a mixed-in-place method or involves the importation of plant-mixed cement-treated base to the roadway site for construction. Satisfactory results are obtained with both methods where the recommended procedures are closely followed.

Cracking of cement-treated base, especially when accompanied by reflective cracking of an usual protective bituminous surface cover, appears to be the major problem encountered with the pavements. However, various measures now exist for curbing or eliminating such cracking. Also, depending on prevailing local conditions, cracking does not necessarily lead to complete structural failure and is not always considered to be highly detrimental to the performance of cement-treated base pavements.

This chapter reviews the research and experience which forms the basis of current cement-treated base practice, with emphasis on its use as a base course material for flexible pavements.

2.2 Historical Background

Conflicting reports exist as to where and when cement-treated base, CTB, or soil-cement as it is sometimes called, was first used as a pavement construction material. There is general agreement, however, that a 2.4-km (1.5-mi) CTB pavement built in 1935, near Johnsville, South Carolina, in the United States was the first scientifically constructed CTB pavement. According to Catton (1959), the success of this test section and others prompted research by the Portland Cement Association (PCA) on CTB. Findings were favorable, and soon the material had gained wide use as a pavement construction material.

In countries such as the United States, Canada and Australia, long lengths of light-trafficked roads through sparsely populated areas often devoid of good quality base course material, makes CTB construction using substandard materials particularly attractive. Davidson (1961) reports that by 1940 over 6 million m² of CTB had been constructed in the United States, and Robinson (1945) makes mention of the successful and economic use of CTB in Canada prior to 1945. Davidson (1961) further points out that by 1960, almost 245 million m² of CTB had been constructed in the United States and Canada. According to Maze (1964), the use of CTB in Australia started with the construction of the first CTB pavement in a suburb of Sydney as early as 1935.

The material has also been used extensively in Europe, Asia and Africa. Clare and Foulkes (1954) were able to substantiate claims that more than 100 million m² of CTB pavements were built in and around Germany between 1938 and 1945; and MacLean and Lewis (1963) reveal that laboratory research and field trials by the Road Research Laboratory (RRL) of Britain started as far back as 1939. Dos Santos (1961), Bhatia (1967) and Newill (1968) also report of CTB construction in Mozambique, Ghana and Malaysia respectively;

and from laboratory tests Ola (1975) determined that the lateritic soils of a number of African and Asian countries were suitable for use as CTB.

Thus, since 1935, CTB has been used successfully in many countries with diverse soils, climates and traffic load characteristics. Indications are that it will be increasingly used, as supplies of conventional base materials become depleted.

2.3 Terminology

A variety of materials are obtained when soils are stabilized with cement. With the differences in the parent soils, mix and structural design procedures, construction practices, and even the use to which a particular stabilized material may be put, a wide range of names have been given to these stabilized materials. In an effort to clearly distinguish between CTB and the other materials stabilized with cement, definitions have been prepared and are presented in this section.

Definitions proposed by the Highway Research Board, HRB, (1961) in a bulletin on soil stabilization with portland cement; the Portland Cement Association, PCA, (1971,1979) in their handbooks on the design and construction of soil-cement; and Johnson (1960), form the basis of the definitions presented in the following paragraphs.

Cement Stabilized Soil or *Cement Treated Soil* is soil to which cement and water has been added to improve its natural qualities and make it stable. No specific quality requirements are expected to be met by the stabilized material.

Cement-Treated Base, CTB, is an intimate mixture of pulverized soil, cement and water, compacted at the optimum moisture content to a high density, to give a material expected to meet specific requirements of volume stability, durability, strength and adequate impermeability. Another name given to CTB is 'soil-cement''.

Cement-Modified Granular Soil is obtained when cement is added to marginal or sub-standard granular soil material with the intention of altering certain characteristics, such as plasticity and swell, and thereby increase the load-bearing capacity of the marginal or sub-standard material.

Cement-Modified Silt-Clay Soil is a silt-clay soil to which cement is added to reduce volume change characteristics that can be brought about by changes in water content. Plasticity is reduced and the load bearing capacity of the soil is increased for a wider range of moisture contents.

Plastic Soil-Cement is a soil and cement mixture with enough water added to give it a mortar-like consistency. The material, in its hardened state, is expected to meet

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'The two names are used interchangeably in this thesis.

specified strength and durability criteria[1].

Cement-Treated Slurries and Grouts are soil and cement mixtures with a high water content and usually containing other chemicals. They are sometimes called 'cement-treated pastes and mortars'.

2.4 Materials

The major materials that compose CTB, soil, cement and water, are all required to have certain attributes if an economic mixture is to be obtained. However, in general these requirements are often not very stringent and a good choice of materials can often be found in most circumstances.

2.4.1 Soil

The soil in CTB forms about 75 to 95 percent by weight of the total mixture, and as a result is a very important component. By and large, any type of soil, provided it can be easily pulverized and does not have a high organic or deleterious material content, may be used for CTB. Economics is often the only other consideration. The soils may be in-place or borrow or old recycled materials.

Gradation and plasticity are usually the preliminary criteria used to determine the suitability of soils for use

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[1]In Great Britain, a similar material called lean-concrete is used as a base course material for heavy-trafficked roads. The 'soil' used is expected to comply with requirements for aggregates of conventional pavement-quality concrete.

as CTB. Following are grain size limits of soils recommended in the 1940's by the HRB (1949) as suitable for successful and economic CTB construction:

Maximum Size ....80 000 μm

Passing 5 000 μm Sieve ....at least 50%

Passing 400 μm Sieve ....15 to 100%

Passing 80 μm Sieve ....not more than 50%

In addition their liquid limit, W_l, and plasticity index, I_p, are not to be more than 40 and 18 percent, respectively.

These limits are in line with recent recommendations by the Federal Highway Administration, FHWA, as reported by Oglesby and Hicks (1982). The FHWA recommendations allow the use of soil materials with I_p of up to 30; and soils of the AASHTO classification soil groups A-2 and A-3 are indicated to be particularly suitable for stabilization with cement.Clearly, well-graded gravels and crushed rock (A-1) of base course quality need not be stabilized; and soils with high silt and clay contents (A-4, A-5, A-6 and A-7), usually moist and difficult to pulverize, require such high cement contents that their use for CTB is uneconomical.

2.4.2 Cement

All types of portland cement may be used for CTB, but normal portland cement (CSA Type 10, ASTM Type 1) is usually used. Moderate heat cement (CSA Type 20, ASTM Type 2) and high early strength cement (CSA Type 30, ASTM Type 3) may be used where normal portland cement is unavailable.

Some researchers, including Davidson and Bruns (1960), report of high early strength cement (CSA Type 30, ASTM Type 3) showing a slight economic and/or structural advantage over normal portland cement. Also, expansive cements - normal portland cement with calcium sulphate or magnesium oxide, and sometimes lime, added to it - have been used in CTB to compensate for shrinkage (Barksdale and Vergnolle, 1963; Wang, 1973). However, these and any other specialized types of cement need not be used for CTB unless it is absolutely necessary, as the extra cost incurred is often unjustifiable. Normal portland cement produces CTB adequate for most purposes.

2.4.3 Water

Any potable water fit for drinking is suitable for CTB. The water serves the two purposes of hydrating the cement to form the binder in CTB and lubricating the particles to facilitate compaction. It should be free of excessive amounts of organic substances, salts, oils, acids or alkalis, and generally all impurities that are detrimental to cement hydration, and consequently to the strength and durability of the CTB. It should also be readily available in the required quantities.

2.4.4 Deleterious Materials

A high organic content and an excess of salts as already mentioned are injurious to CTB. Work done by Sherwood (1962a), shows that organic materials combine with the calcium ions liberated by the hydrating cement in CTB and thereby reduce the cementing action in the material. Excess amounts of salts harmful to CTB are also sometimes found in the soil, or water, or both. Smith (1962) reports of the very damaging and costly effect of some sodium and sulphate salts that were in soils used for a CTB pavement in Northern South Australia.

Contrary to the belief that sulphates combine with cement as in concrete to cause the disintegration of CTB, Sherwood (1962b) has determined that a reaction between sulphates and the clay fraction in soils, is largely responsible for the damage in CTB due to excess sulphates. Consequently, CTB of sandy soils are less affected by suplhates than CTB of clayey soils. This sulphate-clay reaction is apparently intensified in the presence of lime and excess water. With regard to salts in water, Smith (1962) points out that, the salts in curing water and not so much as in the water for mixing, seem to be responsible for most of the subsequent damage.

2.4.5 Secondary Additives

Both pozzolanic and non-pozzolanic additives have been used in CTB to enhance some of its properties. Sherwood

(1962a), for instance, has determined that the addition of 1 to 2 percent calcium to soils will alleviate the problems caused by excess organic materials discussed in the preceding section. Bofinger and Duffel (1972) report that fly ash can substantially improve both the tensile and compressive strengths of sandy CTB after long curing periods. Fly ash has also been considered as part replacement for cement in CTB in some instances, but Davidson *et al.* (1958) conclude it is more beneficial as a non-pozzolanic additive or filler. Lambe and Moh (1958) and Lambe *et al.* (1960) also determined that a group of sodium compounds, small amounts of various alkali metals, and a host of other additives improve the strength of CTB, although their effectiveness is limited in most cases.

Various workers suggest the use of certain additives, e.g. lime, fly ash, granular sodium chloride, sugar, and expansive cements, to reduce shrinkage and cracking in CTB (cf. section 2.10.4, *post*). The addition of calcium lignosulfonate and hydroxylated carboxylic acid to CTB, according to a study by Arman and Dantin (1969), allows the creation of bonds between adjacent layers of CTB which makes the construction of layered CTB more feasible. Much more work is however needed to establish clearly the best kinds of additives for CTB. As it is now, additives should be used carefully and applied on a large scale only if there is a definite knowledge of their positive effect from previous use on a smaller scale and they can be economically justified.

2.5 Stabilization Mechanism

Two distinct mechanisms are apparently responsible for the stabilization effect of cement in fine-grained soils and in granular soils to give CTB. Indications are that, in granular soils the stabilization effect is similar to the binding effect of cement in conventional concrete. This conclusion is based on the observed properties of CTB of granular soils and not on any published experimental work. There is a major difference though, of the hydrated cement gel in CTB not filling the voids within the soil mass as in concrete. The strength of such CTB mixtures therefore, depends on the strength of the bonds created between individual particles at their contact points, and on the strength of the soil grains themselves. Consequently, well-graded soils with their greater total area of contact, form better CTB mixtures than soils with a more open or uniform gradation.

Workers including Lambe *et al*. (1960) and Herzog and Mitchell (1963), based on experiments with clay-cement mixtures, propose two reactions are responsible for the cement stabilization of fine-grained soils. As Herzog (1964) points out, the larger size of cement grains in comparison to clay particles, and the relatively small amount of cement involved, makes a concrete-like stabilization mechanism improbable in clay-cement mixtures.

In a primary reaction, hydrolysis and hydration of cement gives the usual cementitious products with a liberation of lime, which according to Kézdi (1979) is more reactive than ordinary lime. In the high pH environment created, the lime attacks the clay particles and causes the dissolution of silica and alumina from them, as well as from surrounding amorphous compounds, during the proposed secondary reaction. The dissolved substances react with the calcium ions present to give secondary cementitious substances.

Both groups of cementitious substances create clay-cement and clay-clay bonds to give a skeletal or matrix structure, with some unbonded soil particles pinned down in their inter-cellular spaces. The semi-rigid structure gives the fine-grained soils the increased strength, decreased plasticity and resistance to frost action associated with CTB; and the chemical surface effect of the cement reduces the water affinity and retention capacity of the soil.

2.6 Nature of CTB

2.6.1 Flexibility

There has been a quandary on whether to classify CTB as a 'flexible' or 'rigid' material. Marwick and Keep (1942), for instance, regarded CTB as a rigid material and reported of its propensity to crack, supposedly an indication of its rigidity. And as recent as 1964, Saunders (1964) expressed the opinion that, allowing cemented materials to act as flexible materials in pavements was tantamount to regarding them as unbound base material and, consequently, defeating the purpose of the stabilization.

Most engineers, however, now consider CTB as a flexible material. A proposal by MacLean and Robinson (1953), based on theoretical analysis, to regard CTB as a flexible base material in pavements has been widely accepted. Jones (1963) from surface wave propagation tests, Yamanouchi (1973) from in-situ measurements with buried crack and modulus of elasticity meters, and Shields *et al.* (1975) from a calculation of combined surface-base modulus from measurements of pavement deflection basin shapes, are a few of the workers who implicitly advance the flexibility theory. Hairline cracking, it appears, divides CTB into several pieces with a subsequent reduction in the modulus of elasticity; however, due to the fineness of the cracks, there is some degree of interlocking between the pieces and the overall stability is not much reduced. The material therefore acts in a manner similar to a well-compacted crushed stone base.

The higher flexural and tensile strengths retained by CTB in comparison to conventional granular base material, however, makes the term 'semi-rigid' or 'semi-flexible' seem more appropriate. In the opinion of Bhatia (1967), designs based on such a classification would compensate for the possibly thicker than required bases used in CTB pavements.

Most design methods do not explicitly take the additional flexural and tensile strengths into consideration (cf.section 2.8.1, *post*), and could therefore represent an over-design, he explains. However, until such a time that pavement designs are developed for 'semi-flexible' or 'semi-rigid' materials, it seems appropriate to consider CTB as a flexible material and regard the additional strength as a built-in safety-factor.

2.6.2 Engineering Properties and Characteristics

By virtue of the method of its production, a number of factors affect the properties and characteristics of CTB. Variability in the nature of each component material, i.e. the soil, cement and water, the mix design procedure, and the construction and curing practices may result in CTB with appreciable differences in their properties and characteristics. Consequently, in presenting the engineering properties and characteristics of CTB, it is normal to report of representative values for CTB of the kind frequently used, and emphasize more on the order of magnitude than on actual values.

Since in most cases, if not all, CTB used in pavements are at the optimum moisture content and compacted to maximum density, it is considered appropriate to report on the properties and characteristics of materials at these conditions. Accordingly, the following discussion is on CTB at the optimum moisture content and maximum density, as for instance detemined from the ASTM D558 or AASHTO T134 tests for the moisture-density relations of soil cement mixtures. Where necessary, equivalent S.I. units are given to familiarize current practioners with significant early works.

Compressive Strength

Data presented by various workers including Reinhold (1955), Felt and Abrams (1957), Johnson (1960), Ingles and Metcalf (1973) and Kédzi (1979) , give representative unconfined compressive strength values of CTB. In Table 2.1, are given typical unconfined compressive strength values for CTB of the three major soil groups frequently used in the United States compiled by Johnson (1960). These values compare favorably with the data presented by the other workers mentioned above, and are generally representative of the CTB used worldwide.

Work by Felt and Abrams (1957), on the four soils whose gradation and consistency characteristics are given in Table 2.2, shows that compressive strength of CTB increases with increase in cement content as depicted by Figure 2.1. The compressive strengths were also found to increase with increase in time of moist curing. Data presented by Reinhold (1955) in Table 2.3, for tests on CTB of synthetic soils of a sand and clay composition, show a similar increase in compressive strength with increase in cement content.

Both the Reinhold (1955) and Felt and Abram (1957) data, in Table 2.3 and Figure 2.1, respectively, also indicate that generally to attain a given strength, soils with a higher fines content require more cement. However, Reinhold (1955) found evidence of an optimum fines content necessary for maximum strength. This is shown by the data in Table 2.3 for the four synthetic soils, with the soil containing 25 percent clay having the highest strength at all cement contents.

Flexural Strength

In Figure 2.2, data from the tests conducted by Felt and Abrams (1957) on CTB of the four soils with the characteristics given in Table 2.2, show flexural strength increases with increase in cement content. For a given flexural strength, soils with higher fines content are also shown to require more cement. In addition, results from the tests indicated an increase in flexural strength with increase in time of moist curing. It is apparent that the trend of these results is similar to that obtained for compressive strength. In fact, the data obtained by Felt and Abrams indicate that, the flexural strength and compressive strength of CTB are almost linearly related, with the flexural strength approximately equal to 20 percent of the compressive strength at all cement contents and at all ages. Larnach (1960) also obtained similar results with respect to the linearity between the compressive and flexural strengths of CTB; however, the flexural strengths were in the order of 28 percent of the compressive strength.

In the tests by Felt and Abrams (1957), the flexural strengths for CTB of the four soils determined from tests on uniformly densified 76- by 76- by 286-mm (3- by 3- by 11 1/4-in.) beams, cured for 7, 28 and 90 days, ranged from 0.35 to 3.3 MPa (50 to 480 psi), at cement contents ranging from 3 to 18 percent by weight of dry soil.

Modulus of Elasticity

As shown in Figure 2.3, from data obtained by Felt and Abrams (1957) from tests on the four soils with the characteristics given in Table 2.2, and according to the data in Table 2.3, presented by Reinhold (1955) for tests on synthetic soils, the modulus of elasticity of CTB increases with increase in cement content. Also, in general, for a given strength a higher cement content is required by soils with increasing fines content; and as in the case of compressive strength, but to a lesser extent, the Reinhold data in Table 2.3 indicates that the maximum modulus of elasticity is associated with an optimum fines content.

According to the results of Felt and Abrams (1957), within the limits of experimental error, the static modulus of elasticity in flexure is equal to the dynamic (resonance) modulus. Values of the moduli for specimens at cement contents between 3 and 18 percent and curing periods up to 90 days ranged from 4 800 to 29 700 MPa (7 x 10^6 to

43 x 10' psi). On the other hand, static modulus in compression values were found to range between 60 to 75 percent of the static modulus in flexure. In every case, however, the modulus of elasticity increased with increase in the time of moist curing.

Poisson's Ratio

Not much has been reported on the Poisson's ratio of CTB materials. There is the data presented by Reinhold (1955), summarized in Table 2.3, which gives the Poisson's ratio obtained from compressive strength tests, and shows it generally decreases with an increasing clay content. Dynamic Poisson's ratio values determined by Felt and Abrams (1957), from fundamental transverse and torsional frequencies of CTB beams of the four soils with the index properties given in Table 2.2, were quite variable and ranged from about 0.24 to 0.36. Similarly, values determined from strain measurements were extremely variable ranging from about 0.08 to 0.24. Felt and Abrams were of the opinion that the variability was somewhat related to the manner in which the specimens failed. It must however be pointed out that the very mode of failure of a material is itself influenced by its elastic properties, which includes the Poisson's ratio.

Resilient Modulus

Resilient modulus, M_R, defined as the ratio of the repeated axial deviator stress to the recoverable axial strain, is the most frequently used parameter for characterizing the response of CTB to repeated loading. In Table 2.4, are given values of M_R determined by various workers for CTB of a number of soils. The M_R ranges between 90 and 170 x 10^3 MPa (13 x 10^3 and 25 x 10^4 psi), and represents values determined by a variety of laboratory repeated loading tests including compressive, tensile and flexural tests, as well as full-scale tests on prototypes and actual pavements.

The wide range in M_R is due to a variety of factors. Shen and Mitchell (1966), Morgan and Williams (1970) and Yamanouchi (1973) all report of the influence of the number of stress repetitions and the magnitude of the applied stresses on M_R, and the work of Morgan and Williams (1970) show that M_R is dependent on stress path. Work by Shen and Mitchell (1966) and Fossberg *et al*. (1972a), indicate that CTB exhibits anisotropy in its response to repeated loading. Consequently, as shown by the results obtained by Shen and Mitchell (1966) in Table 2.4, there may often be appreciable differences between the resilient moduli determined from different tests.

In general, however, resilient modulus increases with increase in cement content for CTB of both sandy and clayey soils, as reported by Shen and Mitchell (1966) and Morgan and Williams (1970). Shen and Mitchell (1966) and Fossberg *et al.* (1972a) also report of an increase in resilient modulus with increase in time of moist curing. For CTB of the four soils with the index properties given in Table 2.5 Chang *et al.* (1982) obtained similar results as shown in Figure 2.4[3].

Resilient Strain Ratio

Few workers have reported on the resilient strain ratio, ν_R, of CTB, a parameter analogous to the Poisson's ratio of elastic materials. Values of ν_R, the ratio of resilient radial to axial strains, reported by Morgan and Williams (1970) and Fossberg *et al.* (1972a) are given in Table 2.4. Like resilient modulus, the number of stress repetitions, the stress level, the stress path and the anisotropy of CTB influences ν_R. As a result the ν_R determined by various tests may be quite different.

According to the work of Morgan and Williams (1970), there is a reduction in the resilient strain ratio with increase in cement content. Also, although Shen and Mitchell (1966) have determined that an increase in time of curing

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[3]An interesting finding of the work by Chang *et al.* (1982) on the four aggregates, one of which was a high quality aggregate chosen for comparative purposes, was the substantial decrease in resilient modulus for specimens subjected to cycles of wetting-and-drying, when the resilient modulus of similar specimens subjected to cycles of freezing-and-thawing were hardly affected. While this might be peculiar to the particular soils tested, it is quite significant that the resilient moduli were decreased by as much as 20 to 60 percent in all cases.

decreases the magnitude of the resilient strains, Fossberg *et al.* (1972a) indicate that the resilient strain ratio is itself not affected by curing.

Fatigue Characteristics

Fatigue life tests by Chang *et al.* (1982) on CTB of the four soils with index properties given in Table 2.5, show that increases in applied stress, decreases the fatigue life or number of repetitions to failure of CTB materials; and in general, for a particular applied stress, the fatigue life decreased when the CTB materials were subjected to either wetting-and-drying or freezing-and-thawing cycles. Work by Bofinger (1965; 1969) show a similar decrease in the fatigue life of CTB with increase in the magnitude of the applied stress. In addition, he determined that for a given number of load applications, an increase in the cement content increases the magnitude of the stress necessary to cause failure. Presoaking drastically reduced the fatigue life and an increase in the initial dry density increased the fatigue life.

Shen and Mitchell (1966) and Morgan and Williams (1970) suggest the existence of a critical stress, ranging from 0.5 to 0.9 of the static strength, above which a few cycles of stress often causes failure. However, other works including the detailed work by Bofinger (1965; 1969) gives no hint of such a critical stress level.

Other Strength and Elastic Properties

A strength property often used to characterize CTB materials is the California Bearing Ratio (CBR) value. In Table 2.6, are given CBR values of CTB of various types of soils with a minimum unconfined compressive strength of 1.72 MPa (250 psi) after 7 days curing reported by Ingles and Metcalf (1973). Plate-bearing test results reported by Abrams (1959) and Nussbaum and Larsen (1965), show that CTB bases can carry loads up to three times greater than other low-cost base materials of the same thickness. Work by Chiang and Chae (1972) on CTB of a uniform sand and a silty clay, also show that the addition of cement to the soils greatly increases both dynamic shear modulus and damping (or energy dissipation) characteristics.

Plasticity

It is generally known that the addition of cement, even in quantities less than that required to meet CTB criteria, reduces the plasticity of both granular and clayey soils. In fact, the plasticity index, I_p, can be reduced to any desired level with the addition of cement. In Figure 2.5, data presented by Robbins (1964) shows the complete elimination of the plasticity of a gravelly sandy loam with cement addition. The similar and total reduction in the I_p of the material when subjected to cycles of freezing-and-thawing, is indicative of the permanency of the reduction. Further evidence of this is given by data collected by Redus

(1958) from a study of CTB airfield pavements, which show that CTB materials with some plasticity at the time of construction had none when tested one to ten years later. Figure 2.6, which represents data from one of the airfields, shows an increase in the magnitude of the reduction in I_p with increase in cement content, as well as a decrease in I_p at a particular cement content with increase in age of the CTB pavement.

Volume Changes

Swelling, shrinkage, thermal expansion and contraction, and frost heave are the usual forms of volume change in CTB, and may take place either singularly or concurrently. The type of [illegible], cement content, moisture content, thermal properties and susceptibility to frost action of the CTB, as well as temperature variations, are the major factors which influence the occurrence and magnitude of volume change in CTB.

The stabilizing action of the cement added to soils (cf. section 2.5, *ante*) generally decreases the potential of volumetric change and the tendency to swell or shrink decreases appreciably. Results of tests by Mehra and Uppal (1950), for instance, show such a reduction in the volumetric and lineal shrinkage of CTB with increasing cement content. Paradoxically, it is the shrinkage of the cement gel resulting from the addition of cement which is in fact responsible for shrinkage in CTB of non-cohesive soils.

In general, however, there is an optimum cement content at which the different kinds of shrinkage are minimal.

Thermal expansion and contraction of CTB depends on the thermal properties of the material, which are often similar to those of the untreated soil. In Table 2.7 are given representative values of the coefficient of thermal expansion of some compacted CTB presented by Mehra and Uppal (1951) and Catton (1952). The studies by Mehra and Uppal (1951) on the four soils also show that, increase in density increases the susceptibility of CTB to thermally induced volume change and, at a constant density, increase in cement content increases the volume change capabilities of CTB.

Damage to CTB by frost action as a result of alternate freezing and thawing at capillary saturation is very limited, due to the criterion for CTB which limits the volume change during the standard freeze-thaw test (ASTM D560; AASHTO T136) to not more than 2 percent. On the other hand, heaving caused by the advancement of a freezing front in an unsaturated mixture may lead to the severe disruption and damage of CTB pavements. Studies by Kettle and Williams (1976, 1977), however, show that the addition of cement to soils by decreasing permeability while at the same time increasing pore sizes (especially in clayey soils), and by increasing tensile strength, reduces ice lensing and the effect of the forces developed during ice lensing, which are mainly responsible for frost heave.

Permeability

There is, in general, a decrease in the permeability of soils with the addition of cement, but this decrease primarily depends on the nature of the soil and the quantity of cement added. Cement addition to sandy soils largely results in a decrease in permeability, and data presented by Johnson (1960), for instance, indicates that the addition of 8 to 10 percent cement to some coarse to fine sands decreases the permeability by as much as 90 percent.

On the other hand, the extent of decrease in the permeability of fine-grained soils with the addition of cement is much less. In fact, Kettle and Williams (1977) report of an initial increase in the permeability of a clay-like shale of the finest grading with the addition of a small amount of cement. The explanation given is that the initial aggregation of particles increases pore sizes and therefore permeability, but subsequent cement addition accompanied by the pore-filling action of the hydration products, offsets the open pore structure and there is a reduction in permeability in the end.

Catton (1952) reports of a permeability of 5.2×10^{-6} mm/s and 6×10^{-6} mm/s respectively, for compacted CTB of a sandy soil with 8 percent cement and a silty soil with 12 percent cement. Attention is also drawn to the possible increase in the permeability of a CTB pavement due to cracking, which is an inherent feature of that kind of pavement.

Compaction Characteristics

In general, the optimum moisture content for adequate compaction and the maximum density that can be attained for CTB mixtures, are almost the same as those of the untreated parent soil. Deviations, if any, are a few percentage points from the values for the untreated soils.

2.6.3 Factors Which Affect CTB Properties

In addition to the general requirements for the basic component materials, and the effects of secondary additives as well as certain deleterious materials on CTB mixtures, discussed in section 2.4, *ante*, a number of factors greatly influence the properties of the final CTB in pavements. If not properly controlled during CTB production and construction, they may result in a final material without the necessary strength and durability, even though the total amount of cement as determined by the appropriate criteria might have been added.

Soil Pulverization

More often than not, the actual soils which have been determined by the laboratory examination of samples to be suitable for CTB construction from the point of view of proper gradation and physical and chemical properties, need to be pulverized in the field. In general, a degree of pulverization which would ensure 80 percent of the material, excluding larger stone-size particles, passes the 5 000 μm sieve size would be highly desirable. But this is hardly ever achieved, especially in the case of moist cohesive soils.

Work by Grimer and Ross (1957), show a decrease in the unconfined compressive strength of clay-cement mixtures with an increase in the percentage of plus 5 000 μm aggregates. This investigation and others, however, show that the strength and durability of CTB mixtures are not decreased much if the percentage retained on the 5 000 μm sieve size is not more than 30 percent. The HRB (1949) recommendation of at most 50 percent material retained on the 5 000 μm sieve (cf. section 2.4.1, *ante*) should be the absolute maximum.

Cement Mixing

Apparently, the attainment of a uniform cement and soil mixture is highly desirable for an even distribution of strength in a compacted CTB layer. Unequal cement distribution in the vertical direction could, for example, result in the development of unequal shrinkage and other stresses within a pavement, and lead to subsequent failure. Robinson (1952) and Baker (1955) also present data which indicate an increase in the overall strength of CTB mixtures with increase in mixing uniformity.

A crude measure of the uniformity or efficiency of mixing is the homogeneity in the colour of a resultant mixture, but a much better standard developed by the Road Research Laboratory (RRL) has been reported by Robinson (1952). The mixing efficiency is determined as the ratio of the average compressive strength of specimens formed from a field mix, to the average compressive strength of specimens formed from field mix material further mixed in the laboratory. An efficiency of 60 percent by this standard is considered high enough. Baker (1955) also reports of the successful use of radioactive-tracer techniques for measuring the uniformity of mixing.

Using the RRL method, Robinson (1952) determined that an increase in mixing efficiency to about 80 percent, could for example amount to a 5 percent savings in cement at the same moisture content, for a mixture with an expected field compressive strength of 1.72 MPa (250 psi). It should be noted, however, that prolonged mixing is undesirable since partial cement hydration makes compaction difficult and ultimately results in a decrease in strength. Felt (1955) suggests a maximum of 4 hours as the safe mixing time, provided the mixture is intermittently mixed several times within that period.Specifications commonly control the mixing time.

Moisture Content

The moisture content for CTB mixtures is largely determined by the moisture required for compaction since the water needed for hydration is only a small fraction (1/4 according to PCA, 1971) of the weight of the cement added. However, the optimum moisture content which corresponds to the maximum density (cf. section 2.7.1, *post*), does not seem to necessarily give maximum strength and best quality material. Extensive tests conducted by Felt (1955) and Davidson *et al*. (1962) show that, CTB of sands and sandy soils give maximum strength and best quality material when compacted at moisture contents slightly below optimum. On the other hand, CTB of clayey soils and to a lesser extent of silty soils, give the best results when compacted at moisture contents slightly above the optimum, or at least at the optimum moisture content.

Compaction and Compacted Density

Larnach (1960), Maclean and Lewis (1963), and Ingles and Frydman (1966) all report, either directly or indirectly, of an increase in the strength of CTB with increase in compacted density. Consequently, compaction which will give densities equal to or higher than the maximum dry density (ASTM D558; AASHTO T134) is always desirable. As such, an almost immediate start of compaction and use of the adequate compactive effort is very necessary.

West (1959) has demonstrated that for CTB of a clay and sandy gravel, a delay between the end of mixing and compaction reduces strength appreciably. Work by Arman and Saifan (1967) also indicate there are losses in the density, strength and durability of CTB mixtures, whether they are compacted at, below, or above the optimum moisture and cement contents, when there is a delay in compaction of approximately two or more hours. Conglomeration of individual particles accompanied by the setting of the cement gel, which makes compaction rather difficult, are responsible for the losses.

Lightsey *et al.* (1970) have, however, determined that the addition of excess compaction moisture after such delays may significantly reduce the losses in density, strength and durability by lubricating the irregular shaped aggregates and facilitating easier compaction, as well as allowing further cement hydration. Use of set-retarding agents and the provision of thicker bases than required are also other means of reducing the losses. And, as noted by Cowell and Irwin (1970), if compactive efforts existed which could compact to the necessary densities after a delay in compaction, there would be no appreciable losses. Unfortunately, however, most of the currently available highway compaction equipments do not have those capabilities.

Work by Shen and Mitchell (1966) and El-Rawi *et al.* (1967; 1968) also show that compaction methods which enhance a flocculent rather than dispersed structure produces higher strength CTB materials. Consequently, with regard to the strength of CTB mixtures, impact compaction is, for example, preferable to kneading compaction.

Curing Time and Conditions

In general, as discussed in section 2.6.2, *ante*, the strength and elastic properties of CTB mixtures are favorably affected by increase in curing time. Workers including Clare and Pollard (1954), Circeo *et al*. (1962) and Shackel and Lee (1974) have demonstrated an increase in the strength of CTB with increase in time of curing. In fact, evidence presented by Circeo *et al*. (1962) suggest the increase in strength may go on for as long as five years, and possibly further, after the initial compaction.

Investigations by Clare and Pollard (1954) and Metcalf (1963) have shown that at the same cement content and time of curing, CTB mixtures cured at higher temperatures attain greater strengths. An interesting finding of the work by Metcalf (1963) was that cyclic curing, i.e. alternately curing at a low and high temperature, has the same effect as curing at a constant temperature between the two extreme temperatures. Clare and Pollard (1954) also found that CTB hardens at all temperatures except below 0 C, and even at 0 C there is an increase in strength with increase in time of curing.

Shackel and Lee (1974) also considered the effect of the type of curing on CTB and concluded that moist-cured mixtures gain more strength than air-cured mixtures at all cement contents. This, and the other points raised above, no doubt give a clear indication of the attention CTB curing merits.

Nature of Test Specimens

The nature of test specimens may affect the properties of a particular CTB mixture by giving misrepresentative property values. For example, results obtained by Felt and Abrams (1957) indicate that compressive strengths of cylindrical CTB specimens decrease with increase in the length to diameter ratio, and cube specimens have greater strengths than cylindrical specimens. These results are in agreement with ASTM standards[4]. In general, however, the consistent use of specimens of a particular size and shape will reduce the variability introduced thereof. Cylindrical specimens with a length to diameter ratio of 2:1 are therefore normally used in North America for the compressive strength and durability tests.

Workers including Chadda (1956), Felt and Abrams (1957), Davidson *et al.* (1962) and Shen and Mitchell (1966) have also shown that strength parameters, such as compressive strength and resilient modulus, are higher for

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[4]Methods of Securing, Preparing, and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths (C42-49), 1955 Book of ASTM Standards, Part 3, p. 1360.

dry or unsoaked specimens, than for moist or soaked specimens. Reduced surface tension effects, a decrease in bonding forces and water acting as a lubricant between individual particles are believed to be responsible for the reduced strength of soaked specimens[5]. Since, as Lay (1984) points out, most of the materials placed in pavements at or near their optimum moisture contents remain in that state for most of their design life, it is important that specimens be moist-tested rather than tested in the dry state.

2.7 Mix Design

The mix design procedure involves the determination of the minimum cement content, the optimum moisture content and the maximum density at which a CTB will have enough strength and durability for successful use in a pavement. In view of the usual empirical nature of current structural design procedures, the quality of CTB pavements oftentimes depend on the quality of the mix design procedure used. Methods of mix design range from the empirical determination of the cement content to elaborate laboratory tests which give the minimum and most economic cement content. Some put emphasis on the strength requirements of CTB, while others consider durability as the more important requirement. Whatever the

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[5]Paradoxically, Felt and Abrams (1957) found the modulus of elasticity of dry specimens, especially after oven drying were lower than those of moist-tested specimens. The decrease was not much though in the case of air dried specimens.

philosophy behind a procedure, most mix design methods if diligently followed, produce materials which meet both strength and durability requirements.

2.7.1 Optimum Moisture and Maximum Density Determination

Since the optimum moisture content and maximum density of CTB mixtures are more or less equal to those of the parent soils (cf. section 2.6.2, *ante*), tests based on the conventional standard or modified compaction tests (ASTM D698; AASHTO T99 or ASTM D1557; AASHTO T180) are often used on mixtures molded at a preliminary cement content, to determine the optimum moisture content and maximum density. Details of compaction tests for CTB mixtures used in North America (ASTM D558; AASHTO T134) can be found in the *Soil-Cement Laboratory Handbook* of the PCA (1971).

2.7.2 Cement Content Determination

Cement by Soil Series

Work done by Leadabrand *et al.* (1957), Handy and Davidson (1960) and others show that soils of the same series often require equal amounts of cement for their stabilization. As a result, the known cement content for CTB of a particular soil, can sometimes be used for other soils with the same characteristics of subsoil, parent material, climate, vegetation and age. This method is only suitable for use on relatively small projects within a particular locality, and also frequently, for determining the preliminary cement contents at which more elaborate laboratory tests are to be conducted for the minimum and most economic cement content.

Durability Tests

The most common durability type tests for determining the cement content for a CTB mixture are the wet-dry and freeze-thaw tests (ASTM D559; AASHTO T135 and ASTM D560; AASHTO T136) developed by the Portland Cement Association (PCA) in the late 1930's.

In both tests, specimens of CTB at preliminary cement contents are molded at the optimum moisture content and maximum dry density, cured for 7 days in an atmosphere of high humidity at room temperature. Some of the specimens are subjected to 12 cycles of a standard wet-dry test and others to 12 cycles of a standard freeze-thaw test. Based on a number of criteria, the minimum cement content considered enough to overcome the disruptive forces of swelling and shrinkage, and the expansive forces of freezing water, during the wet-dry and freeze-thaw tests respectively, is determined. As these forces, more than any externally applied force, are generally representative of the forces that may be present in CTB in service, this cement content is considered suitable for construction.[1]

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[1] It is emphasized that the durability tests are not weathering tests nor related to the climate of an area. They are simply the most dependable and reproducible means of

The criteria used were developed by the PCA and pertain to CTB losses after a standard brushing procedure, maximum volume change and maximum moisture content of the test specimens, and the trend of the compressive strength of compacted CTB mixtures. Following are the PCA criteria (Catton, 1959) for determining the minimum cement content:

a) CTB losses of specimens subjected to 12 cycles of either the wet-dry or freeze-thaw tests shall not exceed the following for the specified Unified Classification System soil groups:

   Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent.

   Soil Group A-2-6, A-2-7, A-4 and A-5, not over 10 percent.

   Soil Groups A-6 and A-7, not over 7 percent.

b) Maximum volume at any time during the wet-dry or freeze-thaw tests shall not exceed the volume at time of molding by more than 2 percent.

c) At all times during the tests the maximum moisture content shall not exceed that quantity which would completely fill the voids of the specimen at the time of molding.

d) Compressive strength of CTB specimens soaked in water for 1 to 4 hours shall increase both with age and with increase in cement content for cement

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*(cont'd) simulating the disruptive forces that may occur in CTB pavements.

contents equal to and above the ranges which satisfy 1, 2 and 3 above.

These durability type tests, widely used in Canada and most parts of the United States, require at least 32 days for completion. Details of the tests, namely the wet-dry and freeze-thaw tests, can be found in the *Soil-Cement Laboratory Handbook* of the PCA (1971).

Compressive Strength Tests

The most frequently specified and measured property of CTB, unconfined compressive strength, is also often used in the determination of the cement requirements of the material. Specimens molded at the optimum moisture content and maximum density and at preliminary cement contents, are moist-cured for 7 days and tested for their unconfined compressive strengths. The design cement content is determined as the minimum at which the compressive strength of specimens exceed a specified minimum.

In Great Britain, where this method is widely used, Maclean and Lewis (1963) report that a minimum 7-day compressive strength of 1.72 MPa (250 psi) is normally specified for CTB materials[7]. Australia, New Zealand and most Commonwealth countries all use the same minimum compressive strength. Hveem and Zube (1963) report of a

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[7]In Britain, there is an additional requirement that all materials within 450 mm of the running surface of a roadway be frost resistant, according to Road Note No. 29 of the RRL (1970).

rather stringent requirement of a minimum 7-day compressive strength of 5.17 MPa (750 psi) in California for CTB (Class A) mixtures with cement contents between 3 1/2 and 6 percent*.

In general, the compressive strength test method of mix design has given quite satisfacory results. However, as Bhatia (1967) points out, the use of a single minimum compressive strength for both clays and clayey soils and sands and sandy soils, may represent an over-design in the latter case. Sands and sandy soils, by reason of their higher internal friction, are capable of mobilizing much higher strengths under the confined conditions in pavements, than clays and clayey soils.

The CBR Test Method

Since cement addition increases the CBR of soils, an indicator of their load-bearing capacities, it may seem feasible to establish a minimum CBR value for determining the minimum cement requirements of CTB mixtures. Maclean (1956), for instance, suggested the use of a minimum CBR value of 120 percent as criterion for accepting CTB mixtures as base course material for pavements in Britain.

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*Maclean and Lewis (1963) and Hveem and Zube (1963), as do a lot of others, often indicate that the preference of compressive strength tests over the durability tests for mix design stems from the milder weather conditions experienced in the areas where the former are used. This gives the erroneous impression that the durability tests are in some way connected to the weather or climate of an area. As was pointed out earlier on, this is absolutely not the case.

It appears, however, that the CBR value of stabilized soils depends on the type of soil, and the CBR method cannot be relied upon to give consistent values of cement contents. Dos Santos (1961) has shown, for example, that CTB mixtures with the same CBR value may often have different weight losses during the wet-dry test (ASTM D559, AASHTO T135); and similarly, CTB mixtures with the same 7-day compressive strength and cement content tend to have different CBR values. Thus, although CTB mixtures of two different soils may have equal CBR values, their cement contents may be very different. In addition, as the original CBR procedure entails comparison to a well-graded non-cohesive crushed rock assumed to have a CBR of a 100 percent, interpretation of CBR values above that presents some difficulties. That coupled with the inherent flexural and tensile strength of CTB, makes the CBR procedure unsuitable for CTB mix design.

Statistical Methods

Methods for determining the cement requirements of CTB mixtures based on statistical correlations, have been derived in an effort to reduce the time and quantity of material needed for most of the procedures described above. Two of such methods most often used are:

1. The PCA Short-cut Method.

Published reports by Leadabrand and Norling (1953; 1956) and Norling and Packard (1958) give an account of the development of this PCA method. Involving no new tests, data collected from previous durability tests have been correlated to reduce the mix design procedure for sands and sandy soils to the following steps:

a) A grain-size distribution analysis

b) A moisture-density relations test

c) Determination of cement requirements from charts

d) Verification of the indicated cement requirement with compressive strength tests.

Details of this short-cut method are given in the *Soil-Cement Laboratory Handbook* of the PCA (1971).

This procedure does not always indicate the minimum cement content, but reducing the cement content with the view of economizing can be disastrous. Such a decision can only be taken if it is backed by one of the more elaborate laboratory tests discussed above.

2. Surface Area Method.

By correlating the known cement requirements of plastic soils with their corresponding surface areas determined from a glycerol retention test, Diamond and Kinter (1959) derived the following equation for obtaining the cement content of plastic soils:

$$\text{Cement Content} = 0.87\ (\text{Surface Area}) + 3.79 \qquad (2.1)$$

According to Diamond and Kinter, the cement content calculated from this equation is that which allows a maximum of 10 percent weight loss for CTB subjected to 12 cycles of the standard freeze-thaw test (ASTM D560; AASHTO T136). Consequently, adjustments have to be made when the procedure is used for soils which are allowed weight losses of 7 percent and 14 percent by the PCA criteria; the cement contents are increased by 2.0 percent and decreased by 0.7 percent, respectively.

It is also considered advisable to conduct compressive strength tests on specimens molded at the calculated cement content, and at plus-and-minus 2% cement contents away, to ensure the cement is adequate for hardening of the CTB.

Other Short Methods

A 'rapid test method' for determining the cement requirements for small and emergency projects, and also useful for verifying cement contents in the field, is worth mentioning. The method, described in the *Soil-Cement Laboratory Handbook* of the PCA (1971), consists of the inspection of CTB specimens by "picking" with a sharp pointed instrument and "clicking" cured specimens together. The relative hardness of the test specimens is the basis for determining the cement content. Compressive strength tests are considered expedient to affirm the empirically determined cement contents.

Methods requiring shorter times but not put to much practice have been reported by various authors including

Kemahlioglu *et al.* (1967), Yedavally and Anderson (1972) and Dempsey and Thompson (1973). Some agencies also use a combination of shear tests or tensile strength tests, and compressive strength tests to determine the cement content, which in most cases reduces the time and quantity of material otherwise required.

2.8 Structural Design

In spite of the widespread and approximately half-a-century of the serious use of CTB, no universal or 'standard' structural design procedure exists presently. In part, the good performance of in-service pavements mostly designed by empirical methods has not generated much interest towards the development of elaborate structural design procedures. Consequently, apart from methods recommended by the PCA, AASHTO and the California Division of Highways which take some strength parameter into consideration, most CTB structural designs basically comprise of a selection of the CTB and surfacing thicknesses based on past experience.

2.8.1 CTB Thickness Design

The PCA Method

This PCA (1970) method which takes into account the load-deflection characteristics and the structural and fatigue properties of CTB; the subgrade strength; and the pavement design life, is the only one of its kind developed exclusively for the design of CTB pavements. Essentially, it involves the evaluation of a "Fatigue Factor", an expression of the fatigue effects of the total number and weight of axle loads over the design period of a CTB pavement, and the determination of the Westergaard modulus of subgrade reaction *k* from plate-loading tests on the underlying subgrade. The thickness of the CTB layer is read from charts which give thickness as a function of *k* and the fatigue factor. The charts provide for a minimum thickness of 130 mm and a maximum of 230 mm'.

British Practice

Since CTB pavements are regarded as flexible pavements in Britain (cf. section 2.6.1, *ante*), it is the practice to design them as such, using the recommendations of the Road Research Laboratory given in Road Note No. 29 (RRL, 1970) for the design of flexible pavements. CTB thicknesses are determined from charts which relate the subgrade CBR and the cumulative 8 160-kg (18 000-lb) equivalent single axle loads (ESALs) expected on the pavement to thickness. However, CTB

'There were no published reports available to the author on the use of the PCA method in current practice, successfully or otherwise.

material is restricted for use on pavements which carry a maximum of 1.5 x 10^6 cumulative ESALs during their useful service life. The chart allows for CTB thicknesses of between 120 to 150 mm (5 to 6 in.).

A similar procedure is used in Australia where by the CBR method design depths are obtained as would be required for a conventional granular base flexible pavement. In view of the accepted increased stiffness, shear strength and better load-distribution characteristics of CTB, Hansen (1962) reports of the application of a factor between 0.58 and 0.70 to reduce the design depths to values used for CTB as base course.

<u>North American Practice</u>

Generally, the structural design of CTB pavements in Canada and the United States is confined to the arbitrary selection of a thickness based on past experience. Thicknesses recommended by a committee of the Highway Research Board in 1949 (HRB, 1961) are generally regarded as the basis for such designs. The thicknesses are shown in Table 2.8 for various types of subgrade soils.

A method developed by the California Division of Highways for the design of flexible pavements, has provisions for its application to the design of CTB pavements (HRB, 1961). It involves use of the cohesive resistance of the CTB and any overlying material; the traffic effects converted into a 'traffic index'; and the resistance value *R* of the subgrade and/or subbase soils at equilibrium, to determine the base and other thicknesses from a design chart. According to Hveem and Zube (1963), this procedure results in a reduction of 30 to 45 percent in the base thickness than would be required for an equivalent pavement with an untreated granular base.

Similarly, the AASHTO Interim Guide method (1972) for the design of flexible pavements also has provisions for the design of CTB pavements. Structural layer coefficients are proposed for CTB, which in combination with a structural number *SN* – derived from road-bed soil conditions, an analysis of traffic and corrected by a regional factor – can be used to determine the thickness of the soil-cement base required in a pavement structure. However, in an evaluation of the AASHTO design method, Van Til *et al*. (1972) report that most agencies use their own modified coefficients in the design of CTB pavements. In fact, Yoder and Witczak (1975) indicate that the original coefficients proposed by AASHTO are not for soil-cement, but are mostly for higher strength cement treated materials with strengths in excess of 2.76 MPa (400 psi) at 7 days.

2.8.2 Surfacing Thickness

In order to protect CTB from the adverse effects of the environment, and most importantly from the abrasive effects of traffic, the base course is always covered with protective asphaltic surfacing layer(s). In addition, Fossberg *et al.* (1972a) have determined that, for both cracked and uncracked bases of the material, an asphaltic surfacing reduces the deflections in the base. Results from the work also indicated that subgrade stresses and, vertical stresses and horizontal strains in the base, particularly in the upper part, are also reduced with the provision of a surfacing layer.

Thicknesses in North America range from 25 mm to 100 mm. The design chart in Road Note No. 29 of the RRL (1970) used in Britain provides for a minimum surfacing thickness of 50 mm. The type of surfacing material depends on local conditions and preferences. It is not considered advisable to reduce the CTB thickness on account of the structural strength contributed by the surfacing material.

2.9 Construction Practices

There are basically two methods for constructing CTB pavements. They are the mixed-in-place method which involves mixing water and cement to in-situ soil, either of the underlying material or obtained by spreading borrow material on a prepared subgrade; and the plant-mix method where cement and water is added to excavated soil in a central plant and the CTB obtained transported to the roadway site for use. The construction procedures which follow the stage where the CTB mixture is spread on the subgrade or subbase are basically the same for both methods.

Various mixing machines are available for producing CTB by both the mixed-in-place and plant-mix methods. They include the flat-transverse-shaft mixers (single or multiple shafted) and the windrow-type mixers for the former method; and the continuous-flow-type pugmill, the batch-type pugmill and the rotary-drum mixers for the plant-mix method. Details of how these mixers operate can be found in the *Soil-Cement Construction Handbook* of the PCA (1979).

2.9.1 Construction Procedures

The construction procedures basically comprise of subgrade preparation, CTB layer construction, compaction, finishing and curing, and surfacing. Generally, the specific details of construction are determined by local conditions, experience and practice.

In summary, subgrade preparation essentially involves shaping the subgrade to the proper crown and grade, with the removal of unsuitable soil material where necessary. In the case of construction with plant-mixed CTB, the material is hauled to the roadway and spread on the compacted subgrade. The use of automatic spreaders makes this process a relatively simple operation. Where construction is by the mixed-in-place method, the subgrade is scarified and the required amounts of cement and water added to the soil in-situ. There is thorough mixing to ensure a uniform mixture is obtained, and depending on the method used re-spreading of the mixture may be required.

In both methods, immediate compaction of the soil and cement mixture follows, with the intention of compacting the material at its optimum moisture content to the highest density, without causing failure by exceeding the bearing capacity of the CTB mixture. Recommendations on the types of compaction equipment which will give the most favorable results for CTB of the different soil types are given in the *Soil-Cement Construction Handbook* of the PCA (1979). For compaction to be effective, lifts are built in thicknesses not to exceed 200 to 300 mm (8 to 9 in.) after compaction. Above that thickness, the CTB layer is built in two or more lifts.

Finishing comprises the removal of compaction planes and the correction of any surface irregularities. The CTB material is kept damp throughout all surface correcting measures, and a firm and smooth surface is the final result required. The usual method of curing is the application of a light bituminous cover, examples of which include medium cutback, rapid cutback, and emulsified asphalts. Sanding of the bituminous cover to prevent pickup by traffic, or waiting for the volatiles to evaporate before allowing traffic on the pavement is considered good practice. Covering pavements with wet material such as sacks, straw and soil or with waterproofing material such as plastic sheets; or spraying with water are other possible curing methods.

Most practices require a curing period of 7 days before traffic is allowed on CTB pavements. Studies in Japan, Britain and the United States indicate, however, that there is no advantage to be gained from that; and provided the structural capability of a CTB pavement is not exceeded, traffic can be allowed soon after construction without any detrimental effects (cf. section 2.10.4, *post*). Placement of a bituminous surfacing cover is often delayed for some time to minimize reflective cracking of the surfacing (cf. section 2.10.4, *post*). For the same reason, placing the bituminous cover in two lifts about a year apart is common practice.

2.9.2 Preference of Construction Method

There seems to be a general preference for use of the plant-mix method of construction in Canada and the United States, with the mixed-in-place method mostly reserved for minor projects. The frequent use of borrow or imported materials and the general belief that the plant-mix method ensures a more uniform mix are the probable reasons for this preference. There is also the advantage of the plant-mix method ensuring a more regular surface, since most of the spreaders used are usually fitted with automatic control devices which regulate the thickness of the material spread. In addition the plant-mix method is better suited for multiple layer CTB construction than the mixed-in-place method.

It must, however, be pointed out that the general belief that plant-mix construction provides much better mixes than mixed-in-place construction has no firm basis. In Australia, Germany, Holland and Hungary use of the latter method has provided high quality pavements built with mixes as good as any. As Lilley (1972), Ingles and Metcalf (1973) and Kézdi (1979) report, even in cases where borrow raw soil material is used, the mixed-in-place method is still preferred in those countries. The mixed-in-place method of construction is also known to reduce shrinkage cracking by enhancing stress redistribution at the subgrade-base interface (cf. section 2.10.4, *post*).

In some instances, other factors play a role in the choice of method of construction. For example the inability to make the greater initial investment for a central plant and the fleet of trucks needed for a plant-mix operation, tends to make the mixed-in-place method more feasible in developing countries. The possible use of nearby agricultural machinery, such as ploughs and disc harrows, also makes the mixed-in-place construction method more attractive. However, use of such machinery, other than those designed purposely for CTB construction, should not be encouraged.

2.9.3 Specification and Control

Generally, the owner specifies the cement requirement for the CTB mixture, the density to be achieved in the field and the optimum moisture content at which this density can be achieved. Compliance with these specifications and adequate curing will ensure that a high quality CTB material is obtained. Close control initially, to make certain all construction procedures are right, will reduce the amount of testing required in the latter stages of construction.

In some agencies in Canada and the United States, the client checks for the proper cement addition by the use of catch-trays, chemical analysis and from the daily cement consumption. The chemical analysis test also sometimes doubles as a check for uniformity of mixing. On the other hand, as in Australia, Britain and most European practice, the control of cement addition is left to the contractor, with the other compliance testing expected to show up any deviations.

Density is normally specified relative to a standard maximum achieved in the laboratory. For example, the density determined shortly after compaction may be required to be at least 95% of the maximum ASTM D558 or AASHTO T134 or Proctor density. A minimum 7-day compressive strength for cores obtained from the completed pavement is often specified to check for adequate curing and strength gain of the end result. As indicated above, in Britain, Germany, Holland and Australia this minimum strength is also the only measure of whether the proper amount of cement was added by the contractor.

Liability for the proper crown and grade of the roadway and depth of the CTB layer rests with the contractor. The depths are checked by probing and coring, but rarely are the levels checked. While some clients specify the method of construction, others leave it to the discretion of the contractor, but subject to approval.

The general difficulty and undesirability of reworking and reconstructing hardened CTB material, often leads to pavements which fail compliance testing being accepted as is. It is imperative, therefore, that very close attention be paid to first construction in order to achieve very high standards.

2.10 Performance

The continued and increased use of CTB material in pavements over a long period is a strong indication of its satisfactory performance in the opinion of users. Numerous laboratory and field studies have also shown that CTB can provide support equal to, and sometimes greater than, conventional unbound base course materials under different load and environmental conditions.

2.10.1 Pavement Structural Adequacy

While not attempting to generalize the results from any one project, a report by Willis (1947) on the satisfactory performance of a pavement built in 1938 in South Carolina for a decade is significant, in that the standards of the construction procedures and equipment used then were by comparison to todays standards very low. From a study of airfield pavements in the United States, Redus (1958) has also determined that CTB is structurally adequate and performs reasonably well, even under the much higher loads imposed by aircraft. An interesting finding of that study, was the continuing satisfactory performance of some pavements after over 10 years in service, although retrieved samples of CTB base course material failed to meet wet-dry and freeze-thaw tests.

In a paper on the evaluation of CTB test-sections, highways and airfield pavements, Mitchell and Freitag (1959) contended that CTB pavements perform as well as, and in some cases better than, equivalent conventional flexible pavements. In an ensuing discussion, Felt (1960) even went further, postulating that present CTB pavements are even more superior, since much better construction methods are now used. Also in his opinion, an unfair comparison was made between the CTB and theoretical flexible pavements. The latter were designed by the CBR method using the cumulative ESALs on the CTB up to a certain point, and the traffic carried thereafter completely ignored —an added advantage for the CTB.

Zube *et al*. (1969), Wang *et al*. (1972), Shields *et al*. (1975), and Tayabji *et al*. (1982) are a some of the workers who ha[illegible] presented evidence on the structural ability of CTB [illegible]ents and their adequate performance under varying conditions of soil, climate and load. However, it should be noted that only those pavements with the proper mix design parameters and constructed with the proper methods will be structurally and functionally adequate. Lack of any of the above will result in structural deterioration of the pavement, with base cracking as the most probable first cause of failure.

2.10.2 Cracking

Cracking is an inherent problem of CTB pavements. Almost invariably, cracking is observed in all CTB layers, and is very often accompanied by reflective cracking of the pavement surfacing. The initiation and propagation of cracks is generally considered to be the result of stresses caused by shrinkage and temperature changes, traffic loading, and subgrade failures.

According to Dunlop *et al*. (1972), shrinkage may take place through the process of self-dessication, where moisture for cement hydration is drawn from the clay particles in a CTB mixture causing them to shrink; or through the external loss of moisture from the cement gel, called drying shrinkage; or both. The ambient temperature by enhancing evaporation, greatly influences both types shrinkage and consequently the shrinkage stresses developed. Lister (1972a) points out that thermal stresses may also ensue as a result of the [illegible]ing action of an underlying layer to changes in the length of a CTB layer due to temperature variations; and a temperature gradient through the CTB layer may result in warping stresses which increase when the layer is further restrained by its self-weight and the weight of the overlying materials. These shrinkage and thermal stresses developed, are primarily responsible for the characteristic regular pattern of transverse cracking in CTB pavements which, as observed by Redus (1958), may take place in pavements never opened to traffic.

Traffic-induced horizontal tensile stresses at the CTB-soil interface greater than the tensile strength of the material may cause cracking in CTB pavements. According to Shields *et al.* (1975), such overstressing, particularly at the early age before application of the usual protective bituminous toplifts, is responsible for most of the longitudinal and often vertical cracks observed in CTB pavements in the direction of the wheelpaths. Pretorius and Monismith (1972) had earlier predicted similar load-associated cracking in CTB, often preceding the occurrence of transverse shrinkage cracks, but attributed it more to fatigue loading than to overstressing. Yamanouchi (1973) also reports that, repeated loading results in the development of hairline cracks in CTB, which decreases the modulus of elasticity and the resistance to bending and, consequently, increases deflections.

Repeated traffic loading over such already cracked pavements with the combination of transverse shrinkage and traffic-induced longitudinal cracks, results in edge and corner loading situations, and tensile stresses and stress reversals, which according to Pretorius and Monismith (1972) and Fossberg *et al*. (1972b), give rise to the typical double and single "ladder" cracking found in the wheelpaths of CTB pavements.

Of course any action which reduces the support a subgrade gives to a CTB layer, and thereby causes it to withstand greater load-induced stresses than usual, will result in further cracking. According to George (1973), the "drag" of a subgrade due to its movement and cracking may also be responsible for the longitudinal cracking of a CTB pavement; and the eventual reflection of subgrade shrinkage cracks through the pavement surface also contributes to cracking.

2.10.3 Effects of Cracking

As noted by Teng and Fulton (1974), cracks that appear and are retained as fine hairline cracks in a CTB layer will not reflect through the surfacing of a pavement. Such cracks, as also pointed out by Marwick and Keep (1942), Maclean and Robinson (1953) and Norling (1973), are therefore generally not detrimental to the performance of a pavement. In most cases, however, the cracks are much wider and do reflect through to the surface of a pavement. Although unsightly, they initially do not affect the ride quality of the pavement, but may cause weakening of the subgrade by allowing ingress of water. Fossberg *et al*.

(1972b) have determined that, directly under a load, vertical deflections are increased by about 20 percent and subgrade stresses by as much as 50 percent, in a cracked section of a CTB pavement in comparison to an uncracked section. Otte (1979) observes that these stresses and strains are quite significant and may cause the subsequent localized deformation of the subgrade, and could result in a loss in ride quality, even if after an indeterminate period.

2.10.4 Minimizing Cracking

Basically a reduction of the causative stresses will lessen cracking in CTB pavements. Accordingly, shrinkage needs to be curtailed as its associated stresses are responsible for most of CTB cracking. As George (1974) reports, thermal stresses are largely inconsequential in comparison to shrinkage stresses. Furthermore, Lister (1972a) has shown that a bituminous cover of any type with a thickness above 50 mm, decreases the temperature gradient in the base course and thereby reduces thermal stresses in CTB pavements. Traffic stresses will not be detrimental if pavements are properly designed with adequate thicknesses and are not over-loaded. The quick repair of cracked pavements will also prevent their further aggravation by traffic, the ingress of water to the subgrade and the consequent deterioration of the CTB pavement.

From extensive studies on shrinkage and cracking, George (1968a; 1968b; 1969; 1971; 1973) has proposed taking the following into consideration to minimize shrinkage and cracking in CTB pavements:

a) Since moisture inadequacy is the cause of shrinkage, adequate curing should be provided, especially during the first 2 to 4 days. By enhancing the viscous properties of CTB, curing with water also decreases cracking. However, very long curing periods should be avoided as the increased amount of bonded water causes more shrinkage when lost later.

b) Molding moisture contents above the optimum, with the associated higher amounts of bonded water, give more shrinkage susceptible CTB.

c) An optimum cement content equal to, or slightly above that required to meet the PCA criteria is associated with minimum shrinkage.

d) Increasing the amount of the progressively finer clay particles in CTB (e.g. montmorillonite as against kaolinite), as well as an increase in the total amount of clays, increases total shrinkage. On the other hand, large aggregates, 25 mm and above, increases cracking by mobilizing higher stresses.

e) Improved compaction by increasing density and strength, and an increase in base thickness with the resultant increase in base stiffness, decreases cracking in CTB pavements.

Additional methods advocated include the placement of CTB on rough subgrades. George (1974) suggests that the high coefficient of friction developed between the CTB and the subgrade enhances an even distribution of shrinkage and other stresses and reduces cracking. Consequently, the mixed-in-place method of construction is recommended for reducing cracking. Additives such as lime for clayey CTB, fly ash for sandy CTB, sulphates of calcium, magnesium and sodium (George, 1968b); granular sodium chloride for montmorillonitic CTB (Wang and Kremmydas, 1970); expansive cements (Barksdale and Vergnolle, 1968); and sugar (George, 1971), to mention a few, have been shown to reduce shrinkage and/or cracking. In all cases, however, they produced better results in CTB of well-graded soils than those of uniformly graded soils.

Norling (1973) echoes some of the measures mentioned above, and emphasizes the use of surfacing materials of the highest strength and quality, as well as delaying their placement for as long as possible until after much of the CTB cracking has taken place, to minimize reflective cracking. Special treatments such as the use of a bituminous surface treatment between a CTB and its asphaltic concrete surfacing, upside-down design,[10] and asphalt-ground rubber treatments are also suggested by Norling for reducing or delaying reflective cracking.

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[10] In the upside-down design a layer of untreated granular material is placed between the CTB and the bituminous surface.

Wang (1973) cautions, however, against the use of measures for reducing shrinkage and/or cracking, which concurrently decrease the strength (especially tensile strength) of CTB and thereby have little or no effect in the final analysis. Also worth mentioning, is a finding by workers including Yamanouchi and Ishido (1963) and Teng and Fulton (1974), of the unappreciable difference between the cracking intensity of pavements opened to traffic immediately after construction, and those cured for at least 7 days before opened to traffic. In fact, they recommend, that from the point of view of economy of construction and as far as cracking is concerned, the former procedure is considerably better for CTB pavements.

2.11 Summary

It is apparent from the extensive literature review that CTB is a unique pavement construction material. The addition of cement to substandard soils or aggregates to give a material capable of withstanding the repeated action of traffic under varying environmental conditions per se represents some success. Indications are that, using the methods described, the determination of the proper amounts of cement and water to add to soils is not very difficult; and adequate construction and curing methods will reflect favorably on the properties of the soil and give a much better cement-treated base material. In view of the various measures suggested for reducing shrinkage and cracking of

CTB pavements, it appears those major problems can to a large extent be minimized. Thus, on the whole, CTB pavements hold a lot of promise, and the possibility of constructing them in place of more conventional pavements should often be investigated. It is also important that research work be directed at unearthing all the advantages of the material and at the same time provide solutions to the problems encountered with the use of CTB.

Table 2.1 Compressive Strength of Cement-Treated Base [1]

Soil Type	Compressive Strength, MPa [2]	
	7-day	28-day
Sandy and Gravelly Soils:		
AASHTO Groups A-1, A-2, A-3	2.1-2.8	2.8-6.9
Silty Soils:		
AASHTO Groups A-4, A-5	1.7-3.5	2.1-6.2
Clayey Soils:		
AASHTO Groups A-6, A-7	1.4-2.8	1.7-4.1

[1] The cement contents used for the soils are those which satisfy minimum requirements as determined by the PCA wet-dry (ASTM D599; AASHTO T135) and freeze-thaw (ASTM D560; AASHTO T136) tests.
[2] 1 MPa = 145 psi.

Table 2.2 Gradation and Consistency Characteristics of Soils

Sieve Size	Percentage Passing			
	Soil 1	Soil 2	Soil 3	Soil 4
3/4 in.	100		100	
No.4	75	100	85	
No.10	55	80	72	100
No.40	27	58	57	99
No.200	3	19	27	92
Liquid Limit	16	17	28	26
Plasticity Index	NP[1]	NP	15	7
AASHTO Soil Groups	A-1-b	A-2-4	A-6	A-4

[1] NP = nonplastic

Table 2.3 . Compressive Strength, Modulus of Elasticity and Poisson's Ratio for 28-Day Cured Cement-Treated Base

Soil	Cement/Soil Ratio	Ultimate Compressive Strength, MPa[1]	Modulus of Elasticity, MPa	Poisson's Ratio
A	1:6	8.77	13 590	0.120
	1:8	5.80	11 010	0.138
	1:10	4.18	8 930	0.142
C	1:6	11.77	14 030	0.125
	1:8	7.86	11 180	0.136
	1:10	5.65	9 120	0.136
D	1:6	7.12	9 120	0.111
	1:8	4.92	8 190	0.129
	1:10	3.97	6 520	0.095
F	1:6	5.25	4 510	0.090
	1:8	3.83	3 780	0.070
	1:10	2.94	2 910	0.053

Characteristics of the Synthetic Soils

Soil	Composition, %		Liquid Limit	Plasticity Index
	Sand	Clay		
A	100	0		NP[2]
C	75	25	16.8	NP
D	50	50	25.0	8.93
F	0	100	38.5	17.52

[1] 1 MPa = 145 psi
[2] NP = nonplastic.

Table 2.4 Typical Values of the Resilient Parameters of Cement-Treated Bases

Workers	Year	Soil	Cement Content, Used, %	Moisture Content, %	Dry Density, kg/m³ [1]	Curing Time, days	No. of Load Repititions	Parameter[3]	Value, x 10^3 MPa [2]
Shen and	1966	A-6 Silty Clay	13	14-22	1568-1712	7	up to	M_R	1.38 - 62.1
Mitchell				approx.	approx.		25 000	M_{RF}	2.35 - 3.04
		A-2-4 Sand	7	7-12	1920-2112	7		M_R	55.2 - 166
				approx.	approx.			M_R	10.4 - 18.6
Morgan and	1970	Medium poorly	2	10	1808	7	10^6	M_R	0.0897 - 0.414
Williams		graded sand						ν_R	0.0 - 0.3
Fossberg,	1972a	Silt-gravel	5.5	—	2160-2208	10 to 250	—	M_R	8.28 - 15.9
Mitchell and						approx.		ν_R	0.10 - 0.20
Monismith									
Yamanouchi	1973	Volcanic	4	21.0	1632	up to	10^7	M_R	0.393 - 0.538
		sandy soil				7	10^5		0.0966 - 0.393
Chang,	1982	A-1-a Oceanlake	3	7.0	2243	up to	up to	M_R	17.3 - 34.5
Hicks and Vinson		marine basalt				90	10^6		
		A-1-a Eckman Creek marine basalt	6	13.8	1842				13.8 - 20.7
		A-1-b Sandstone	6	14.0	1810				10.4 - 18.6
		A-3 Sand	9	9.0	1856				4.83 - 9.66

[1] 1 kg/m³ = 0.0624 lb/ft³.
[2] 1 MPa = 145 psi.
[3] M_R = Resilient modulus in compression.
M_{RF} = Resilient modulus in flexure.
ν_R = Resilient strain ratio.

Table 2.5 Aggregate Gradings and Properties

Sieve Size	Percentage Passing: Marine Basalts: Ocean Lake	Eckman Creek	Tyee Sandstone	Dredge Spoil Sand
19.0 mm (3/4 in.)	100	100	100	100
12.5 mm (1/2 in.)	100	100	93	96
9.5 mm (3/8 in.)	86	86	89	93
6.3 mm (1/4 in.)	67	67	81	87
4.75 mm (No. 4)	55	55	75	85
2.00 mm (No. 10)	40	40	60	77
425 μm (No. 40)	18	18	39	62
75 μm (No. 200)	9	9	15	1
Liquid Limit	23	20	27	
Plastic Limit	5.3	2.9	4.2	NP[1]
AASHTO	A-1-a	A-1-a	A-1-b	A-3

Table 2.6 Typical CBR Values For Cement-Treated Base

Soil Type	CBR [1]
Well graded gravel-sand-clay; sands or gravels	More than 600
Silty sand; sandy clays; sand and gravel	600
Silty-sandy clays; poorly graded sands	200
Silts; silty clays; very poorly graded soils	Up to 100
Heavy clays; organic and sulphate rich soils	Up to 50

[1] The CBR values are for CTB with a minimum unconfined compressive strength of 1.72 MPa (250 psi).

Table 2.7 Coefficient of Thermal Expansion of Some Cement-Treated Base Mixtures

Workers	Year	Soil	Cement Content, %	Coefficient of Thermal Expansion, mm/mm/C x 10^{-6}
Mehra and Uppal	1951	Sandy loam	2.5 - 10	2.50 - 3.22
		Silty loam	2.5 - 10	2.78 - 3.11
		Silty clay loam	2.5 - 10	2.17 - 3.39
		Loam	2.5 - 10	2.56 - 3.50
Catton	1952	Sandy soil	8	3.83
		Silty soil	14	3.44
		Clayey soil	12	3.17

Table 2.8 Recommended Thicknesses of Cement-Treated Base as Base Course Material and Corresponding Recommended Thicknesses of Granular-Type Stabilized Base Courses

AASHTO Classification of Subgrade Soil	Cement-Treated Base Course Thickness (mm)[1]	Cement-Modified Granular Soil Type Base Course (mm)[1]
A-1-a	0	0
A-1-b	125	125
A-3	125	125
A-2-4	125	125
A-2-5	125	150
A-2-6	125	150
A-2-7	125	150
A-4	150	200
A-5	150	200
A-6	150	200
A-7	150	200

[1] The CTB thicknesses were recommended for highway pavements with average traffic that did not exceed 100 trucks per day with load ranging from 1 800-kg gross load to 8 160-kg axle load, or a total of 1 000 vehicles per day including the afore-mentioned truck traffic. Base courses of the thicknesses given were expected to be supplemented by subbase in the cases where the loadings exceeded those for which they were intended according to the recommendations.

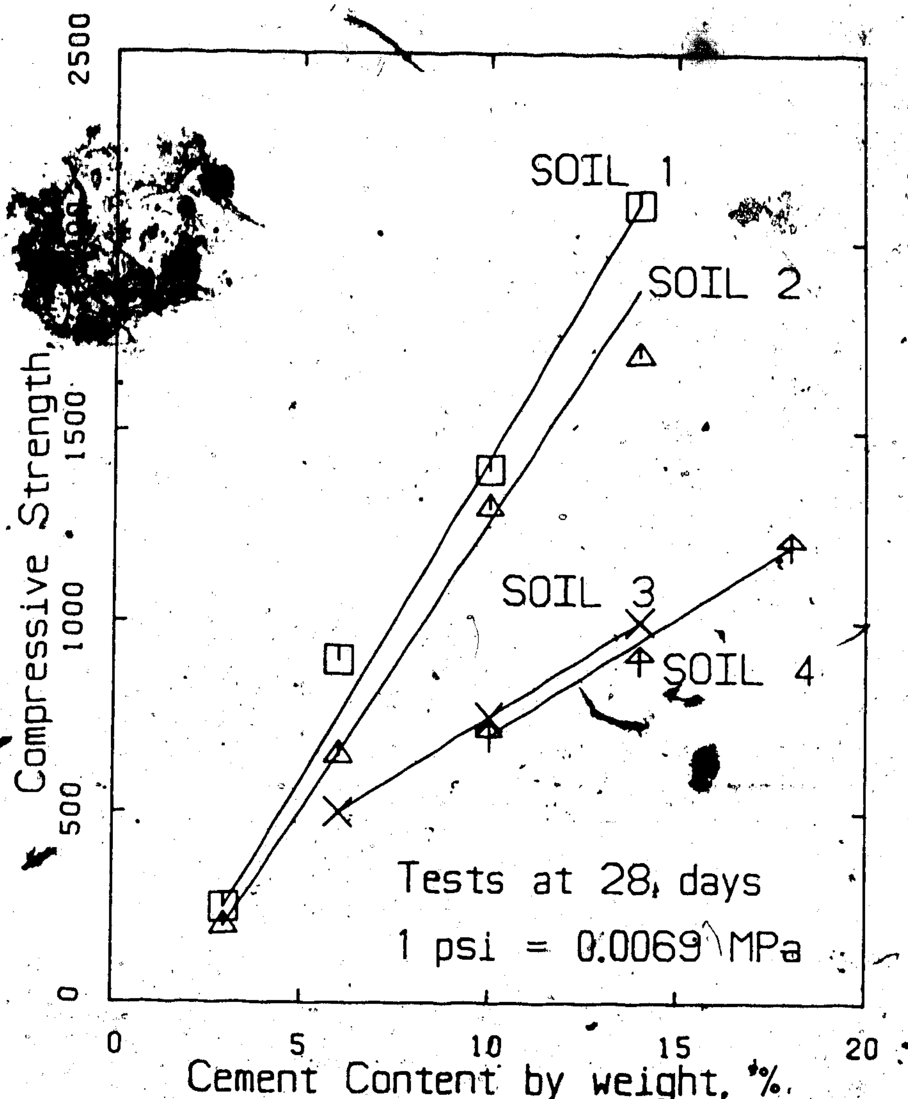

Figure 2.1 Compressive Strength, Soil Type and Cement Content

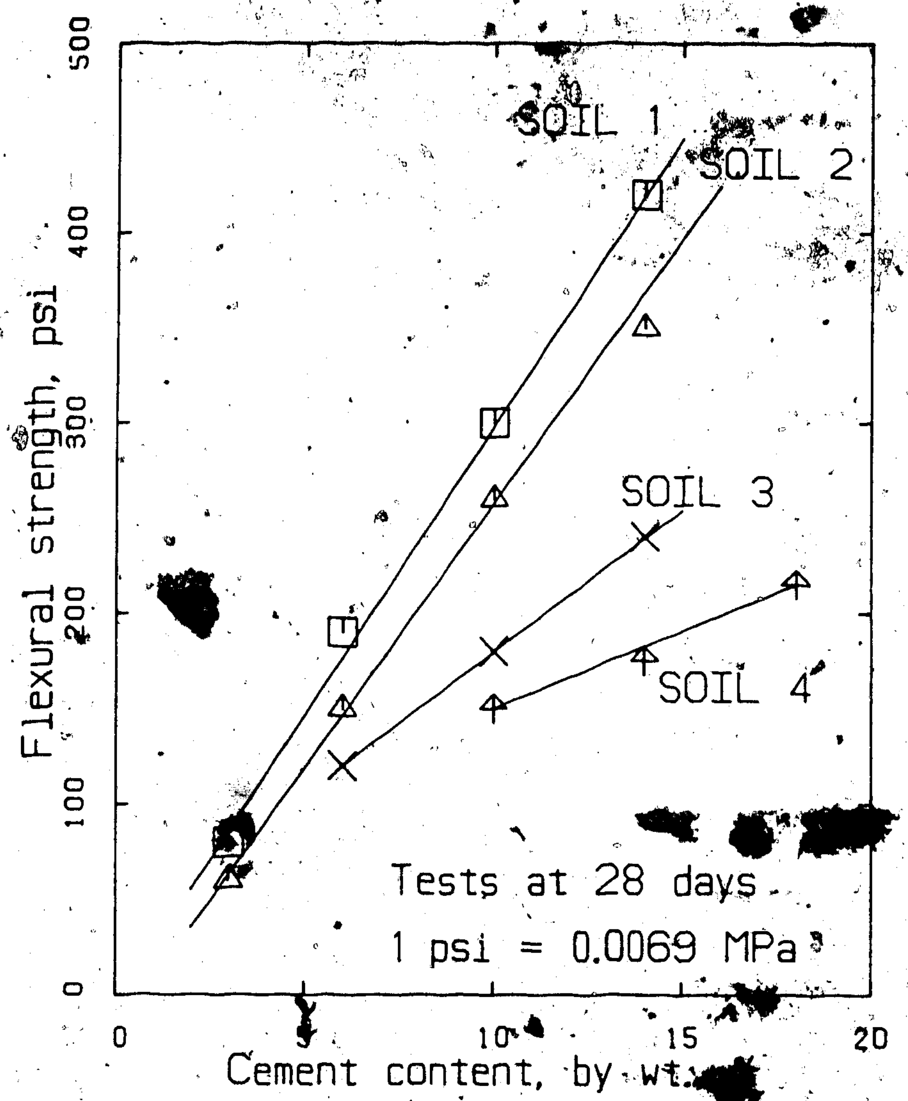

Figure 2.2 Flexural Strength, Soil Type and Cement Content

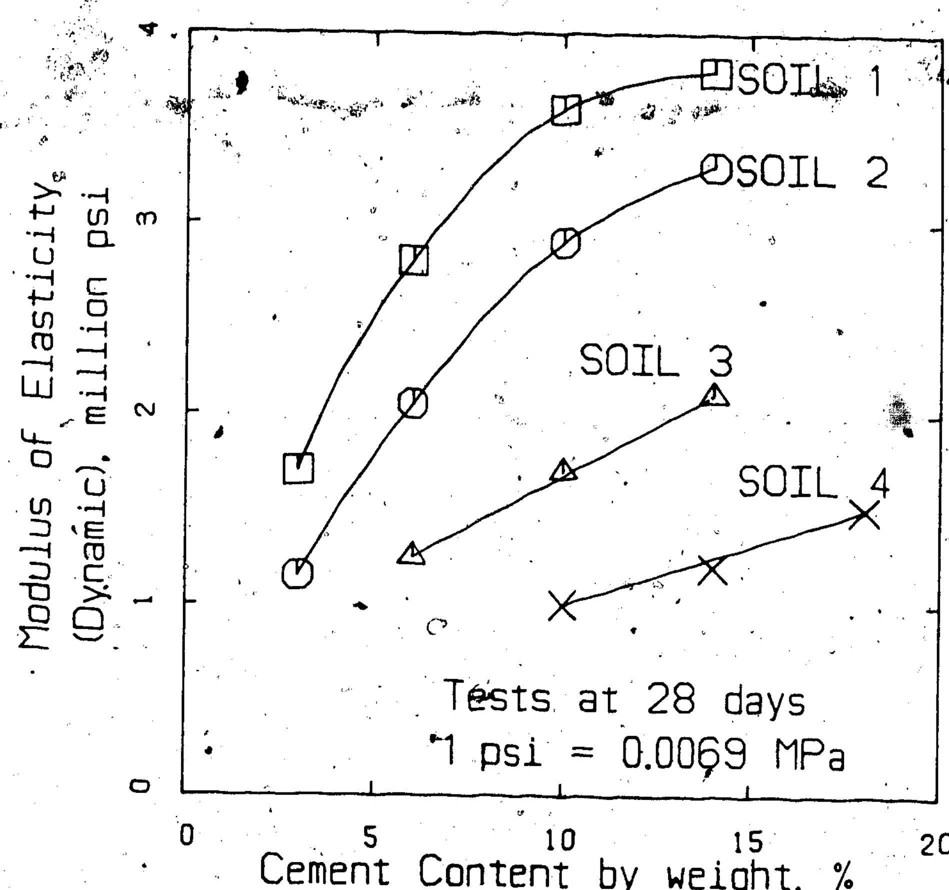

Figure 2.3 Modulus of Elasticity (Dynamic), Soil Type and Cement Content

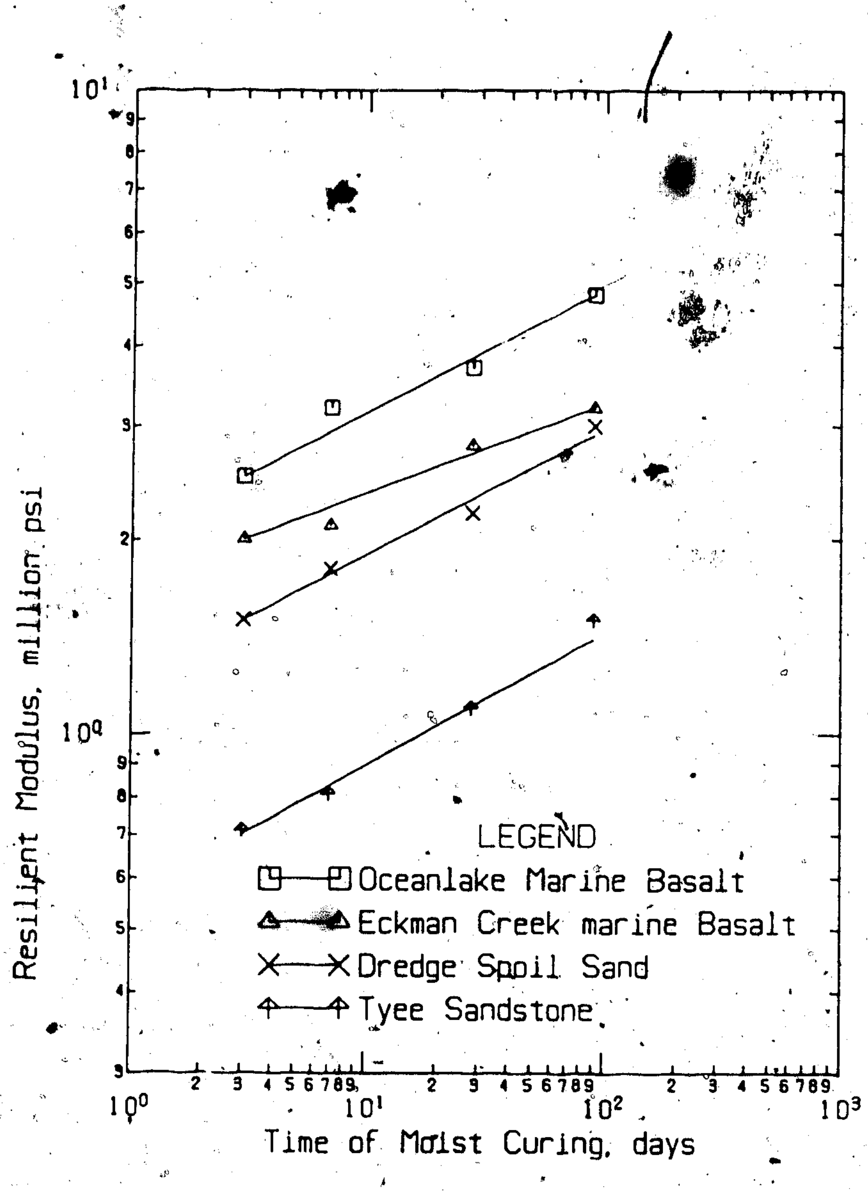

Figure 2.4 Resilient Modulus versus Time of Moist Curing

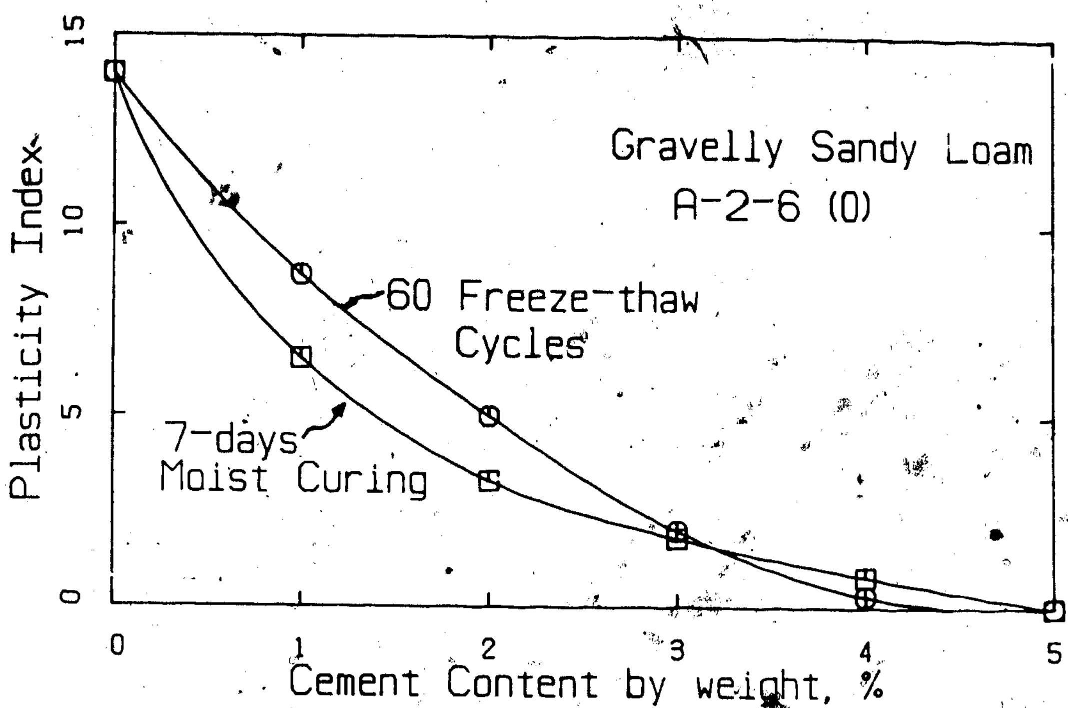

Figure 2.5 Reduction in Plasticity Index with Increase in Cement Content

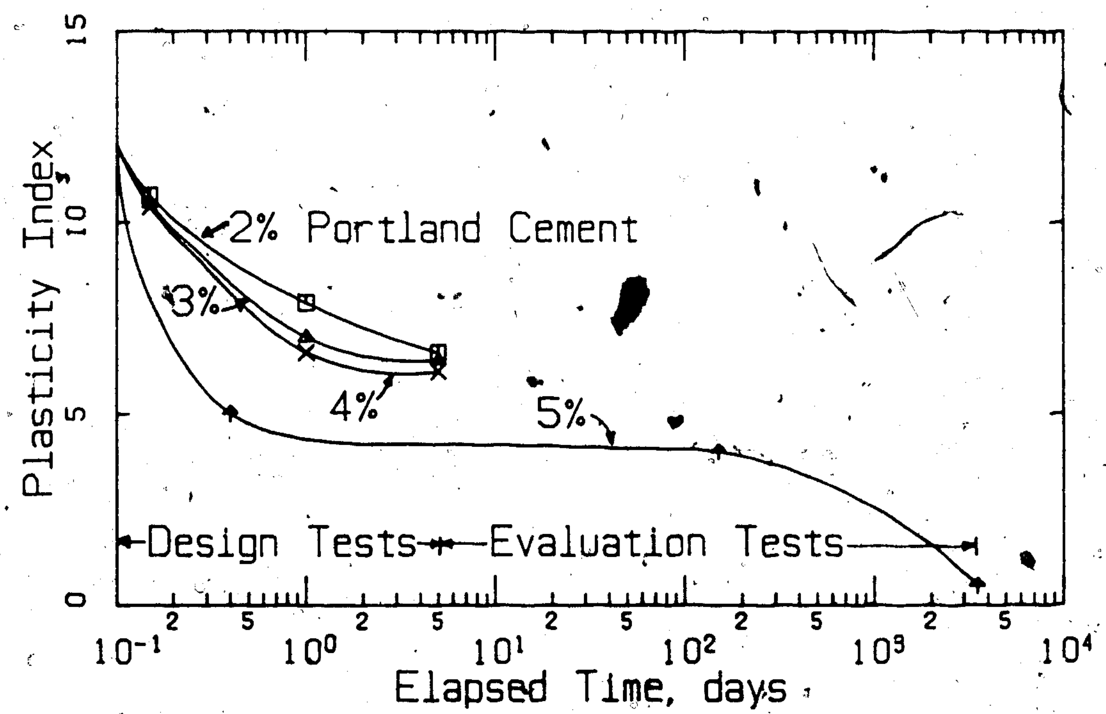

Figure 2.6 Reduction in Plasticity Index with Time

3. CEMENT-TREATED BASE MIX DESIGN IN ALBERTA

3.1 Introduction

The cement-treated base mix design procedure of Alberta Transportation used in Alberta is primarily a modification of the standard AASHTO and ASTM test methods of design. Using charts developed by the Portland Cement Association for a shortcut mix design method for sandy soils, a cement content is estimated which forms the basis of the cement contents at which the mix design tests are conducted.

T[illegible] design tests, comprising a freeze-thaw test, a wet-dry test and unconfined compressive strength tests, are conducted on specimens molded at an optimum moisture content and maximum dry density. The resulting cement content recommended for construction represents the minimum and most economic cement content which will give cement-treated base or soil-cement with the required strength and durability for use in pavements.

The basic criteria for selecting this minimum cement content in Alberta are, a loss of material by weight of not more than 3 percent after brushing of the freeze-thaw and wet-dry tested specimens, and a minimum 7-day compressive strength of 2.07 MPa (300 psi).

Generally, this procedure has established cement contents of between 5 to 10 percent as those adequate for the sandy soils used in Alberta for cement-treated base or soil-cement construction. However, a cement content of 8 percent is in most cases considered to be maximum a above which the material will not be a viable economic replacement for conventional roadbase material.

By the summer of 1986, as many as 740 separate mix designs had been performed by the central laboratory of Alberta Transportation, since the inception of large scale cement-treated base or soil-cement construction in 1959. This chapter, as part of a synthesis on cement-treated base practice in Alberta, deals with the mix design aspect of cement-treated base construction in the province.

3.2 Description of Materials Used

Before giving the details of the mix design procedure used in Alberta, the characteristics of the soils, cement and water usually used for CTB or soil-cement in the province are described in this section.

Soils

The aggregates used for CTB in Alberta are generally sands or sandy type soils, which under the Unified Soil Classification System fall into the category of SP and SM sands, with a further requirement for the [illegible] sands that they have liquid limits, W_l, less [illegible] and plasticity indexes, I_p, not more than 6. The corresponding soils by the AASHTO classification system are the A-3, A-2-4 and A-2-5 soils. Gradation limits for these soils as specified by Alberta Transportation are as follows:

Passing 40 000 μm sieve	100%
Passing 10 000 μm sieve	85-100%
Passing 1 250 μm sieve	40-100%
Passing 315 μm sieve	17-100%
Passing 80 μm sieve	6-30%

As illustrated by the gradation limits in Figure 3.1, this represents a wide range of soils permitted for use in Alberta. An example gradation is shown as the fine SAS A design line.

Most of the soils have no more than a trace of plasticity, and are quite uniform with coefficients of uniformity, Cu[1], ranging from 3 to 5. Deleterious materials mostly found in the soils include top soil fiber, pieces of coal and lumps of clay and silt. In small quantities they are not harmful, but where quantities are appreciable the soils are used only if the deleterious materials can be disposed of by screening.

It is worth mentioning that attributes considered highly desirable for the soils are a gradation with 80 percent or more material passing the 5 000 μm sieve, and 6 to 30 percent material passing the 80 μm sieve size.

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[1] Cu is the ratio D_{60}/D_{10}, where D_{60} is the particle diameter with 60 percent material finer and D_{10} is the particle diameter with 10 percent material finer.

Cement

Normal Portland Cement (CSA Type 10) is the usual type of cement used for CTB construction in Alberta. Although existing specifications do permit the use of the other specialized types of cement, there has virtually been no need for their use in the province. This is quite an advantage as those types introduce an extra cost item when used, as a result of their specialized nature.

Water

The water used for soil-cement construction in Alberta is mainly obtained from nearby streams, sluices, ponds and other such supplies, which are relatively abundant in this province in the prairie region of Canada. Where there is doubt as to the quality of a source, laboratory tests are conducted to determine the suitability of the water for CTB construction. In Table 3.1, are given Alberta Transportation guidelines on the maximum allowable limits for some of the more common deleterious materials found in such supplies.

3.3 Types of Mix Design

The cement-treated base (CTB) mix designs conducted in Alberta can be put into two categories. A majority of the designs have been carried out to determine the mix design parameters for upgrading soil materials for particular road construction projects with already defined locations and limits. Also, in the effort to locate aggregates for such projects and as part of ongoing prospecting activities of Alberta Transportation, sources are sometimes found whose aggregates may not be necessarily earmarked for a specific project at the time of discovery. In some cases, however, mix designs are still undertaken for such aggregates with the intention of using them for projects which may be located in their vicinity in the future.

In Alberta, the aggregates considered for CTB construction are all from aggregate prospect pits, as the plant-mix method of production is the only method used in the province. With a source located, preliminary field investigations are undertaken to ascertain the suitability of the aggregates for use, with respect to such factors as haul-distance, in-situ moisture content, compactibility, overburden depth and the accessibility of the aggregates. If the aggregates are found to be in sufficient quantities, of the right quality, and accessible, samples are obtained from test pits and sent to the laboratory for detailed testing to determine the specific mix design parameters for construction.

3.4 Preliminary Aggregate Testing

A visual inspection of individual test-pit samples is the first step in the laboratory towards the determination of the suitability of particular soils for soil-cement design by Alberta Transportation. This inspection involves a description of the grading, texture and shape of grains of the aggregates; the determination of the presence of deleterious materials; and an estimation of I_p, all from a visual appraisal of the soils. A routine sieve analysis of a split 300- to 400-g sample is then conducted for each of the individual bulk test-pit samples to determine their gradation.

Depending on the results obtained so far, a number of aggregate tests may still be required to further ascertain the suitability of the soils. These indicative tests summarized in Appendix A, will most often clarify any abnormal results obtained during the actual design tests. The tests are made up of a 'color test' for determining the presence of organic impurities; a 'silt-band test' which indicates the quantity of fine materials; a 'coal test' for determining the amount of coal and other such lightweight materials; and an 'Atterberg limits test' for evaluating the relative amounts of clays in the soils. (The color and coal tests are similar to the standard ASTM C40 and C123 tests respectively).

3.5 Selecting the Appropriate Soils for Design

The selection of soils truly representative of a given source is paramount to the success of a mix design procedure. Notwithstanding the preliminary test results showing the suitability of aggregates from a particular prospect pit for CTB construction, and the proper application of the parameters obtained from a mix design, the final results of the construction of a soil-cement pavement could be disastrous if in the first instance the right soils were not chosen for design.

To this end, bearing in mind the results of the preliminary tests and with reference to the test pit plan, a decision is made whether to blend all the materials from the different test pits and conduct one design, or to separate them into two or more groups each requiring a separate design. The gradings of the soil samples are the main criteria for this grouping or distinction into one or more bulk samples.

Where very uniform sands with Cu less than 3 are encountered, efforts are immediately made to locate soils close to the original source with a gradation that will increase the Cu when the two soils are blended. In general, uniform sands require more cement to attain the necessary strength and durability by reason of a reduced total area of contact between grains. Also, as evidenced in the province, although some uniform soils may have cement requirements economically acceptable, they often prove to be difficult to compact in the field irrespective of their moisture content at time of compaction.

3.6 Mix Design

In Alberta, the mix design tests to determine the most economic cement content comprises of a freeze-thaw test, a wet-dry test and unconfined compressive strength tests on specimens molded at the optimum moisture content and maximum density. Following are the details of the mix design procedure.

3.6.1 Preparation of Aggregates

As indicated in the preceding section, samples from the test pits of a source may all be combined for one design or separated into two or more groups with separate designs. Whichever the case, the preparation of the representative sample or samples for a design is the same. An average of the individual gradations determined for each test pit sample during the preliminary testing is calculated to give what is considered as the design gradation. To get a representative bulk sample, the individual samples are thoroughly blended in such quantities as to give a combined sample with a gradation approximately equal to the calculated design gradation. A washed sieve analysis of a split 300- to 400-g sample of the combined sample is used to determine it's gradation.

In the case of very uniform sands with Cu less than 3 to which fines are added to increase the Cu, using an aggregate proportioning chart and keeping in mind the specified gradation limits, the proportion of coarse to fine aggregates which will give the best result is determined by trial and error. This proportion is used for the design and for construction.

As a result of the use of charts developed by the Portland Cement Association, PCA (1971) for a shortcut design method, there is a further distinction between aggregates with material retained on the 5 000 μm sieve and aggregates with all material passing that sieve size. In all about 100 kg of soil material is needed for the design tests.

3.6.2 Estimation of Cement Content

The above-mentioned charts developed by the PCA through the work of Leadabrand and Norling (1953; 1956) and Norling and Packard (1958) for a shortcut design method for sandy soils, are used to estimate the cement content for the prepared aggregates for further investigation. As a first step the amount of aggregates smaller than 50 μm is determined by extrapolation from the aggregate gradation chart.

For aggregates with all material passing the 5 000 μm sieve size, the extrapolated value of the material smaller than 50 μm and the percentage of material retained between the 5 000 μm and 315 μm sieve siezes are used to determine an average maximum density from Figure 3.2. Using this maximum density and the percentage of material smaller than 50 μm an estimate of the cement content is read off Figure 3.3. As an example, if the material extrapolated from the grading curve to be smaller than 50 μm is 39 percent and the material retained between the 5 000 and 315 μm sieves is 29 percent, then from Figure 3.2 the average maximum density will approximately be 1 890 kg/m³. With this density and the 39 percent material smaller than 50 μm, the estimated cement content from Figure 3.3 is 8 percent.

In the case of aggregates with material retained on the 5 000 μm sieve size, again the extrapolated value of material smaller than 50 μm, and the material retained on the 2 000 μm sieve size are used to estimate an average maximum density from Figure 3.4. The material smaller than 50 μm, the material retained on the 5 000 μm sieve size and the average maximum density are together used to find an estimate of the cement content from Figure 3.5. As an example, if the material smaller than 50 μm by extrapolation is 32 percent and the material retained on the 2 000 μm sieve size is 23 percent, then from Figure 3.4 the average maximum density is approximately 1 950 kg/m³. Suppose the material retained on the 5 000 μm sieve size is 18 percent, then from Figure 3.5 the estimated cement content is 6 percent [12] [13].

The estimated cement content in both cases forms the basis of the cement contents at which a moisture-density relations test and other design tests are conducted to

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[12] As indicated in Figures 3.5, the material retained on the 5 000 μm is not expected to be greater than 40 percent. Any material retained on the 20 000 μm sieve size is converted to an equivalent weight of material retained between the 20 000 μm and 5 000 μm sieve sizes.

[13] Figures 3.2 through 3.5 are modified versions of the original PCA (1971) graphs developed after extensive laboratory correlations, used by Alberta Transportation.

arrive at the most economic cement content for use in construction.

3.6.3 Optimum Moisture Content and Maximum Density Determination

Next in the Alberta Transportation mix design procedure is a five-point moisture-density relations test, similar to the conventional ASTM D558 and AASHTO T134 moisture-density relations tests. The test is performed on soils with a cement content one (1) percentage point below the cement content estimated as described in the preceeding section, to determine the optimum moisture content and maximum dry density of the soils with cement added, to use in the design tests.

A standard mold with a 2.5-kg rammer dropped from a height of 304.8 mm is used for the test. The material is placed in three lifts compacted at twenty-five (25) blows per lift, to give a specimen approximately 101.6 mm (4 in.) in diameter and 116.8 mm (4.6 in.) high, in accordance with PCA practice (PCA, 1971). As a precaution against the formation of compaction planes, each lift face is scarified after compaction before the next lift is placed on. in the design tests. An example of a curve obtained from a moisture-density relations test is shown in Figure B.2 of Appendix B.

Typical laboratory maximum dry densities for CTB mixtures in the province range from about 1 600 to 2 160 kg/m^3 (100 to 135 lb/ft^3 and are obtained at moisture contents between approximately 9 to 15 percent by weight of dry soil. Although the optimum moisture content and maximum [illegible] so determined are used for molding CTB specimens for the other design [illegible] serve as a guide during construction, the actual values used for construction are determined from moisture-density relations tests conducted on the CTB mixture produced in the field.

As part of the design test, the laboratory specimens used for the moisture-density test, called "7-day control proctor" specimens, are cured for seven days and broken to determine their compressive strengths in a manner described in the subsequent section.

3.6.4 Design Tests

In the province, the design tests of freeze-thaw, wet-dry and unconfined compressive strength tests are performed on specimens molded at three (3) cement contents. These are the cement content used for the moisture-density relations test, (i.e. the estimated cement content less 1 percent), and the cement contents plus-and-minus 2 percent away[14].

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[14] In recent years, additional freeze-thaw test specimens have been molded at the cement contents plus-and-minus 1 percent away from the cement content used for the moisture-density relations test, ostensibly for higher accuracy. It is doubtful from an inspection of design charts whether this actually affects the final reccomended cement content.

Preparation of Test Specimens

At each of the three (3) cement contents under investigation, four (4) sets of specimens are formed for a 7-day freeze-thaw test, a 7-day wet-dry test, a 7-day compressive strength test and a 28-day compressive strength test. The specimens are molded at the optimum moisture content and maximum density previously determined, using the same compaction equipment and method used for the moisture-density test specimens. Three (3) specimens are formed within each to give a total of thirty-six specimens.

As the names of the tests suggest, apart from the 28-day compressive strength test specimens which are cured for twenty-eight (28) days, all the other specimens are cured for seven (7) days in a moist room at a temperature of 21 C and a relative humidity between 95 to 100 percent prior to testing. The specimens used for the moisture-density test discussed above are also cured for 7 days under the same conditions. Throughout the tests that follow the specimens are visually inspected frequently for any signs of deterioration.

Unconfined Compressive Strength Tests

After curing for the respective ages the unconfined compressive strength test specimens are removed from the moist curing room and tested. The specimens cured for 7 days called the "7-day control" specimens are tested after that period; the 28 days cured specimens called the "28-day control" specimens after 28 days; and the moisture-density relations test specimens called the "7-day control proctor" specimens after 7 days.

In each case the specimens are soaked in water for a period of 4 hours, and in a saturated surface dry state they are capped with a mixture of 7 to 8 parts of sulphur to 1 part fine clay, and broken for their compressive strengths. Where the length to diameter ratio, L/D, of a specimen after curing is other than 2.00, a correction factor based on ASTM C42 specifications and recommendations of the U.S. Bureau of Reclamation (1966) is applied to the measured compressive strength. For the specimens used in Alberta with initial dimensions of 101.6 mm (4 in.) diameter and a length of 116.8 mm (4.6 in.) this factor is usually approximately 0.9.

Freeze-Thaw Test

The freeze-thaw test is similar to the ASTM D560 and AASHTO T136 freeze-thaw tests. Following 7 days of curing, the specimens are removed from the moist curing room and placed on water saturated felt pads in appropriate containers. Testing commences with placement of the specimens in a freezer or freezing room with a temperature of -23 C at all times for 24 hours. After this 24-hour freezing period, the specimens are transferred into a moist room with a temperature of 21 C and a relative humidity between 95 and 100 percent, where they are allowed to thaw for 24 hours. During this thawing period, water is made available to the specimens through the saturated felt pads for free capillary absorption.

The 48-hour period between placement of the specimens in the freezer to their removal from the moist room is considered as one cycle. Turning the specimens end for end at the completion of each cycle, this procedure is repeated for 12 cycles. At the end of the last cycle, the specimens are taken through a brushing procedure which involves brushing the total surface area of each specimen twice with a wire brush applying a pressure of approximately 5 kg. The specimens are then measured for their dimensions and weight, and the percentage of material lost calculated from the initial and final dry weights. To complete the test, the freeze-thaw specimens are broken in the manner described in the preceding section, but without soaking in water, for their compressive strengths.

Wet-Dry Test

The wet-dry test is also very similar to the ASTM D559 and AASHTO T135 standard wet-dry tests. At the end of 7 days curing, the wet-dry test specimens are removed from the moist room and immersed in water at room temperature for 5 hours, ensuring complete submergence throughout this period. The specimens are then transferred into an oven and dried at a temperature of 71 C for 42 hours. The specimens are then usually allowed to cool down for one hour to complete one wet-dry cycle of 48 hours.

The procedure is repeated for a total of 12 cycles after which the specimens are brushed in the same way as the freeze-thaw specimens. From the calculated initial and final dry weights the percentage of material lost by each specimen is also similarly determined as for the freeze-thaw specimens. The wet-dry test specimens are not tested for their compressive strength.

3.6.5 Choosing the Design Cement Content

From the freeze-thaw and wet-dry tests, the percent soil-cement loss at each of the three (3) cement contents under investigation, is calculated as the mean of the losses of the three (3) specimens molded at each particular cement content. Similarly, the compressive strengths for the 7-day control, 28-day control and freeze-thaw specimens at each cement content, are determined as the average of the results obtained for each set of three specimens.

The soil-cement losses for the freeze-thaw and wet-dry tests are plotted against cement content; and the compressive strengths of the 7-day control, 28-day control and freeze-thaw specimens also against cement content. These graphs are all plotted with the same abscissa as shown in Figure B.1 of Appendix B, and are primarily the basis for the selection of the cement content. The compressive strengths of the 7-day control proctor specimens are also plotted against the molding moisture content as shown in Figure B.2, with the same abscissa as the moisture-density test curve.

Criteria for Cement Content Selection

The fundamental criteria of Alberta Transportation requires that the freeze-thaw and wet-dry losses are less than 3 percent, and the 7-day compressive strength exceeds 2.07 MPa (300 psi). Using the graphs obtained from the tests similar to those in Figure B.1, a cement content is chosen which basically satisfies these criteria. However, other factors are considered before the final design cement content is selected.

The 28-day control compressive strengths are expected to be well above the 7-day control strengths; and the strengths of the freeze-thaw specimens, which are cured for an initial period of seven days before the freeze-thaw tests, are expected to be higher than the 7-day control strengths, though not necessarily as high as the 28-day control strengths. Also, the compressive strengths should all increase with increasing cement content, and the specimens should show no sign of serious deterioration throughout the tests.

The graphs of the compressive strength of the 7-day control proctor specimens and dry density, versus molding moisture content (Figure B.2), also gives an indication of how the compressive strength of the soil-cement mixture is affected by moisture and density changes.

Design Cement Content

Taking all the above into consideration, a final design cement content by dry weight of aggregate is chosen which is half a percentage point higher than the minimum that just satisfies the fundamental criteria described above. This cement content is what is recommended for construction, and the results of the design tests are presented as in Table B.1. Typical cement contents for the SP and SM sands of Alberta using this design method range between 5 and 10 percent by weight of dry aggregate. A majority however fall between 6 and 8 percent and a cement content of 8 percent considered as the maximum which will allow an economic construction.

In all, even with the exclusion of the time up to the end of the preparation of the test specimens, the mixture design procedure requires a total of 31 working days for the completion of tests, made up of seven (7) days of curing and twelve (12) 48-hour cycles of freeze-thaw and wet-dry tests.

3.7 Differences Between the Alberta Mix Design Procedure and the Standard Methods

In general, the fundamental principles behind the mix design procedure of Alberta Transportation presented in the preceding sections, are the same as those of the standard ASTM and AASHTO test methods and, consequently, the Portland Cement Association (PCA) mix design method summarized in the *Soil-Cement Laboratory Handbook* of the PCA (1971). There is, however, a significant difference in the manner the test specimens are brushed during the freeze-thaw and wet-dry tests. In the Alberta procedure, the specimens are only brushed after the twelve cycles of the test in each case, and with the application of a force of about 5 kg (11 lb). The PCA procedure on the other hand, requires brushing of specimens after each cycle with a force of approximately 1.4 kg (3 lb). Consequently, while the maximum allowable soil-cement loss for the Alberta procedure is 3 percent, the PCA value for most of the soils encountered in Alberta is 14 percent.

There is a hint of the PCA method giving slightly higher cement contents than the Alberta method for the sands in the province. Evidence of this is somewhat supported by data from a number of mix design tests conducted at the soil-cement design laboratory of Alberta Transportation in late 1975. However, the number of tests and differences in the cement contents (in the order of 0.5 percent) are so small further extensive comparisons are necessary before any conclusive judgements can be made.

3.8 Simplified or Short-cut Methods of Design

A number of workers have made efforts to come up with alternative mixture design methods which would require less time than the freeze-thaw and wet-dry test method presently used in Alberta. Notable among them are attempts by Hutchinson (1963) and Yedavally and Anderson (1972).

The work of Larnach (1960) on the strength of "partially" compacted soil-cement in relation to dry density and cement/voids ratio was the basis for both investigations. Larnach determined there is a relationship between the unconfined compressive strength, S, and the cement/voids ratio, C/V, of soil-cement of the form:

$$S = A(C/V)^N \tag{3.1}$$

where A is a constant representing the intrinsic strength of a mix, i.e. the strength at C/V equals one, and N is a constant which depends on material characteristics [15]. Furthermore, based on a knowledge of a relationship between dry density and C/V, Larnach (1960) was able to show compressive strength is related to dry density at any particular cement content; and that for soil-cement mixtures of equal dry density, compressive strength and cement content are almost linearly related.

Hutchinson's Approach

In a paper in 1963, Hutchinson (1963) pointed out that the development of mix design charts which would allow the cement reqiurements of a sand to be readily estimated from the sand gradation and thereby eliminate time consuming mix design methods would be very beneficial. Such charts would make the continuous adjustment of construction cement

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[15] This relationship is a specific case of a more general "law" proposed by Feret in 1896 relating the strength of cement and aggregate mixtures to cement/voids ratio.

contents possible without the need for any detailed field testing.

With this in mind, Hutchinson introduced the term grading modulus, GM, which he defined as the relative surface area of a sand. A sieve analysis with the six standard sieves and the 80 μm sieve is essentially the only test required to determine GM. By fitting cohesive strength data from triaxial compression tests and C/V data of 55 soil-cement mixtures from 11 Alberta sands to Equation (3.1), Hutchinson established a relationship between the constants A and N, and GM. On that basis, he developed a graphical model which gives the cohesive strength as a function of GM and C/V. Data from freeze-thaw tests were then imposed on the graphical model to separate satisfactory and unsatisfactory mixes, and the chart shown in Figure 3.6 obtained.

Further tests on a large scale, Hutchinson hoped, would lead to the development of a final chart for mixture design. Through correlations with field performance, 7-day cohesive strengths could be established for various sands. With the GM of a sand known, mix design would then essentially comprise of a determination of C/V from the chart, taking the cohesive strength limits and the freeze-thaw durability criteria into consideration.

Limitations

Although the use of cohesive strength in place of unconfined compressive strength as in the original Larnach (1960) equation may not be detrimental per se, lack of cohesive strength data has probably contributed to the Hutchinson (1963) procedure not being investigated much further. A method employing the preferred and most commonly determined parameter of compressive strength would probably fare better.

By definition also, it appears an infinite number of sand gradations will give the same GM. A high GM value seems to only indicate a high fraction of fines, while a low GM indicates a higher fraction of coarser material. Consequently, GM may be too broad a parameter to use in the design method. The chart may end up establishing the same cement contents for sands with nothing in common other than the same GM. In fact contrary to the findings of Hutchinson (1963), Yedavally and Anderson (1972), from an analysis of mix design data for over 100 sand sources of Alberta, conclude that strength and consequently A and N are not functions of GM.

Comparison with other specific cases of Feret's "law" also gives the same indication. For instance, by inference from the works of Powers and Brownyard (1948) and Gilkey (1961), for a soil-cement mixture, the constant A is probably more related to the intrinsic strength of the cement than to the much stronger aggregates. N is also dependent on aggregate charateristics such as grading, surface texture, shape, strength, stiffness and maximum size, most of which are not directly related to GM. Thus, the relationship between strength and GM may only have been the result of similarities in the 11 sands tested by Hutchinson (1963). Further investigations with a wider range of sands is necessary to determine the reliability of the procedure.

3.8.1 Yedavally and Anderson's Approach

Using another approach, Yedavally and Anderson (1972) fitted compressive strength and cement/voids ratio data, grouped on the basis of the maximum dry density attainable, to Equation (3.1). The data used were obtained from over 350 mixture design data compiled over the years by Alberta Transportation. From regression analysis, they determined the constants A and N are related to density; and for a particular density, the unconfined compressive strength of a soil-cement is linearly related to the C/V ratio.

As C/V is apparently related to the density and cement content of a soil-cement mixture, Yedavally and Anderson (1972) investigated for a possible relationship between the strength and cement content of mixtures within the various density groups. They came up with the chart shown in Figure 3.7, which indicates unconfined compressive strength is a function of cement content for different densities, and is in agreement with the findings of Larnach (1960). Drawing on an earlier work by Circeo *et al*. (1963), which reported a logarithmic relationship between the freeze-thaw loss and cement content by weight of a soil-cement mixture, Yedavally and Anderson (1972) also developed the chart in Figure 3.8 for soil-cements with the different densities shown.

The simplified design method suggested by Yedavally and Anderson (1972), involves the use of Figure 3.8 in conjunction with specified limits of allowable soil-cement loss, to obtain the cement requirement for a mixture whose maximum density is previously determined from a Proctor density test. Figure 3.7 is used to confirm if the cement content so determined will meet compressive strength requirements. As Yedavally and Anderson (1972) point out, use of the charts in this way could limit an estimation of the cement content to the two days required for the Proctor density test.

Limitations

No mention is made of the cement content at which the Proctor test is conducted to determine the density range to use in the simplified procedure. It appears the method of estimating cement content described in section 3.6.2 *ante* would therefore be a required part of the procedure, making it dependent on the PCA (1971) short-cut method. Such a dependence of one simplified method on another could lead to unreliable results.

Particularly low and unsual cement contents are also obtained from Figure 3.8 for the soil-cement mixtures in the density range of 101 to 105 psi (1609 to 1690 kg/m^3). In fact the graph of that group breaks a pattern set by the others''. Investigation of mixture design data for soil-cements of this group indicates this is the result of the wet-dry test, and not the freeze-thaw test, being the governing criteria in the selection of the design cement content. The soils in this density group are mostly silty sands (A-2-4 or SM soils) with optimum moisture contents ranging from 14 to 18 percent by weight of dry soil. For such soils the shrinkage and swelling forces resulting from the wet-dry test are more paramount to durability. Consequently, soil-cement losses during the wet-dry test are a very important part of their mix design procedure.

3.9 Importance of Wet-Dry Test

There is the tendency for most practices to relate the freeze-thaw and wet-dry tests to the climate of an area. For instance, in the development of the simplified or short-cut methods described above, Hutchinson (1963) and Yedavally and Anderson (1972) only considered the freeze-thaw test as essential to mixture design in the cold climate of the province. This is an erroneous impression which should be corrected.

''Correlation of 26 data points by the author, as against the 10 by Yedavally and Anderson (1972), indicates the relative position of the graph is unchanged, removing any doubts as to the authenticity of the graph.

As pointed out in the preceding section, this may be disastrous for soil-cement mixtures of silty soils for example. In fact inspection of Alberta Transportation data reveals that three possibilities are almost equally likely to occur in a particular design. There are the instances where both the wet-dry and freeze-thaw test equally govern the selection of the design cement content. In other cases, soil-cement loss during the freeze-thaw test is the more important criteria. Finally, there are soil-cements whose durability and strength are governed mainly by the wet-dry test as mentioned above.

3.10 Summary

Cement-treated base mixture design in Alberta is by the procedure outlined above involving a freeze-thaw test, a wet-dry test and unconfined compressive strength tests. This method has been successfully used over the years, and often provides the most economic and adequate cement content for construction, as well as the optimum moisture content and maximum density which serve as guidelines during construction. A soil-cement loss of less than 3 percent by weight after standard brushing of test specimens and a minimum compressive strength of 2.07 MPa (300 psi) are required of a suitable CTB mixture.

One unfavorable aspect of the mixture design method is the over 32 days required for conducting the necessary tests. As a result, there have been attempts to derive shortcut methods of design requiring no elaborate laboratory tests. Such shorcut methods could be used for minor rush projects where the substantial savings in time and labor accruing would compensate for the risk involved in their use, or, for example, checking the mix design parameters of new soils encountered on an ongoing project. For major projects though, the practice by Alberta Transportation of conducting mixture designs during the usual six-month fallow period between consecutive construction seasons, eliminates most of the adverse effects due to the length of time required for the standard tests.

Table 3.1 Limits for Deleterious Materials Found in Water for CTB

Material	Limits, ppm[1]
Sodium and Potassium Carbonates and Bicarbonates	1 000
Sodium Chloride and Sulphate	20 000
Calcium and Magnesium Carbonates	400
Magnesium Sulphate and Chloride, and Iron Salts	40 000
Miscellaneous Inorganic Salts (sodium sulfide)	500 (100)
Seawater salt	35 000
Acids	10 000
Alkalies	5 000
Solids in Industrial Waste Waters	4 000
Organic Matter in Sanitary Sewage	20
Sugar	500
Silt or Suspended Particles	2 000
Oils	20 000

[1] ppm = parts per million.

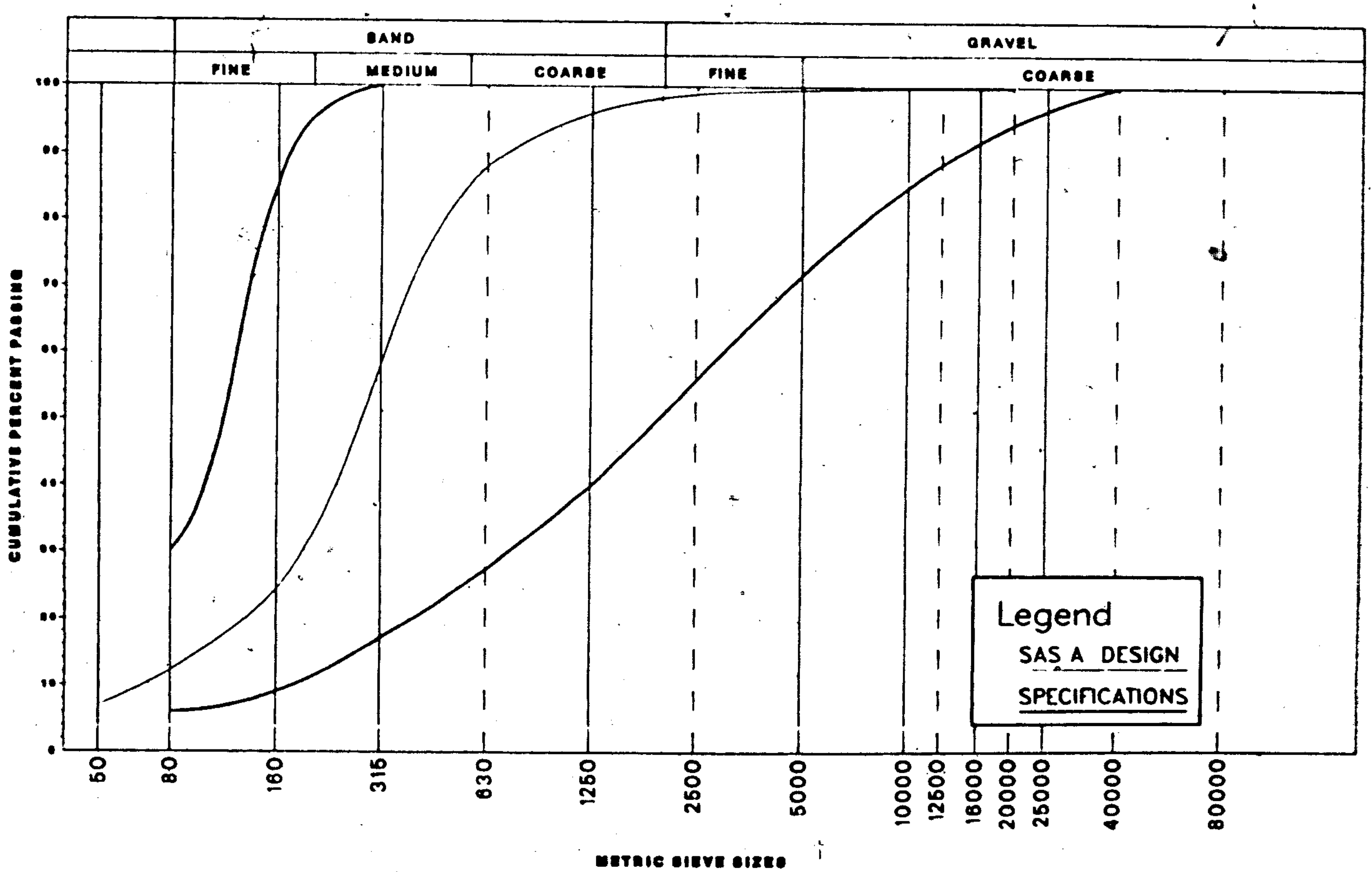

Figure 3.1 Specified Gradation Limits for CTB

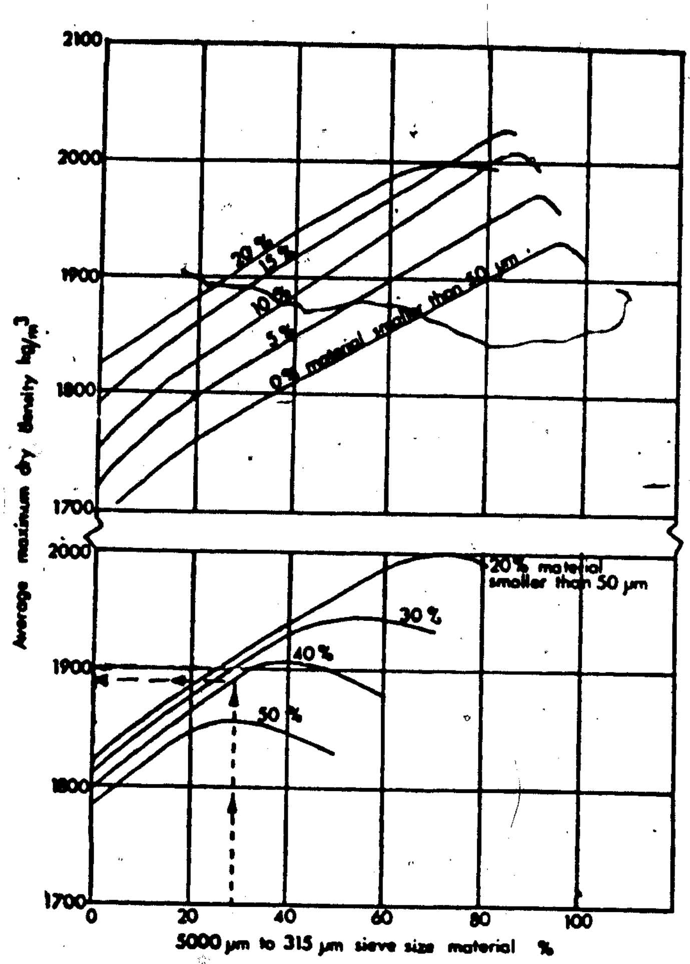

Figure 3.2 Average Maximum Densities of Soil-Cement Mixtures not Containing Materials Retained on the 5 000 μm Sieve

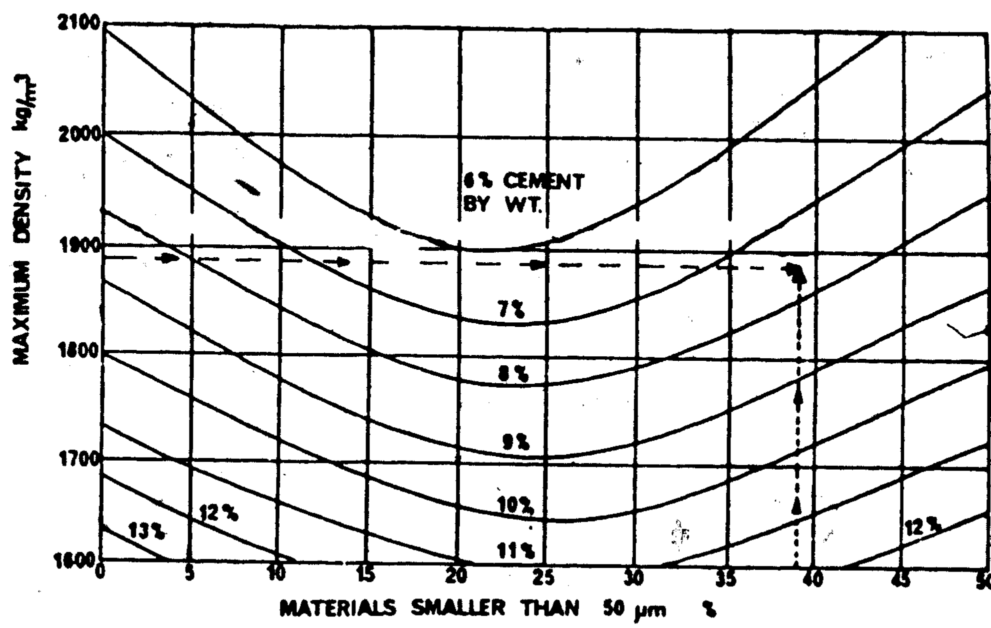

Figure 3.3 Indicated Cement Contents of Soil-Cement Mixtures not Containing Materials Retained on the 5.000 μm Sieve

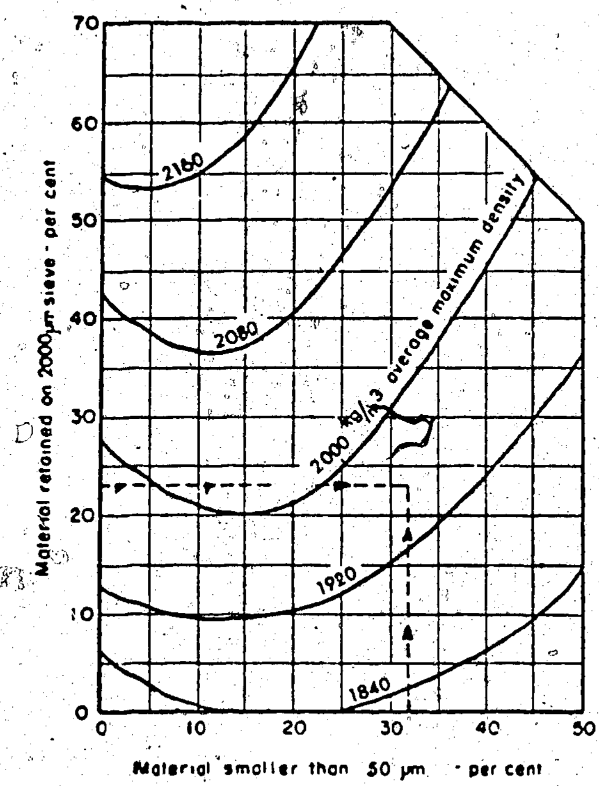

Figure 3.4 Average Maximum Densities of Soil-Cement Mixtures Containing Materials Retained on the 5 000 μm Sieve

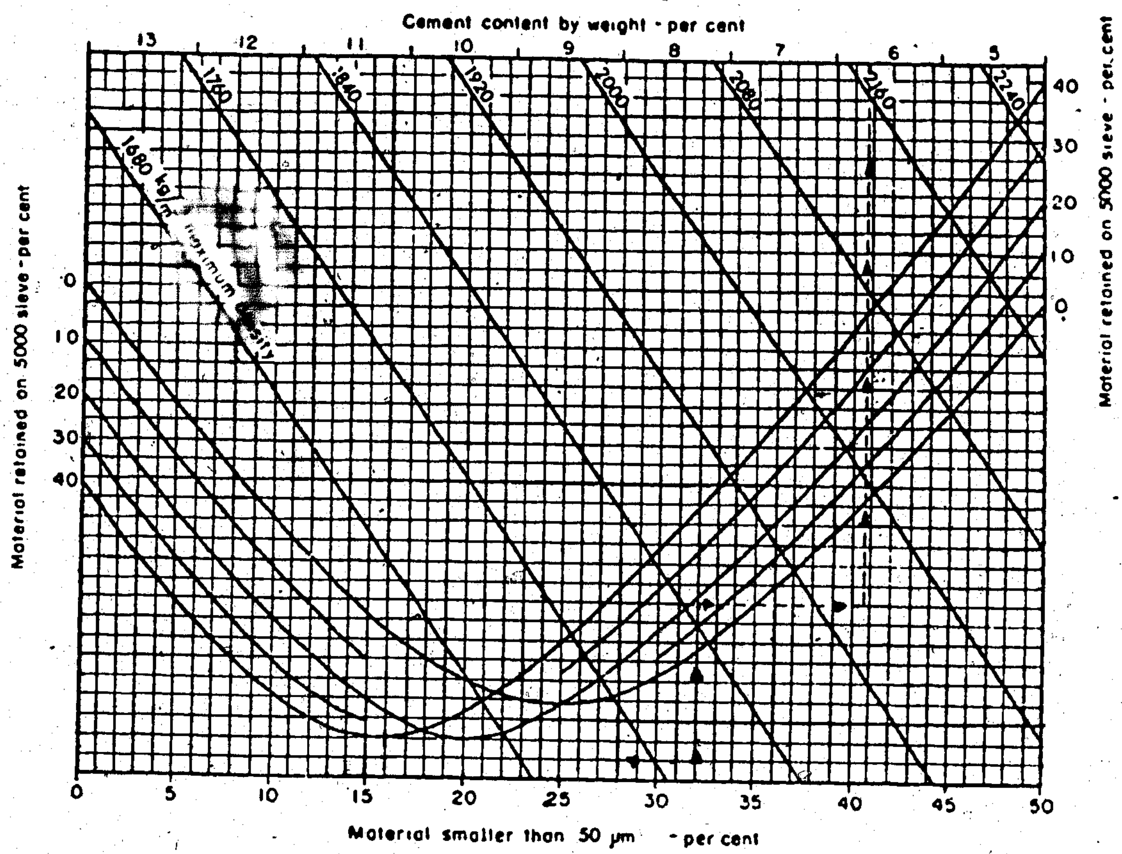

Figure 3.5 Indicated Cement Contents of Soil-Cement Mixtures Containing Materials Retained on the 5 000 μm Sieve

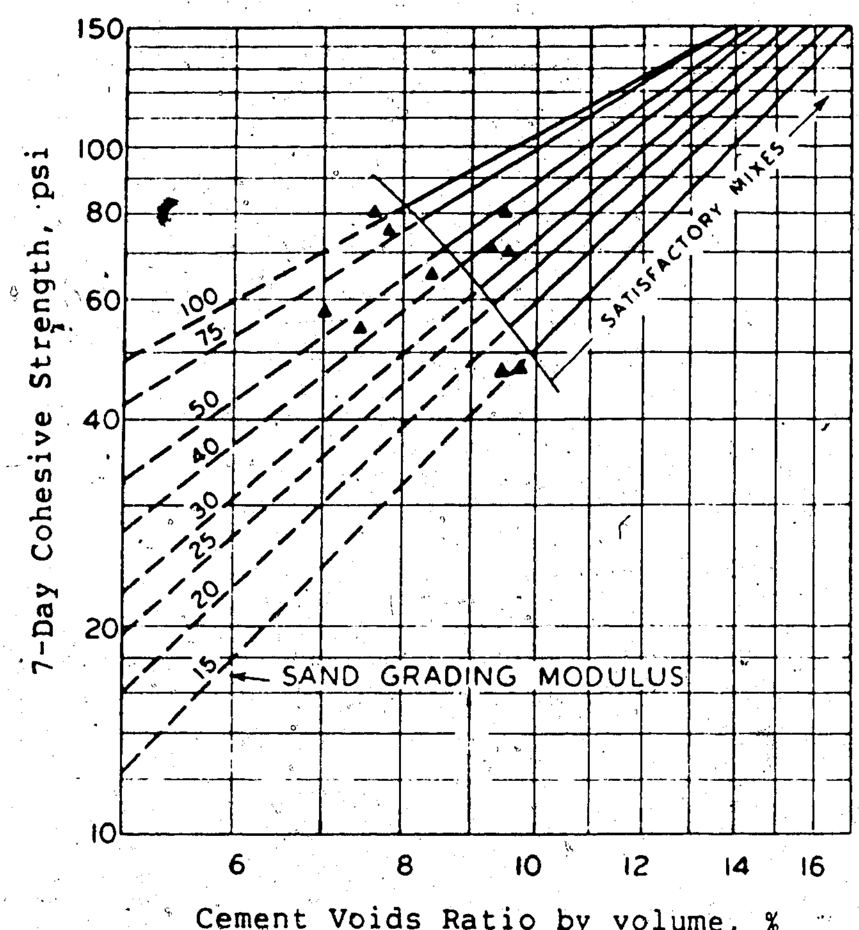

Figure 3.6 Graphical Model Relating Cohesive Strength and Grading Modulus to Cement/Voids Ratio

1 psi = 0.0069 MPa

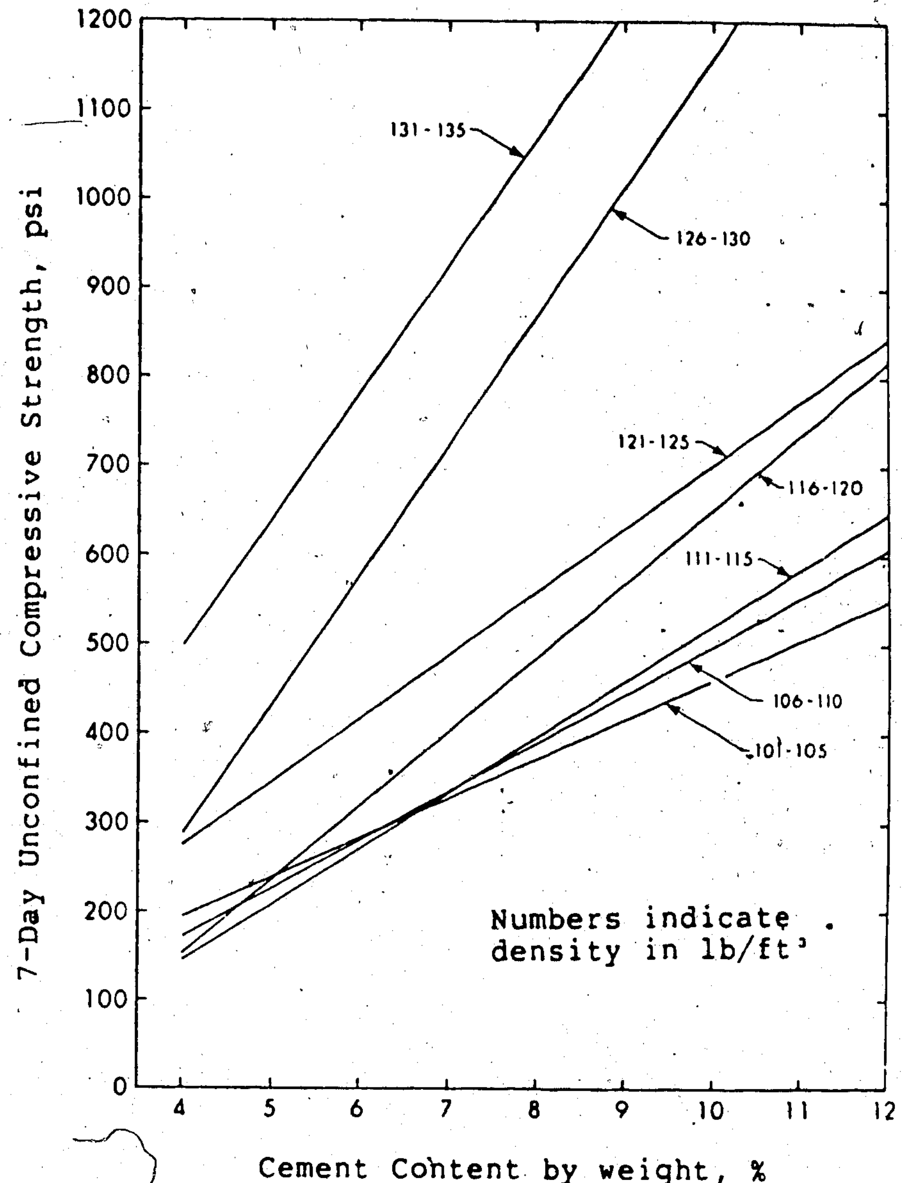

Figure 3.7 Compressive Strength as a Function of Cement Content for Various Densities

1 psi = 0.0069 MPa; 1 lb/ft³ = 16.0 kg/m³

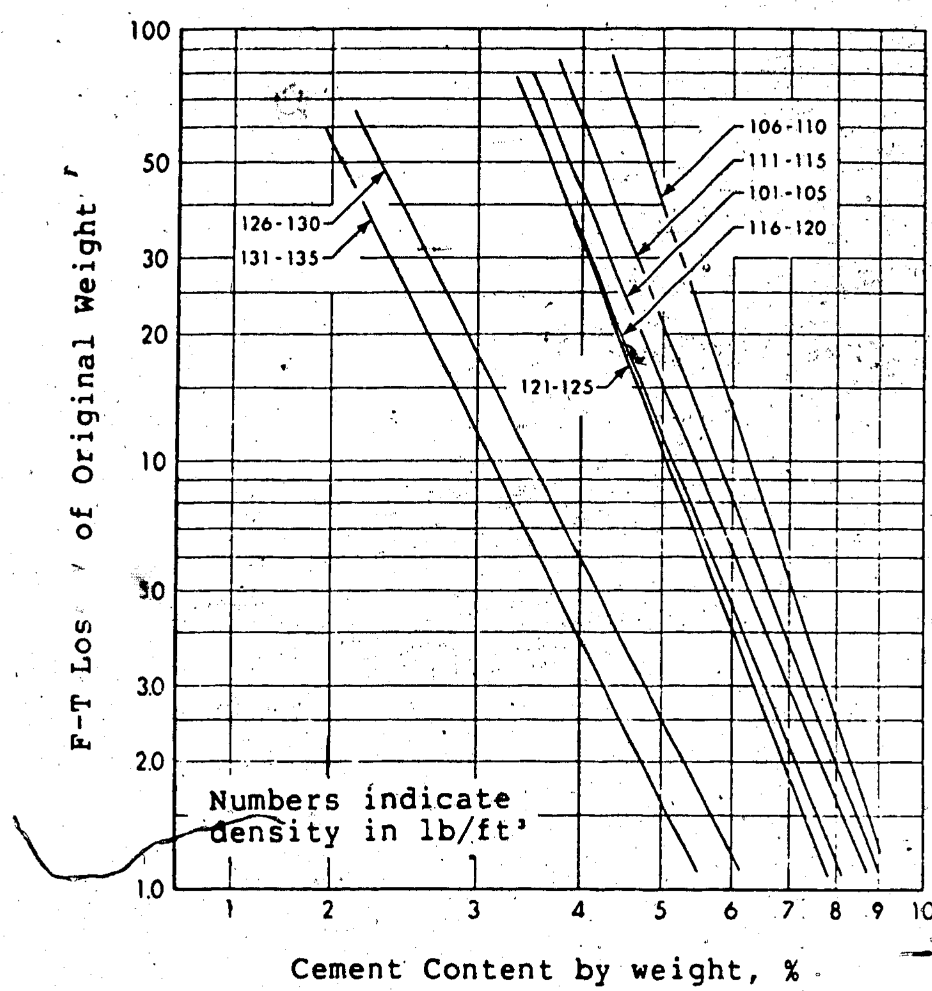

Figure 3.8 Freeze-Thaw Loss as a Function of Cement Content for Various Densities

1 lb/ft³ = 16.0 kg/m³

4. THE STRUCTURAL DESIGN OF CEMENT-TREATED BASE PAVEMENTS IN ALBERTA

4.1 Introduction

The structural design of cement-treated base or soil-cement pavements in Alberta is predicated on the assumption of a high standard of mix design, and construction and maintenance practices. With the assurance of such high standards, the actual structural design is confined to the arbitrary selection of the thicknesses of the various layers of the pavement structure, taking into consideration the traffic volume expected on the roadway in question.

This structural design which relies on past experience has been in use in the province since the beginning of serious cement-treated base construction in 1959. The heavy reliance on precedent has not precluded the provision of structurally adequate soil-cement or cement-treated base pavements. Major contributory factors in that direction are the similarities in the aggregate and other materials used, the mix design and construction practices, and the environmental conditions as far as cement-treated base pavements in Alberta are concerned.

This chapter outlines the structural design procedure for cement-treated base or soil-cement pavements in the province.

4.2 Structural Composition of Pavements

All cement-treated base (CTB) pavements in Alberta basically consist of a layer of CTB on a prepared subgrade with an overlying surfacing of asphaltic materials. In Figure 4.1, is shown a typical transverse cross-section of the CTB pavement structure usually used in the province. It comprises of a CTB layer overlaid with an asphalt stabilized granular base course (ASBC) layer, and surfaced with an asphalt concrete pavement (ACP). The thicknesses of the various layers are varied to give the required structure for the different traffic and environmental conditions found in the province.

A type of pavement structure previously used for light-trafficked roads in Alberta comprised of a CTB layer and an overlying surfacing of ACP, with the ASBC layer omitted. However, its use has ceased for some time now and few if any exist in in the province at present.

4.3 Basis for the Design

In general, as discusssed in Chapter 3, the types of soil used for soil-cement construction in Alberta are very similar. They are mainly SP sands and, in some instances, SM sands with liquid limits 28 or less and plasticity indices less than or equal to 6. Design cement contents normally range from 6 to 8 percent by weight of dry aggregate, and in all cases continuous stationary plants are employed in the production of the CTB mixture used for construction.

Subgrade soils are usually silty clays, sandy clays and clays of low to medium plasticity, and environmental conditions are virtually the same across the province. Consequently, there is not much variation in the type and/or conditions of service on the CTB pavements in the province.

With this background, it is reasonable to assume that with proper subgrade preparation, the same pavement structure will be adequate to a large extent for carrying the same volumes of traffic in any part of the province. On this basis, the dimensions of a new pavement are chosen such that they conform with those used for previously constructed pavements which carry traffic volumes similar to that anticipated on the new road. As part of this empirically based design, however, high standards of mix design, construction and maintenance practices are required for reliable and successful results to be obtained.

4.4 Thickness Design

As is evident from above the structural design of soil-cement pavements in Alberta is therefore simplified to the selection of the thicknesses of the different layers of a pavement structure.

4.4.1 Road Classification

In line with this design philosophy CTB roads in Alberta are classified into four groups. The basis for this classification is the anticipated cumulative traffic up to the time of first resurfacing, which in the province is between 10 to 15 years. This puts CTB roads into the classes of those which carry light traffic, medium traffic, moderately heavy traffic and very heavy traffic. In Table 4.1 are given the cumulative 8 160-kg (18 000-lb) equivalent single axle loads (ESALs) to terminal serviceability level for each of the groups.

4.4.2 Thickness Selection

The base course thicknesses which in Alberta have proven over the years to be adequate for the various cumulative ESAL categories are shown in Table 4.1. In the design of a new road, the expected cumulative ESALs to terminal serviceability is determined, and the final compacted thickness of the soil-cement base course selected using these thicknesses as guidelines. As part of the design, a 50-mm thick ASBC is provided on top of the CTB layer, and finally, an ACP thickness is selected for the particular traffic category. The ACP thicknesses for each category are also shown in Table 4.1.

In practice, all the pavements in the province fall into the categories of roads which either carry light traffic or moderately heavy traffic. Consequently, only the CTB layer thicknesses of 150 mm (6 in.) and 200 mm (8 in.) have been used for CTB pavements in Alberta as of now. The 50-mm ASBC is always placed on the soil-cement base course, and ACP thicknesses used range from 50 to 100 mm depending on the type of load to be carried by the road.

4.5 Reliability of Structural Design

The updated Alberta CTB thicknesses shown in Table 4.1 compare favorably with AASHTO (HRB, 1962) and PCA (1970) figures culled from the text *Pavement Management Guide* of the Roads and Transportation Association of Canada (RTAC, 1977) in Table 4.2. It is emphasized that the dimensions given in Table 4.2 are for comparative purposes and are not design recommendations.

Although, according to layer equivalencies for Alberta reported by RTAC (1977), the 50-mm ASBC in CTB pavements is approximately equivalent to 70 mm of CTB, it is the practice in the province to regard it more as a protective layer for preventing reflective cracking of the ACP wearing surface, than as a pavement component contributing to structural strength. Therefore, the thicknesses of the other pavement layers are never reduced on account of the strength contribution of the ASBC. This apparently makes for a better pavement, as does the ACP thicknesses used in the the province. These latter thicknesses are higher than the minimum of 40 mm (1.5 in.) recommended for use by the PCA (1970) in frost areas where snowplows are used, a condition which applies to the province of Alberta.

4.6 Summary

The structural design procedure presented, although very empirical in nature, has been successfully used in Alberta for the construction of over 3 000 km of CTB roads. The success can mainly be attributed to the general uniformity in the materials, mix design methods, construction and maintenance practices used in the province. Consequently, the designs based on precedent have been quite reliable. Very high standards of mix design and construction practices are also contributory factors to the success of the structural design method.

The thicknesses provided have been structurally adequate for the different road classes, and the inclusion of the 50-mm thick asphaltic stabilized base course is more of the norm than an oddity. This practice is apparently very good, as the asphaltic stabilized base course is very beneficial to the overall performance of the pavement structure, serving as a protective layer against reflective cracking, although not providing much support by way of giving strength to the pavement.

Table 4.1 CTB Base Course Thickness Requirements for Various Traffic Categories

Category	Cumulative Equivalent 8 160-kg Single Axles, ESALs	CTB Thickness, mm, (in.)	ACP Thickness, mm, (in.)
Light Traffic	5×10^5	150 (6)	50 (2)
Medium Traffic	10^6	175 (7)	50 (2)
Moderately Heavy Traffic	2×10^6	200 (8)	100 (4)
Very Heavy Traffic	Over 2×10^6	230 (9)	150 (6)

Table 4.2 Comparison of CTB Thickness Requirements

	Cumulative Equivalent (8 160-kg) Single Axles, ESALs, to Terminal Serviceability Level				
	10^5	5×10^5	10^6	2×10^6	5×10^6
Alberta[1]		150 mm	175	200	230
AASHTO Road Test Special Base Experiment (HRB, 1962)[2]	130 mm	180	210		250
PCA Fatigue Design (PCA, 1970)[3]	160 mm 155 mm	170 165	180 170		185 175

[1] Updated design includes 50 mm asphalt stabilized granular base course; 50 mm, 100 mm and 150 mm ACP for ESALs up to 5×10^5, 2×10^6 and 5×10^6, respectively. Medium plasticity sandy clay and silty clay sub-soils, no subbase; mainly SP sands stabilized with 6-8% cement.

[2] 75 mm ACP and 100 mm sandy gravel subbase; sandy gravel base stabilized with 4% cement.

[3] 50 mm ACP for higher thicknesses; 75 mm ACP for lower thicknesses; *Granular* soil cement, weak subgrade.

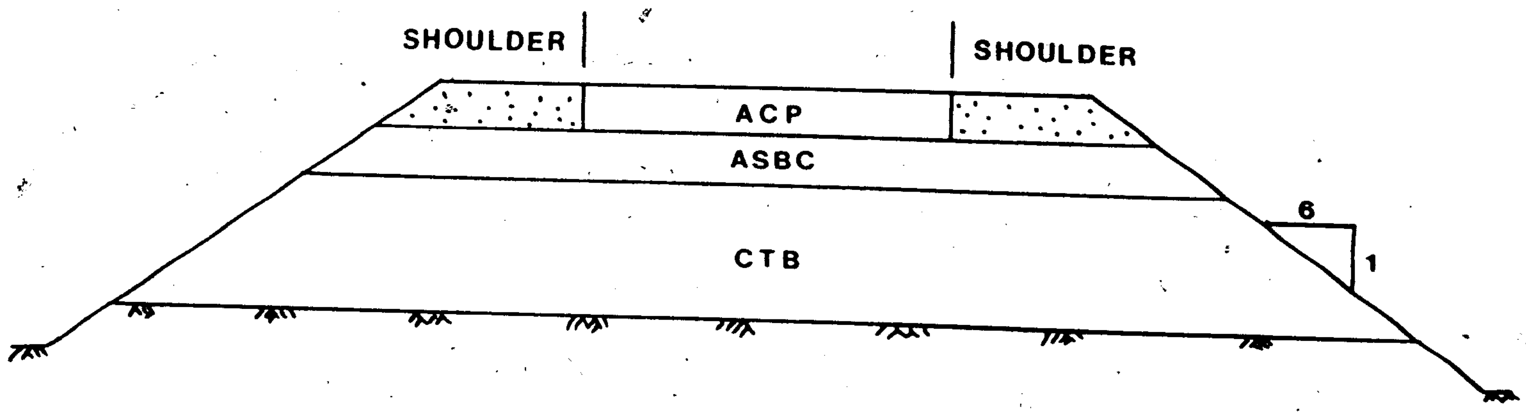

Figure 4.1 Typical Cross-Section Showing Elements of a CTB Pavement

5. CEMENT-TREATED BASE CONSTRUCTION IN ALBERTA

5.1 Introduction

Cement-treated base construction in Alberta is mostly by the plant-mix method. Using one hundred percent borrow material, cement-treated base is obtained by mixing aggregates, cement and water at a central plant and then transported to the road construction site in dump trucks.

At the site, the cement-treated base is spread on a prepared subgrade of the roadway and immediately compacted at the optimum moisture content to a maximum density. The compacted surface is finished to give a smooth and tight surface and sprayed with an asphaltic fog seal coat as a prelude to curing.

There is an initial curing period of at least five days during which the base course is kept free of general traffic. A further minimum curing period of another five days elapses before an usual overlay of asphalt stabilized granular base course is constructed. The road at this stage is opened to traffic until the next construction season when a final surface course of asphaltic concrete is constructed on the base courses.

This chapter is a description of the procedures, equipment and quality control measures employed in the construction of cement-treated base pavements in Alberta. Also presented are the methods of measurement and payment.

5.2 General Construction Procedures

The use of predominantly single-sized borrow sands for cement-treated base (CTB) material in Alberta is responsible for a general preference for central plants in the province. Specifically, as recommended by Alberta Transportation in an effort to standardize construction procedures, continuous central plants are employed in the production of CTB for construction projects.

This material, produced during what may be regarded as a first phase of construction, is transported in haul trucks to the roadway site for the construction of a base course layer on a prepared subgrade in the second phase of construction. According to Alberta Transportation specifications, the plant and construction equipment for these two phases of construction, should be capable of producing and placing a minimum of 200 tonnes per hour of CTB material. A set of quality control tests associated with each of the two phases ensures the provision of the best quality base course. Also, to allow the newly constructed base course attain adequate strength and durability, production and placing of the CTB material is not permitted when temperatures are equal to or below 5 C; or when conditions indicate temperatures will be in that range within 24 hours.

The last phase of construction, not of major concern here, involves the construction of the overlying protective asphaltic material layers, which usually comprise of an asphalt stabilized granular base course (ASBC) and an asphalt concrete surface wearing course (ACP). The different phases of the construction of a CTB pavement in Alberta are discussed in the subsequent sections of this chapter.

5.3 Plant Production of CTB

The plant for CTB production is usually located at an aggregate source which, as pointed out by Dacyszyn (1961), should be able to supply at least a third of the total quantity of aggregates needed for a particular project. This is necessary to ensure enconomy of construction. Water is either drawn directly from a nearby source or supplied by water tankers; and the cement is drawn directly from trailer-type bulk carriers or silos transported to the site. A field laboratory and a scale-house with a scale for weighing loaded trucks complements the plant equipment.

5.3.1 Preliminary Preparations

This involves stripping borrow pits of their vegetative and otherwise unsuitable top-soil cover and the stockpiling of the aggregates obtained from them. Where blending of aggregates with different gradation, either of the same pit or from separate pits, is necessary, operations are carefully controlled to ensure a homogeneous mix is obtained. Aggregate degradation and/or segregation is kept at a minimum, and where required, very wet aggregates are dewatered by aeration or pumping. A plant layout for efficient operation and production, as well as the minimization or complete elimination of potential sources of hazards to personnel is also ensured.

5.3.2 Plant Operation

The units which make up a CTB plant are usually remote controlled from a control panel set high above the plant, often with a capability for the independent control of the separate units. An essential feature of such a plant is provision for the possible diversion of the aggregates and cement from their respective feeders into external receptacles for measuring. The various units are described below. Illustrations are presented of a typical CTB plant in the province.

Aggregate Feeder

The aggregate feeder comprises of a hopper with either an adjustable or fixed gate which discharges aggregates from a stockpile onto a conveyor system. Plate 5.1 shows a partially buried feeder discharging aggregates onto a conveyor at **F.** A loader or bull-dozer ~~con~~tinually replenishes the hopper of aggregates to ensure a continuity in operations. The aggregates are transported to screening units as a first step in their preparation for use.

<u>Screening Unit</u>

The screening unit removes stumps, roots, large rocks, lumps of aggregates larger than the maximum specified size and other unsuitable material from the aggregates. Two screening units are often employed. A first screening unit removes very large materials such as stumps, roots and rocks, and a second unit screens off aggregates larger than the maximum specified size. Plate 5.2 shows such screens **S** and **T** respectively. For more effective screening, the second screening unit (**T**) usually has two screens, as for example shown by **M** and **N** in Plate 5.3.

<u>Cement Feeder</u>

Cement feeders usually comprise of a surge bin with a vane feeder which adds cement withdrawn from a silo to the screened aggregates. The cement is pumped under compressed air to prevent bulking. The vane feeder may either have a fixed speed and discharge cement at a fixed rate, or be of a variable speed with a variable cement discharge rate. The cement feeder **B** in Plate 5.4 is a typical example which withdraws cement from a silo **A** and adds it the screened aggregates on the conveyor at C. (**G** and **W** in Plate 5.4, houses the electricity generator and control panel respectively).

Pugmill

The measured amounts of aggregates, cement and water are mixed together in the pugmill. It comprises of a rectangular boxlike chamber with two rotating shafts running parallel to the direction of flow. The shafts are fitted with paddles which do the mixing. To ensure a more uniform mix, water is added to the mixture some distance (minimum of 0.5 m) upstream of the entrance of the pugmill, where the soil and cement are introduced, to allow an initial dry mixing of the soil and cement.

P in Plate 5.4, shows the general location of a pugmill relative to the other units of a plant. Plate 5.5 is a close-up from the upstream end of the pugmill, showing the relative positions of the two rotating shafts Q and R. In the general position of the arrow D, is a dam which may be adjusted up or down, to increase or decrease, respectively, the time of mixing in the pugmill.

Storage Bin

A temporary storage bin at the end of the train of units provides a means of storing the CTB mixture for some time. In this way, plant production may continue even when haul trucks are temporarily unavailable. A typical temporary storage surge bin H is shown in Plate 5.6.

5.3.3 Plant Calibration

A plant calibration is carried out at each new plant setting before plant operation or whenever required by the project manager. Depending on the types of cement and/or aggregate feeder of a plant, there are different approaches to calibration.

For a plant with a constant speed cement vane feeder, the cement feed rate is naturally fixed. From this cement feed rate and the mixture design cement content, a projected dry aggregate feed rate is calculated. By trial-and-error, an aggregate feeder setting at which the dry aggregate feed rate is within plus-or-minus 5 percent of the projected value is then determined. This constant cement feed rate also fixes the plant CTB production rate and determines factors such as the capacity of construction equipment, number of haul trucks and the pace of construction.

An initial estimation of the wet mix production rate is required when calibrating a plant with a variable speed cement vane feeder. This estimate is established by taking into account the plant and construction equipment capacities; the haul distance and number of haul trucks; and with respect to the water that may be needed, the moisture content of the stockpile aggregates. From the estimate and the mix design parameters, the projected cement and dry aggregate feed rates are calculated. Separately, and by trial-and-error, the cement and aggregate feeder settings at which actual feed rates come to within plus-or-minus 5 percent of the projected values are determined.

In both situations described above, determination of the aggregate or cement feed rates at a particular setting, is accomplished by diverting material from the particular feeder into an external receptacle and measuring the discharge per some unit of time. With the plant running at the aggregate and cement feed rates so determined, the rate of water addition is gradually increased from zero to a rate at which accompanying tests indicate the moisture content is at the required level.

The quality of the CTB material produced with the plant operating at the calibrated settings is checked, and remedial action, which may include prolonged mixing in the pugmill or recalibration at a lower setting, taken if the quality is judged poor. On the other hand, there may be recalibration to increase the production rate, if the efficiency of production obtained at the initial setting is considerably high. Typical wet CTB production rates range from 400 to 600 tonnes per hour for the plants used in the province.

5.3.4 Plant Quality Control

In order to make plant quality control tests more effective, the total daily production is divided into four and the following tests conducted on each quarter called a production unit.

a) a production rate test

b) a moisture-density relations test.

c) a CTB mixture moisture content test

d) a cement titration test

e) a sieve analysis of stockpile aggregates

In addition, a bulk cement content is determined for the CTB mixture produced each day, i.e. after each fourth production unit. The quality control tests are briefly discussed in this section.

To ensure the samples for the tests are representative of the mixture produced, the production rate test is performed before the other tests, and the samples for tests b), c) and d) are usually obtained from the same haul truck. The tests are then conducted concurrently within an hour, to minimize the detrimental effects resulting from partial cement hydration.

Production Rate Test

This involves measuring the discharge of CTB in tonnes per hour produced at the plant. At least one test is performed every hour, and remedial action is taken when the average production rate determined for a production unit is not within plus-or-minus 5 percent of the calibrated value. As part of the test, the cement vane feeder discharge rate is determined from the cement production rate, and corrective measures taken where it is not within plus-or-minus 0.3 percent of the calibrated value.

Moisture-Density Relations Test

A three-point standard moisture-density relations test, using a 2.5-kg rammer, dropped from a height of 304.8 mm, is performed on a representative CTB sample of each production unit. The optimum moisture content and maximum dry density so determined are the values which govern the field compaction of the material within the particular production unit. Generally, the average dry density of a production unit should compare favorably with the laboratory design value. A new mix design is necessary if there are consistent variations of more than plus-or-minus 80 kg/m^3, and there is a noticeable change in aggregate gradation.

Moisture Content

A moisture content test using the conventional oven drying method at a temperature of 110 C plus-or-minus 5 C, or the open pan method with measures taken to prevent loss of material by splattering, is conducted on a representative sample of each production unit. For sandy soils, the moisture content is expected to be equal to or slightly below the optimum moisture content. Consequently, a deviation of up to 2 percent by weight below the optimum is allowed. Higher than optimum moisture contents are, however, avoided for CTB of the sandy soils used in the province.

Cement Titration Test

This chemical analysis test method is used to determine the cement content of a CTB mixture. It involves the determination of the amount of an acid (HCL) of known concentration needed to neutralize a CTB solution in water. The cement content of the CTB is then obtained from a standard curve of cement content against weight of acid titrated. This standard curve is earlier obtained by titrating a CTB-water solution of known cement concentration with the same acid. For aggregates with appreciable amounts of alkalies, an adjustment is made for the acid which goes into neutralizing the alkalies. A variation in the cement content determined in this manner of more than plus-or-minus 5 percent from the design value requires immediate corrective measures.

Sieve Analysis Test

A washed sieve analysis of a representative aggregate sample obtained from the feeder is conducted to determine the aggregate gradation for each production unit. With specifications requiring all the aggregates pass the 40 000 μm sieve, the sieve sizes used are a top size of 20 000 μm, followed by the 10 000, 5 000, 1 250, 315, 160 and 80 μm sieve sizes. The gradation graph obtained should compare favorably to the design gradation. Blended aggregates are given special attention during this test.

<u>Bulk Cement Content</u>

The bulk cement content of the production for each day is determined as the ratio of the total cement consumption to the total dry aggregates used by weight. The total cement consumption is determined from the cement invoices, and the dry aggregate weight from the scale-house data. In the calculation of the dry aggregate weight, the average of the moisture contents for the production units in a day, is taken as the moisture content of the total wet mix produced. The bulk cement content so determined is mainly for comparative purposes, but excessive and repeated variations warrants remedial action to improve the quality of material subsequently produced.

5.4 Base Course Layer Construction

Following production, the CTB is transported to the construction site in self-unloading trucks, which typically in the province have capacities ranging between 12 and 15 tonnes. Of prime importance is the haul distance from the plant to the roadway. As partial cement hydration results in difficulty in compaction and a consequent decrease in density, strength and durability, it is essential to limit the time between when water is added to the dry soil-and-cement mix in the plant and the completion of compaction in the field, to a maximum of two (2) hours (cf. section 2.6.3, *ante*).

5.4.1 Construction Steps

Construction of the CTB base course layer usually involves subgrade preparation followed by construction of the base course layer. On the average, 1.5 to 2.0 kilometers of roadway is constructed in the province per day for typical CTB projects. The construction steps are described in the following paragraphs.

Subgrade Preparation

This comprises shaping the subgrade to the final crown and grade. The subgrade soil is ripped to a depth of at least 150 mm (6 in.), pulverized, mixed and compacted at the optimum moisture content, to a density of at least 98 percent of the maximum standard proctor density. The depth of preparation is increased up to a limit of 300 mm (12 in.) where the subgrade soil is very wet.

Mat Construction

The plant-mixed CTB hauled to the roadway site is spread on an adequately moistened subgrade by spreaders fitted with automatic depth control devices. The material is placed in widths of not less than 3 m, and depths not to exceed 200 mm (8 in.) of final compacted material. Thicker lifts are placed in two or more lifts, and joint construction procedures (discussed in a subsequent section) employed where adjacent lifts have to be placed more than thirty (30) minutes apart. Also, throughout placing, the spreader(s) are never allowed to run empty in order to ensure the provision of a base course of uniform thickness.

Compaction

For CTB of the sandy soils found in Alberta, adequate compaction is usually obtained by rolling initially with steel-wheel rollers (preferably vibratory), followed by pneumatic-tire rollers. Immediate compaction is essential, and compaction paths are overlapped by at least 25 percent. With the help of the project manager, a contractor is usually able to determine the rolling pattern which will give the best results at minimum cost and in the shortest possible time.

Finishing

Finishing operations comprise of shaving the compacted base course to remove high spots and a final rolling with a pneumatic-tire roller to obtain a smooth and tight surface. Compaction planes are removed by lightly scarifying the affected areas and recompacting to the maximum density. A light spray of water is applied where necessary to keep the material damp throughout finishing. Correction of depressions by filling them with freshly mixed material and then compacting to the specified density is considered bad practice.

Curing

Curing follows immediately after compaction and finishing. Keeping the finished surface clean of any loose or foreign material and damp with a light spray of water, a curing coat is applied. A fog seal coat of a rapid curing cutback or emulsified asphalt is usually sprayed on the surface by means of a distributor at a rate of between 0.5 kg and 1.0 kg per square meter. The base course is then kept free of traffic except for construction and local traffic which cannot be diverted to any alternate route and allowed to cure for a minimum of five days. This period is extended if temperatures fall below 5 C during curing.

Where the base course is constructed in more than a single lift, each lift is separately sealed and allowed to cure for the minimum period stated above. Exceptions to this rule are allowed when both lifts can be placed and compacted within two hours (cf. section 5.4, *ante*).

Joint Construction

Joint construction is necessary where adjacent layers are placed more than thirty (30) minutes apart. This is usually required for construction starting at the begining of a work day. Joint construction can either be undertaken at the end of the day or just prior to construction the next day. Using a grader or hand tools, a joint is formed by cutting well back into the base course, to a fully compacted face. The face cut is made vertical to the surface of the compacted subgrade, and aligned with the roadway centerline or cut normal to the centerline, for longitudinal and transverse joints, respectively. The face is thoroughly cleaned and tightly packed with freshly mixed CTB, which is compacted to the maximum density for construction of the adjoining section. Placement of about an extra 50 mm of uncompacted material to give an overlap after compaction, which is later shaved off, is considered good practice.

5.4.2 Field Quality Control

Field quality control in Alberta, comprises of both quantitative and qualitative tests conducted to ensure a high quality base course is constructed. The various tests are discussed under the headings of the parameters they measure.

Moisture Content

Although this qualitative test is not specified, it is useful for determining whether the material delivered to site has the moisture content necessary for adequate compaction. Hand molded balls of CTB mixtures with moisture content equal to or near the optimum, just hold together when dropped from a height of 0.3 m, and when squeezed tightly into a cast, let off just enough moisture to dampen the hands. Conversely, balls of CTB with moisture contents higher than the optimum tend to wet hands, while those with too low moisture contents crumble on impact with the ground.

after a free fall of about 0.3m. It is emphasized that these tests are subjective and are only used in Alberta to help attain better construction results.

Density

Prior to coating with a fog seal coat, an in-place conventional sand cone (AASHTO T191) or rubber ballon field density test (AASHTO T205) is conducted on the base course immediately after compaction and finishing. Tests are conducted at locations which mark the beginning of production units, and often in at least two more randomly selected locations spaced a minimum of 100 m apart within a production unit. A density equal to at least 97 percent of the maximum standard proctor density determined for the unit (cf. section 5.3.4, *ante*), is the minimum required for adequate compaction.

Compressive Strength

Compressive strength tests are conducted after seven days of curing on cores obtained from the base course at chainages which mark the beginning of each production unit, and at one or more random locations within each unit. In general, cores are usually secured at the end of the sixth day of curing to allow for a soaking period of 4 hours before testing on the seventh day. Basically, compressive strengths above 2.07 MPa which compare favorably with design values imply an adequate strength gain. Compressive strength tests after higher curing periods are performed as a check for locations with inadequate strengths. Remedial action is only required if the results do not compare favorably with design values.

Depths and Levels

Compacted depths are qualitatively checked by monitoring the depth of loose base material spread, as determined from probings prior to compaction. Quantitative measurement is also possible from the cores [illegible] for the compressive strength tests, although such results are often rather late to serve any useful purpose. [illegible] levels are not specifically checked. However, once a subgrade is cut to the proper crown and grade, the final compacted level is often assumed to be correct provided the mat constructed is of uniform thickness.

5.4.3 Remedial Construction

Depending on the results of the quality control tests, and from observation of the fog sealed base course prior to the construction of the asphaltic overlays, remedial construction may sometimes be required. A strong non-compliance with density or compressive strength requirements may, for instance, show up as visible failures in the base course. For hardened CTB, the repair of such excessively failed areas involves the complete excavation of the failed material well past the limits of failure and down to the subgrade surface. The material removed is then replaced with freshly mixed CTB which is thoroughly compacted, finished and sprayed with a seal coat. The repaired section is barricaded and allowed to cure for at least five days just as ordinarily constructed CTB. Junctions between adjacent old and new material are also constructed in the same manner as normal construction joints as discussed in section 5.4.1, *ante*. Remedial construction, where required, completes construction of the base course layer.

5.5 Construction of Asphaltic Layers

As mentioned previously, the asphaltic cover layers built on soil-cement base courses in the province usually comprise of an asphalt stabilized granular base course, ASBC, overlaid with an asphalt concrete surfacing, ACP. A curing period of at least ten (10) days is required by specifications between the time of application of the fog seal coat and the construction of the asphaltic cover layers.

The construction of the asphalt stabilized base course is always in the same construction season as the CTB base course. The granular material is either road-mixed with an MC-250 asphalt or plant-mixed with an MC-250 or MC-800 asphalt. Construction of the asphaltic concrete surfacing is usually delayed until the next construction season. This has also been observed by Dacyszyn (1961) and Shields and

Hutchinson (1961). A tack coat is usually applied between the asphalt stabilized base course and the underlying CTB as well as the overlying asphaltic concrete surfacing. A rapid curing cutback applied at a rate of 0.4 kg to 0.5 kg per square meter is often used.

5.6 Reporting Plant and Road Test Results

To ensure an easy reference and maximum utilization of plant and road quality control test results, daily reports to the contractor, quality control charts and log books are kept on all CTB construction projects in the province. The important test results for the production of each day are recorded on a 'Daily Report to Contractor' form and submitted to the contractor with a copy kept by the project manager. The form gives pertinent information on the quality of the CTB material produced and laid, the subgrade preparation, and the aggregates used each day.

On the quality control charts, the results from the plant and road tests are presented graphically, with the quantities plotted on the ordinate against the production unit on the abscissa. These charts permit the organization and utilization of the data collected as a result of their visual impact and ability to show trends. However, although the quality control charts are very effective in highlighting a lowering in the quality of a material produced for for example, they do not offer information on the location of a problem spot. A good interpretation of the results by the user is therefore required to locate such spots for the necessary corrective measures.

At least two log books documenting the progress of work at the plant site and at the roadway site are also kept. The plant log book usually contains information of plant calibration, tonnage checks, production units, a general job diary, etc.; while the book kept at the roadway site also gives information on subgrade preparation, daily progress, rolling patterns, core strengths, and any other useful data.

5.7 Measurement and Payment

The measurement of and payment for the works involving the construction of a CTB pavement in Alberta are usually grouped under the following categories:

a) subgrade preparation
b) construction of CTB base course layer
c) construction of asphalt stabilized granular base course
d) construction of asphaltic concrete surface course

Only the works involving the construction of the CTB base course layer is discussed here. The other works are common to most of the different kinds of pavements constructed in the province.

Method of Measurement

There is measurement of

a) the quantity of CTB in tonnes produced and accepted for construction and compacted to the specified density and thickness. It includes the CTB material used for any incidental work approved by the project manager, but not material used for the repair of failed sections resulting from poor workmanship on the part of the contractor. The quantity of the water used for mixing, compaction, curing and any other works specified by the project manager; and other materials such as sand for sanding the seal coated surface, are considered an inherent part of this measured quantity.

b) the haul of CTB material. This is made up of a basic loading factor in tonnes, and the haul in tonne-kilometers. The haul distance is taken as the average of the two distances to the opposite ends of the section of roadway under consideration.

c) the works involving brooming and application of the fog seal coat in square meters.

Payment

There is payment for the

a) total quantity of CTB at the price bid per tonne. This price is compensation for all the cost and work leading to the production of CTB from the raw materials, and placing of the CTB base course.

b) haul of the CTB material used. Payment is in terms of a basic loading factor for the total quantity of CTB at the price bid per tonne or per cubic meter; and the haul of the material at the price bid per tonne-kilometer or per cubic metre-kilometer.

c) works involving brooming and application of fog seat coat at the price bid per square meter.

5.8 Summary

In this chapter the procedures typically used in the construction of CTB pavements in the province have been prsented. The predominant use of the same kind of equipments (e.g. the continuous flow central plant); the similarity in aggregates; and use of the same principles of construction has contributed in no small way to a largely successful construction program. Not only are contractors familiar with the various aspects of CTB construction, but field quality control has become more effective over the years. The method of reporting quality control test results, namely by the use of daily contractor reports and quality control charts, also ensures maximum utilization of the results, and contributes to a high quality of first construction of CTB pavements in the province. With the difficulty in reworking or reconstructing hardened CTB this is an asset.

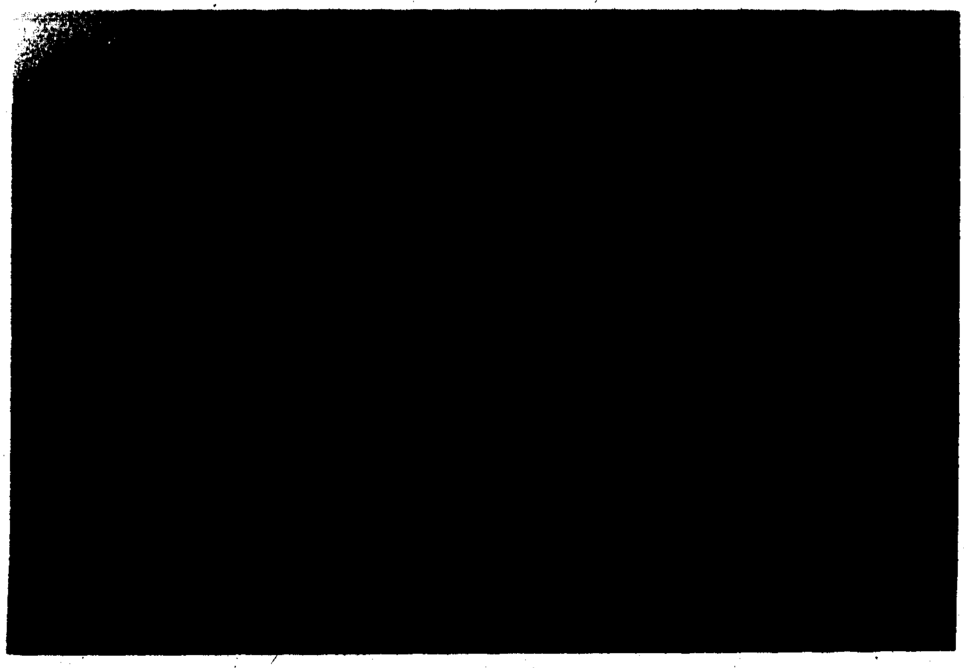

Plate 5.1 Aggregate Feeder

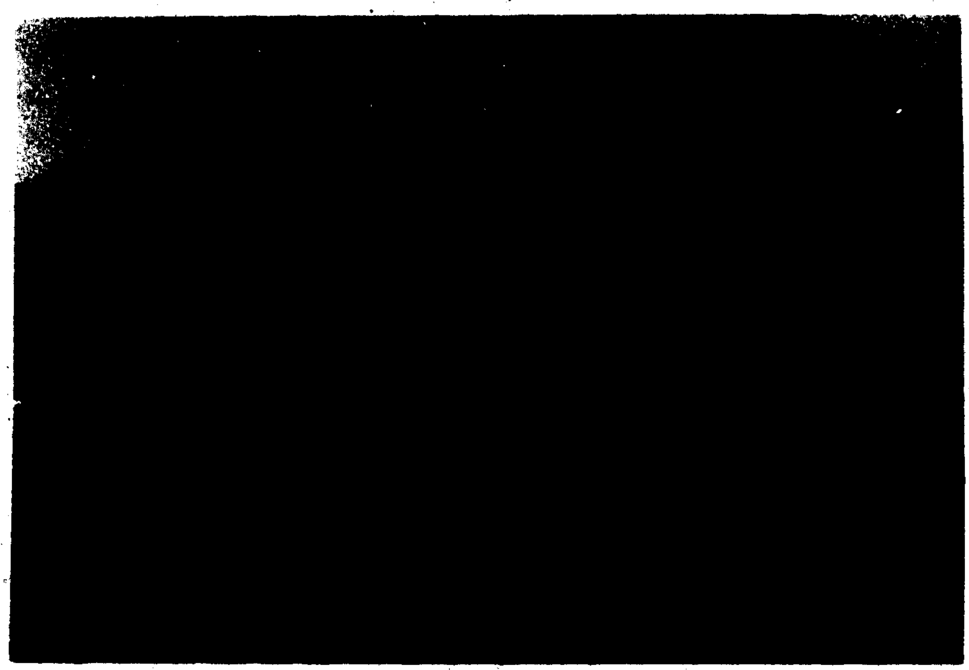

Plate 5.2 Screening Units

Plate 5.3 Double Screens Of a Screening Unit

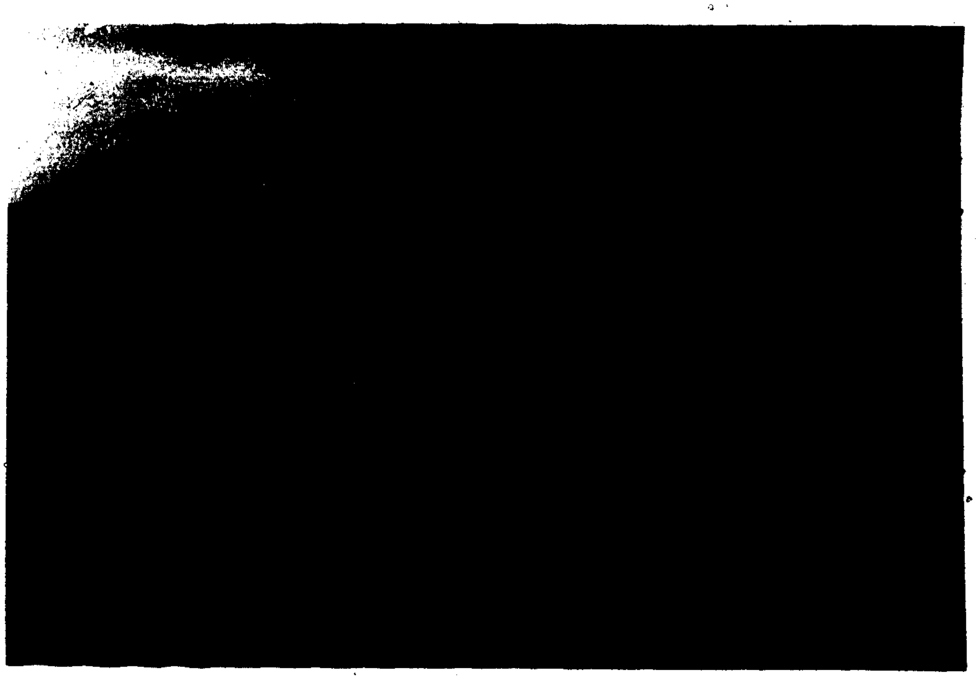

Plate 5.4 Cement Feeder, Pugmill and Control Tower

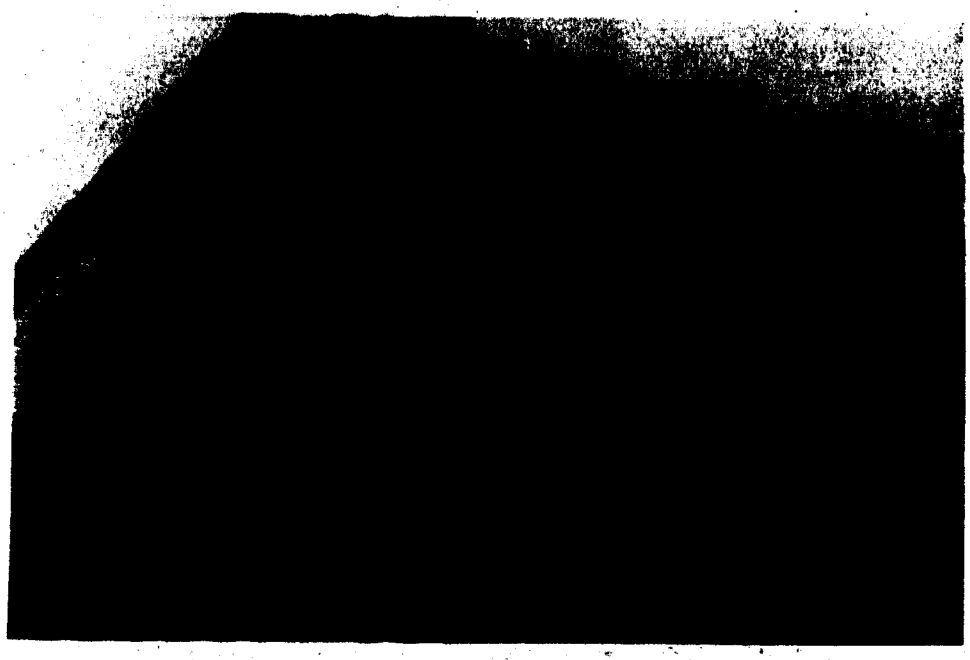

Plate 5.5 Outlet of Pugmill

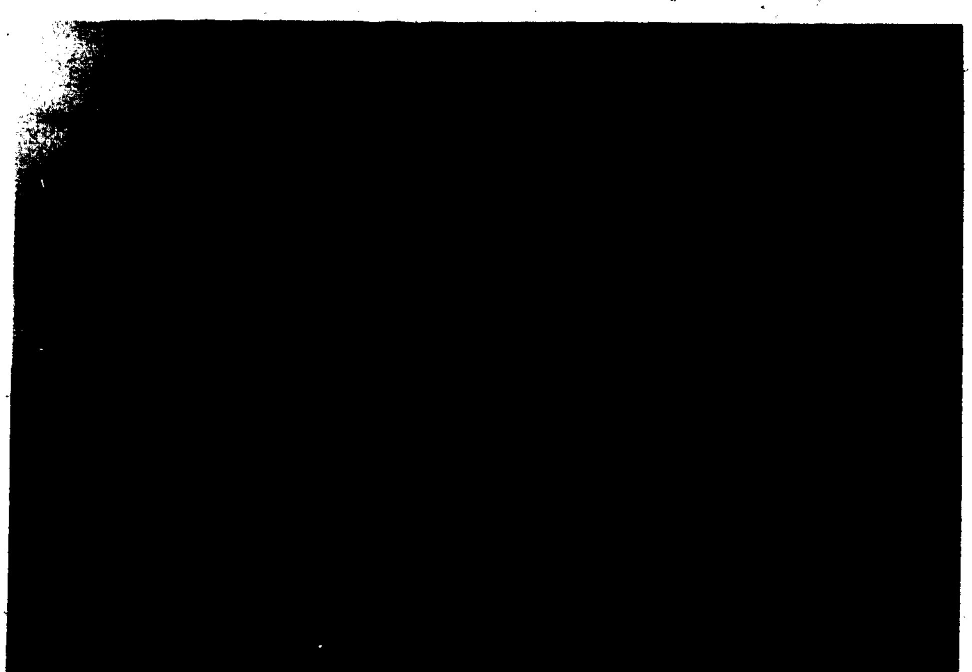

Plate 5.6 Temporary Storage Hopper and Loading of a Haul Truck

6. PERFORMANCE EVALUATION OF CEMENT-TREATED BASE PAVEMENTS IN ALBERTA

6.1 Introduction

As previously mentioned, since the beginning of the use of cement-treated base material in Alberta in 1959, over 3 000 kilometers of that type of pavement has been built in the province. Clearly, this is an indication of some satisfaction with the performance of cement treated base pavements and a recognition of the physical benefits accrued from them in Alberta. However, apart from a few instances where data presented in studies aimed at deriving performance prediction models give some insight into the field performance of cement-treated base pavements, not much published information is available on actual performance evaluations under the various traffic and environmental conditions in the province.

Consequently, an evaluation of typical cement-treated base pavements was conducted in order to determine whether the mixture and structural design criteria and construction procedures used have provided satisfactory pavements; and whether performance evaluations using presently available methods can provide adequate data for making effective rehabilitation decisions. Performance was evaluated on the basis of field measurements of Benkleman beam rebound, riding comfort index and visual condition rating; and the serviceability trends in cement-treated base pavements in the province were investigated.

6.2 Evaluation Procedure

Over the years, Alberta Transportation has collected and compiled data on the various performance indicators, together with information such as year of construction, structural composition, traffic, subgrade soil type and rehabilitation measures for the primary highway network in the province. This data has been computerized by Alberta Research Council to give a comprehensive and easily retrievable data base.

In this evaluation, data on some typical cement-treated base (CTB) pavements from the computerized data bank were critically examined and performance assessed on the basis of trends in strength, roughness and surface distress. Factors responsible for these trends, and consequently for performance losses in CTB pavements in the province were then examined.

In the past, there have also been a number of studies directed mainly at deriving performance prediction models, which give some information on the performance of CTB pavements in Alberta. Chief among them are studies by Shields and Hutchinson (1961), Shield *et al.* (1975) and Karan *et al.* (1983). Findings from these studies and the evaluation of the typical pavements were combined to give an overall account of the performance of CTB pavements in the province.

6.3 Performance Indicators

Benkleman beam rebound, ∂, riding comfort index, RCI, and visual condition rating, VCR, are the performance indicators which, as part of the Alberta Transportation data base, were used in the assessment of the strength, roughness and surface distress of pavements, respectively.

Benkleman Beam Rebound

Measurements of ∂ for the CTB pavements examined have been obtained every three years since 1971 using trucks with rear axle loads of 8 160 kg (18 kips) and tire pressures of 0.55 MPa (80 psi). As Karan *et al.* (1983) report, tests are usually conducted at a rate of 10 per mile, but are increased to 26 tests per mile for sections approaching terminal serviceability. Data on the mean Benkleman beam rebound, ∂, and the maximum probable rebound, $(\partial + 2\sigma)$, are available in the data base for the pavements studied.

Riding Comfort Index

Attempts to take RCI measurements for the typical pavements started in 1969 with Portland Cement Association (PCA) car roadmeters. Except for a few lapses of a year or two in some instances, most of the pavements have RCI measurements taken on a yearly basis, and thus the RCI data represents the most comprehensive of the performance data on the pavements. The pavements are rated on a scale of 0 to 10, representing the worst and best ride qualities respectively, with a value of 5.0 considered as the minimum below which rehabilitation measures are necessary.

Visual Condition Rating

VCR is a measurement of the structural deterioration of a pavement as perceived from surface features such as pavement fracture, distortion, disintegration and visible maintenance works. Rated on a scale of 0 to 100, representing the worst and best surface distress conditions respectively, a VCR of below 35 is considered to indicate an immediate need for rehabilitation measures, while a VCR between 35 and 45 indicates such measures will be required within a year or so (Shields *et al.*, 1975). Measurements were started in 1975 but have not been on a regular basis. As a result VCR data on the CTB pavements examined were minimal.

6.4 Selection and Features of Typical Pavements

As part of the evaluation procedure, representative projects were selected for investigation for specific information on performance. Contract documents of these selected projects were searched for pertinent information on mixture design, structural dimensions, construction and other features which might have influenced performance.

6.4.1 Criteria for Selection

In an attempt to develop performance prediction models, Karan *et al*. (1983) determined that the Benkleman beam rebound, RCI and VCR of CTB pavements in the province are not significantly affected by the climate of an area. Also, the influence of subgrade type, mix design and construction practices are considerably minimal, if the correct subgrade preparation and mix design and construction procedures are followed. On the other hand, Shields *et al*. (1975) point out that performance is appreciably affected by the load intensity on a road.

Consequently, two representative projects from the primary highway network of the province were selected for investigation. One project was selected for each of the two principal traffic-load categories recognized in the province during the design of CTB pavements (cf. section 4.4.2, *ante*). Project 12:12 and Project 2:28, were selected to represent the light-trafficked roads and heavy-trafficked roads respectively.

6.4.2 Project Features

A 20.97-km soil-cement section of Project 12:12 constructed in 1966 was one of the representative sections selected for investigation. This 2-lane road with 3-m (10-ft) shoulders comprises of a 150-mm (6-in.) soil-cement base course and a 50-mm (2-in.) ASBC, overlaid with 50 mm (2 in) of ACP in 1967 to complete construction. A part of this soil-cement section carried an average of 29 ESALs per day, while the rest carried 44 ESALs per day in the year it was opened to traffic. By 1985, these had increased to approximately 220 and 360 ESALs per day, respectively.

The other soil-cement pavement examined is a section of Project 2:28, a 4-lane divided highway with 1.8-m (6-ft) and 3-m (10-ft) inner and outer shoulders respectively. Built in 1965, this pavement comprises of 200 mm (8 in) of a soil-cement base course, 50 mm (2 in) ASBC, and 100 mm (4 in) ACP. An 8.07-km section of the total 13.77-km road was surfaced with ACP one year after construction, while the rest was overlaid with ACP in the following year. As part of the major highway between the two population centers of Edmonton and Calgary in Alberta, this project which carried an average of 300 ESALs per day in its first year, was carrying as much as 1 500 ESALs per day in 1985. This translated into an annual average daily traffic (AADT) of about 10 000 in 1985.

Both Project 12:12 and Project 2:28 were constructed with plant-mixed CTB material from soils identified as SP sands under the unified classification system, i.e. A-3 sands under the AASHTO classification system, using Type 10 (CSA) portland cement. Cement contents ranged from 6 to 7 percent. Subsoils for both projects were mainly uniform sands, and medium to low plasticity sandy clays and clays.

6.5 Performance Trends of Selected Projects

As a first step in the evaluation of the performance of the typical CTB pavements, $\bar{d}$, RCI and VCR values up to the time of first resurfacing were plotted against time after construction[17]; this in recognition of the fact that $\bar{d}$, VCR and RCI, respectively, show the influence of loading, environmental conditions, and the combined effects of loading and environmental factors on performance.

<u>Benkleman Beam Rebound</u>

Two distinct trends were discernible from the graphs of $\bar{d}$ against time after construction. As Figure 6.1 shows, for the light-trafficked road, Project 12:12, an initial increase in $\bar{d}$ with time, is followed by a decrease in $\bar{d}$ after peaking. On the other hand, in the case of the heavy-trafficked road, Project 2:28, Figure 6.2 shows that $\bar{d}$ remains constant for sometime, then increases rapidly to failure

It should be mentioned that the $\bar{d}$ measurements used in plotting the graphs were taken during the months of June to October inclusive, and according to Shields and Dacyszyn (1965) represent the critical values for CTB pavements in the province. However, values measured at pavement temperatures below 10 C were omitted, as in most cases they turned out to be erratic, probably as a result of the

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[17] Similar trends are obtained when $\bar{d}$, RCI, and VCR are plotted against the cumulative ESALs.

pavements starting to show the effects of low temperatures on them.

Riding Comfort Index

RCI is plotted against time after construction in Figure 6.3 and Figure 6.4 for Project 12:12 and Project 2:28, respectively. In both cases there is a general decreasing trend in RCI with increase in time after construction. However, while in the case of Project 2:28 (Figure 6.4) the individual data points closely follow a systematic trend, this is not the case for Project 12:12 (Figure 6.3). In the latter instance, the data points are more or less spread about a general trend line.

Visual Condition Rating

From the visual condition rating data available on Project 12:12 and Project 2:28, it is evident that VCR generally decreases with increase in time after construction. Figure 6.5 and Figure 6.6 show this decreasing trend for Project 12:12 and Project 2:28 respectively. Attention is drawn to the fact that minimum data points (two in the case of Project 2:28) were in general available for the projects.

6.6 Performance of CTB Pavements

The trends exhibited by the soil-cement sections of Project 12:12 and Project 2:28 are in general representative of the performance of CTB pavements in the province. Differences in factors such as traffic loading and environmental characteristics may, however, result in the rate of change in the individual performance indicators varying from project to project; and sometimes for specific projects, from section to section.

6.6.1 Performance Losses

By and large, the performance of CTB pavements in the province deteriorate with use due to a combination of factors. With age, the structural and functional capabilities of a pavement as detected from measurements of the various performance indicators decreases until major rehabilitive measures, often in the form of overlays, are needed to restore the pavement to an acceptable level of service.

In Alberta, as was also reported by Shields and Hutchinson (1961), transverse and longitudinal shrinkage cracks usually occur in CTB pavements three to four days after completion of construction. The cracks, which are mostly transverse vertical cracks, often appear even without appreciable traffic loading (Redus, 1958). And either by simple overstressing (Shields *et al*., 1975), or fatigue effects (Pretorius and Monismith, 1972), or both, traffic loading causes longitudinal vertical cracking usually along wheel paths and at construction joints between adjacent spreads (Shields and Hutchinson, 1961).

These different types of cracks normally extend through the base course and with time reflect through the ASBC layer and, subsequetly, through the ACP layer usually placed one construction season after the initial completion of construction of the base courses. Edge and corner loading of the cracks, and the possible intrusion of water to the subgrade with the resultant softening and consequent reduction in subgrade support, results in further disintegration, rutting and loss in strength of the pavements.

Not unlike other materials with cement as a constituent however, the strength of soil-cement base courses increase with time due to further curing. Also, although opinions differ, it is conceivable that autogenous healing of cracks; frictional resistance along crack faces; interlocking between pieces of the cracked base course; and the compacting action of traffic; possibly contribute to an increase in strength, if not at least a retention of some of the initial strength.

Thus, there se[illegible] balancing of strength gains and losses in CTB p[illegible] service. The magnitudes of these strengths are apparently dependent on such variable factors as mix design, structural thicknesses, efficiency of construction and traffic intensity; with their effects so intertwined that the contributions of each to the total strength are not readily discernible. This balancing in strengths it appears, is responsible for the variable trends in the performance of CTB pavements in Alberta as is discussed in the subsequent sections.

Pavement Structural Strength

Three types of trends in the strength of CTB pavements as measured by the Benkleman beam rebound, $\bar{d}$, are predominant in the province.

In the first type of trend, $\bar{d}$ remains almost constant until about two to four years prior to terminal serviceability when it increases at a rapid rate to failure. This trend, an example of which is shown in Figure 6.2 for the (moderately) heavy-trafficked Project 2:28, seems to be associated to pavements with high traffic intensities. Either as a result of the compacting action of high intensity traffic, or the thicker base courses (200 mm) used thereof, strength gains in such pavements are able to counterbalance strength losses, and consequently $\bar{d}$ or the strength is initially constant. Increased strength losses, probably due to further pavement disintegration, then sets in and $\bar{d}$ decreases until pavement failure. Data obtained by Shields *et al*. (1975) and presented in Figure 6.7 for roads with a high traffic intensity (i.e. with a daily traffic factor above 50) show similar results. Also, Lister (1972b) in a previous report on tests by the Transportation and Road

Research Laboratory on soil-cement pavements with sands of a similar gradation as those used in Alberta, report that seasonal and annual rebounds varied only slightly until critical conditions occurred for the pavements under relatively intense heavy commercial traffic.

A steady decrease in strength with time, i.e. a steady increase in ḋ, is the second type of trend in CTB pavements. As data obtained by Shields *et al.* (1975) show in Figure 6.7, this trend is apparently associated to pavements with low traffic intensities (i.e. those with a daily traffic factor below 50). This trend is probably the result of strength gains due to the compacting action of traffic and of the 150 mm thick base course not being adequate to counteract strength losses.

A low traffic intensity is also associated with the third type of trend. An initial decrease in strength represented by a steady increase in ḋ, is subsequently followed by an increase in strength manifested in a decreasing ḋ. An example of this trend is shown in Figure 6.8. Probably due to certain environmental changes and the minimal effects of the very low traffic intensities, it appears some CTB pavements gain strengths later in service life which sometimes exceed the losses in strength. Consequently, ḋ remains constant or decreases. Shields *et al.* (1975) obtained similar results for a CTB pavement section but reported it as an anomaly. However, the findings of this study coupled with results of an evaluation by Wang *et al*. (1972) indicates such trends in strength are normal for some CTB pavements.

Pavement Roughness

In general, CTB pavements become rougher to ride with time and the riding comfort index, RCI, decreases. This is mainly due to cracking and the subsequent disintegration of pavements, as well as the various factors responsible for performance losses discussed previously in this section. However, while in some cases the RCI decreases in a systematic trend with predictable RCI values as in Figure 6.4, in most instances a general decreasing trend is interspersed with appreciable increases over short periods as in Figure 6.3. There is also a common feature of CTB pavements otherwise determined to be in need of major repair in the form of overlays, having quite satisfactory ride qualities well above minimum standards (Shields *et al*., 1975).

The higher than expected RCI values and the increases in ride quality over short periods of time, may be due to the strength gains discussed previously; and possibly to a rearrangement of the cracked up pieces of the base course under the smoothing action of traffic, to give a base course which closely resembles a well-compacted conventional crushed-rock type base course.

<u>Pavement Surface Distress</u>

For most of the CTB pavements in the province, not much data has been collected on surface distress as measured by the visual condition rating, VCR. However, the minimal data available suggests that surface distress increases with time and consequently VCR decreases with time. Typically, cracking, surface deformations such as rutting along wheel paths, and visible maintenance works in the form of patching and mostly crack filling, all increase with the age of a CTB pavement. These increases are also apparently due to the factors previously discussed, which generally increase performance losses in CTB pavements.

6.7 Predicting Performance

The ability to predict performance is essential to the efficiency and planning of rehabilitation measures for a network of pavements. In section 6.2, *ante*, mention was made of previous attempts by Shields and Hutchinson (1961), Shields *et al*. (1975) and Karan *et al*. (1983) to develop methods for predicting the performance of CTB pavements in Alberta.

In their attempt, Shields and Hutchinson (1961) successfully determined an analysis of surface deflection bowls obtained from Benkleman beam measurements, could give an indication of the contribution of both the subgrade and combined base-and-surface layers of a CTB pavement to the total structural strength. However, as a result of limited field data at the time, rational correlations between surface deflection bowl measurements and field performance features were not possible. Such correlations would have made performance prediction viable.

In subsequent work, Shields *et al.* (1975), from a plot of Benkleman beam rebound, $\bar{d}$, against cumulative ESALs, were able to define a "critical mean rebound limit" for CTB pavements (Figure 6.7). However, attempts to develop mathematical models for extrapolating the present cumulative ESAL on a pavement to a terminal value, on the basis of $\bar{d}$ measurements and the critical rebound line, and therefore determine the useful service life of pavements, failed as a result of inadequate field data. By iteration though, Shields *et al.* developed a procedure based on graphical solutions for predicting the service lives of CTB pavements. The procedure did not, however, take into account other aspects of performance such as pavement roughness and surface distress, measured by the RCI and VCR respectively; and is rather lengthy. Consequently, it is generally considered more as an introduction of an approach to the performance management of CTB pavements in Alberta, than an effective performance prediction procedure.

According to Christison (1986), this study by Shields *et al.* (1975) was fundamental to the development of the most recent performance prediction method presented by Karan *et al.* (1983). Karan *et al.* developed three recursive mathematical models for predicting the Benkleman beam rebound, ∂, riding comfort index, RCI, and visual condition rating, VCR. The ∂ and VCR, transformed into a structural adequacy index, SAI, and a visual condition index, VCI, respectively, are together with the RCI combined into a single parameter called the pavement quality index, PQI, by use of a model. This PQI, a number on a scale of 0 to 10, expresses the overall quality of a pavement, and provides a basis for comparison of the performance and rehabilitation needs of pavements.

The three models developed by Karan *et al.*, represent a culmination of the work on performance prediction for CTB pavements in the province at present. Consequently, as part of this study, an attempt was made to determine their effectiveness by applying them to the selected projects investigated.

6.7.1 The Performance Prediction Models

The three performance prediction models developed by Karan *et al.* are as follows[18]:

The equation for predicting Benkleman beam rebound, ∂, is given by:

$$LOG_e \partial = 0.068\,84 + 0.926\,38 \times LOG_e \partial_B + 0.115\,44 \times (AGE + 1)/(AGE_B + 1) + 0.025\,14 \times LOG_e \text{ cumulative annual average daily ESALs} \quad (6.1)$$

---

[18] Equations (6.1) and (6.3) are corrected versions of the equations presented by Karan *et al.*, which were misprinted in the original publication; and were obtained from co-author Christison (1986) through personal communication.

where d_B is the previous Benkleman beam deflection at age AGE_B and AGE is the present age of the pavement[1].

The equation for predicting the riding comfort index, RCI, is given by:

$$RCI = -4.288 + 5.802 \times LOG_e\ RCI_B - 0.174\ 4 \times \Delta AGE \qquad (6.2)$$

where

RCI_B = previous RCI, and

ΔAGE = 4 years.

The equation for predicting the visual conditon rating, VCR, is given by:

$$VCR = 33.094 + 0.006\ 67 \times VCR_B^2 - 1.252\ 8 \times LOG_e\ (\Delta AGE + 1) \qquad (6.3)$$

where

VCR_B = previous visual condition rating, and

ΔAGE = 4 years.

6.7.2 Comparison of Measured and Predicted Performance Parameters

The models presented above were applied to sections of the projects investigated in this study, and the predicted d, RCI and VCR values compared to actual field measurements. In general, the average of two consecutive measurements of a parameter was used as the initial value for calculating the

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[1] The product of the cumulative annual average daily ESALs and the yearly use factor of 300 days in the province gives the cumulative ESALs carried on a pavement to the present.

predicted parameter values from the models. Where the two consecutive measurements were spaced more than 2 years apart, however, the first measurement was used, provided it was not clearly an outlier, in which case the subsequent measurement was used in the calculation, and so on.

Benkleman Beam Rebound

In Figures 6.8 and 6.9, plots of predicted $\bar{d}$ as computed from Equation (6.1), are compared to field measurements of $\bar{d}$ for sections of Projects 12:12 and 2:28 respectively. For the available data, it appears Equation (6.1) does not reliably predict $\bar{d}$. An inspection of similar plots for other sections of the pavements gave the same indication. Evidently, additional factors other than traffic load and age as indicated by Equation (6.1), affect $\bar{d}$ and need to be included in the model.

As was pointed out in section 6.6.1, *ante*, the combined effects of traffic, age, curing, thickness of the base course, unequal subgrade support, and variables associated with the design and construction of CTB pavements, influence structural capacity in at least three different ways. In fact, Shields *et al.* (1975) in their work on CTB pavements, identified two distinct trends in the Benkleman beam rebound, $\bar{d}$, although none of the pavements they examined had carried more than 10^6 cumulative ESALs during their useful service lives. Not unexpectedly, therefore, the single model developed by Karan *et al.* (1983) is not adequate for predicting $\bar{d}$ for all CTB pavements, some of which have carried over 2×10^6 cumulative ESALs during their useful service lives. Other individual factors, and their interactions, which may be identified as having an effect on structural capacity, need to be taken into account in future models for predicting $\bar{d}$.

Riding Comfort Index

The model developed for predicting RCI, Equation (6.2), was used to calculate future RCI values for sections of Projects 12:12 and 2:28, and the predicted RCI values compared to field values as in Figures 6.10 and 6.11, respectively. According to the comparisons, the model developed by Karan *et al*. (1983) reasonably predicts the RCI for the sections investigated. Similar results were obtained for other sections of both Project 12:12 and Project 2:28, but there were also some sections for which the model did not in anyway compare favorably with measured values. Furthermore, as shown by Figures 6.10 and 6.11, there is a tendency for the field RCI values to lie above the predicted curve, i.e. the model underestimates the RCI. It is suggested that the absence of a factor or factors in the model, which take into account the positive contribution of the base course curing and/or the compacting action of traffic to the RCI (cf. section 6.6.1, *ante*), is responsible for this. In most cases, however, the magnitude of this underestimation is such that, if extrapolation of the predicted RCI curve were the sole criteria for determining the useful service life, the resultant error would be at most 4 years, which is comparatively minimal.

Visual Condition Rating

A major problem encountered in the attempt to compare predicted values of VCR from Equation (6.3) to actual field values was a lack of data in most instances. For example, all the sections of Project 2:28 examined had only two measurements of VCR obtained during their entire useful service lives. As a result meaningful comparisons were not always possible. Comparison of predicted VCR values to actual field measurements for different sections of Project 12:12 was the only alternative available.

Figures 6.12 and 6.13, shows predicted VCR values plotted together with actual field values for two sections of Project 12:12. As Figure 6.12 illustrates, the model in one case closely predicts the VCR, although all the field VCR values fall below the predicted VCR curve. The case shown in Figure 6.13, is however more representative of the other sections of the pavement. In those instances, the model overestimates the VCR. Apparently, factors other than age, which contribute to failures visible at the surface of CTB pavements and thereby decrease VCR, need to be featured more prominently in the model. Traffic, the climate of an area, and even the not so easily quantifiable factor of the timeliness with which a particular agency repairs failures to curtail further failure, are examples of such factors.

It is apparent from the foregoing discussion that more work is necessary to arrive at improved performance prediction models for CTB pavements. The models developed by Karan *et al.* (1983), however, do represent a firm basis for the development of such models.

6.8 Serviceability

Although a majority of the CTB pavements in Alberta have useful service lives of between 10 to 15 years, an inspection of performance data indicates that the useful service life of a CTB pavement in the province can lie anywhere between 5 to 20 years. As discussed elsewhere in this chapter, a variety of factors, and possibly their interactions, tend to influence the performance of CTB pavements to varying degrees; and many a time there is no clear indication of the specific effect of a particular factor. As a result, serviceability trends vary from one CTB pavement section to another.

For example, on a particular road of virtually the same design , differences in traffic loading and/or intensity over the length, may specifically lead to appreciable variations in serviceability. For instance, as a result of traffic from another road (Highway 21:20) joining Project 12:12 at a certain point, the latter road comes to be made up of two sections on the basis of traffic intensity, as was mentioned in section 6.4.2, *ante*. Thus, while the portion of the road with increased traffic had to be resurfaced in 1980 after 13 years of service when it had carried up to about 385 000 cumulative ESALs; the rest of the road had not required any rehabilitation measures by 1985, after carrying up to approximately 414 000 cumulative ESALs in 18 years of service.

In other instances, however, the reasons for differences in serviceability are not so obvious. The performance of a CTB pavement (Highway 21:24/26) built in 1963, ostensibly to the required specifications, illustrates this case. Although the traffic intensity over the length of the road was virtually the same, and one section of the road had a useful service life of 14 years after carrying approximately 464 400 cumulative ESALs, the remaining portion had to be resurfaced after only 6 years in service in 1969, after carrying approximately 176 400 cumulative ESALs.

A look at the contract documents of this project gives an indication of the possible causes of the early failure of a section of the pavement, which were not so obvious in the first instance. As it turns out, the latter section was built with sand from an alternative source when the sand originally scheduled for use was found to be too wet. The design cement content of 5 percent used for this project is also low compared to other projects in the province. There may also have been poor quality control of certain portions of the project. Any one or a combination of these factors may have been responsible for the early failure.

Such responses in performance to individual and/or a combination of factors, and to varying degrees, seems to be a common feature of CTB pavements. Variation in the quality of the final soil-cement base material produced is likely responsible for this. General trends in performance, as described in section 6.6.1, *ante*, are however also common to CTB pavements. Cracking due to shrinkage and/or traffic loading (fatigue and/or overstressing) are the major causes of failure. If cracks are not repaired soon after they become visible, further deterioration occurs and the pavement reaches terminal serviceability in the near future.

6.9 Summary

Performance evaluation is essential to the determination of the structural and functioal capabilities of a pavement, and for helping to make the correct rehabilitation decisions for a network of pavements. In this chapter, an evaluation of CTB pavements in Alberta, on the basis of strength, roughness and surface distress has been presented. According to the study, a wide range of trends in performance are possible for CTB pavements in the province. Therefore, no sweeping statements can be made on the performance of typical CTB pavements at this stage. However, there is a general indication that, provided the proper mix and structural designs, and the best construction and maintenance procedures are followed, a useful service life of at least 15 years can be expected of CTB pavements built in the province.

It is apparent that further field investigation of closely monitored pavements and the meticulous collection and compilation of continuous performance data is necessary before adequate performance prediction methods can be developed to aid in making effective rehabilitation decisions. The methods presented in this chapter, represent the foundation on which such future work may be based on, to arrive at useful and improved performance prediction methods.

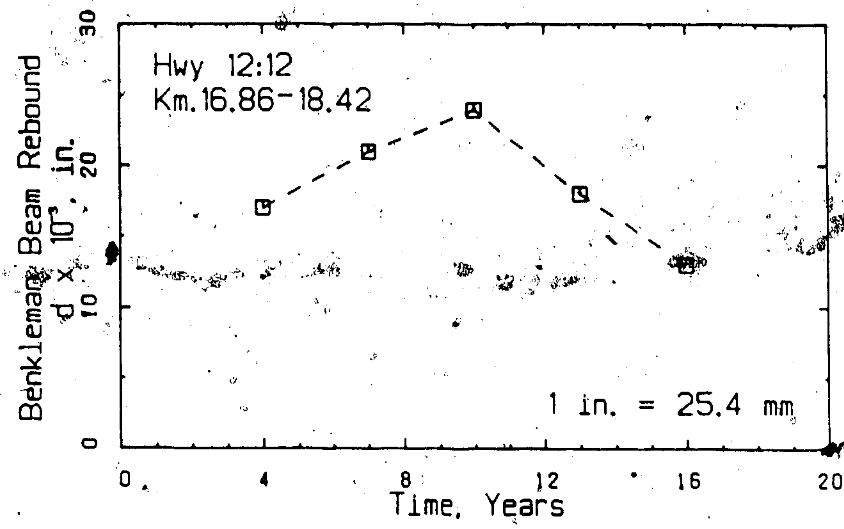

Figure 6.1 Measured Benkleman Beam Rebound versus Time -Hwy 12:12

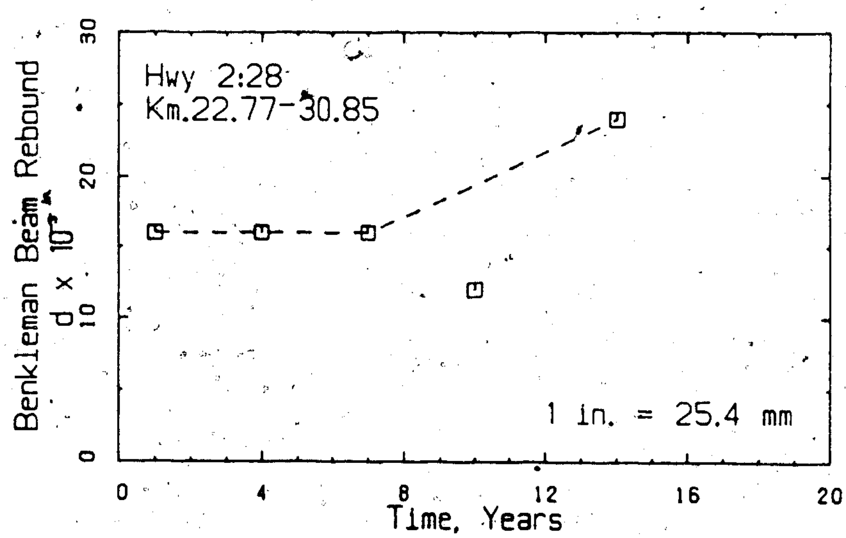

Figure 6.2 Measured Benkleman Beam Rebound versus Time -Hwy 2:28

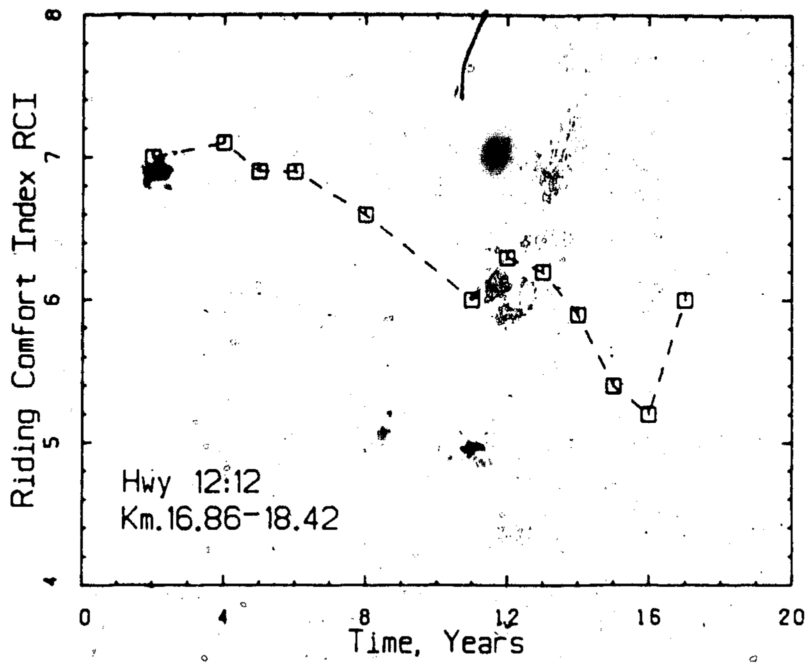

Figure 6.3 Measured Riding Comfort Index versus Time -Hwy 12:12

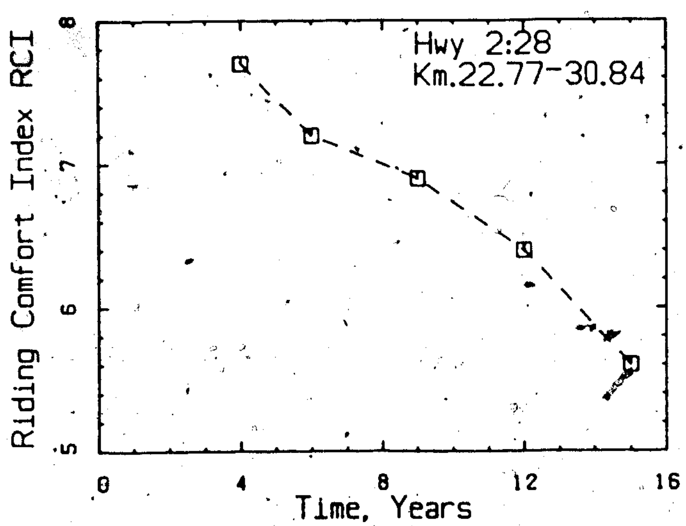

Figure 6.4 Measured Riding Comfort Index versus Time -Hwy 2:28

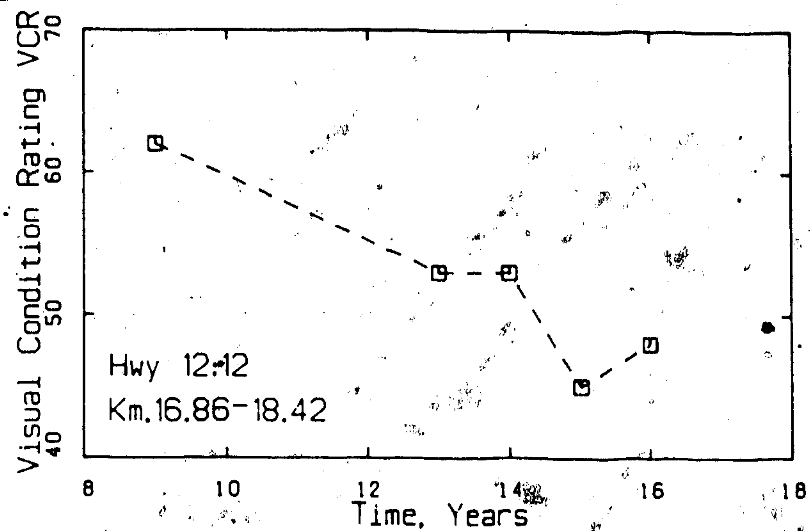

Figure 6.5 Measured Visual Condition Rating versus Time -Hwy 12:12

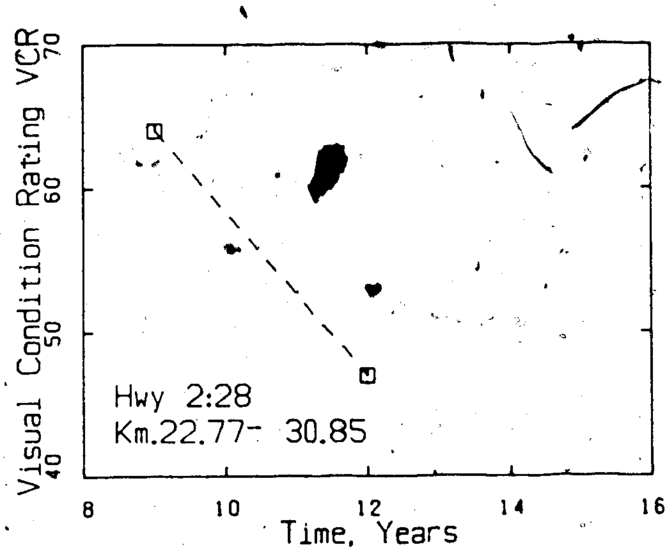

Figure 6.6 Measured Visual Condition Rating versus Time -Hwy 2:28

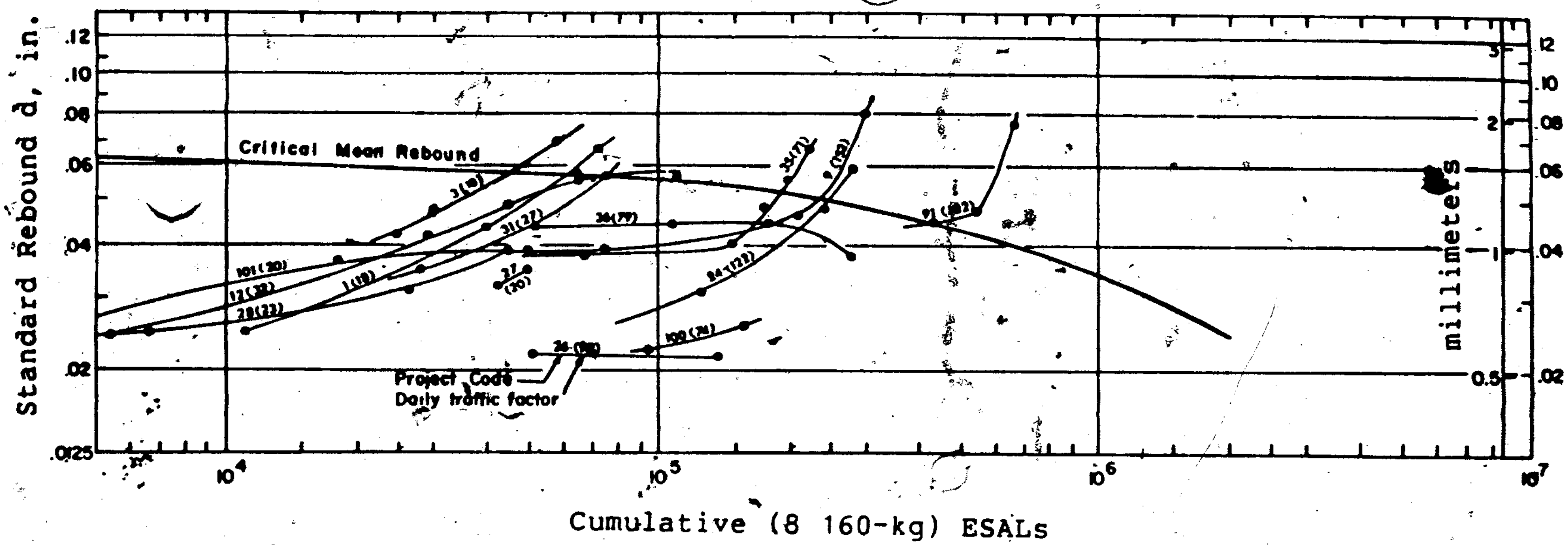

Figure 6.7 Benkleman Beam Rebound versus Cumulative (8 160-kg) Equivalent Single Axle Loads

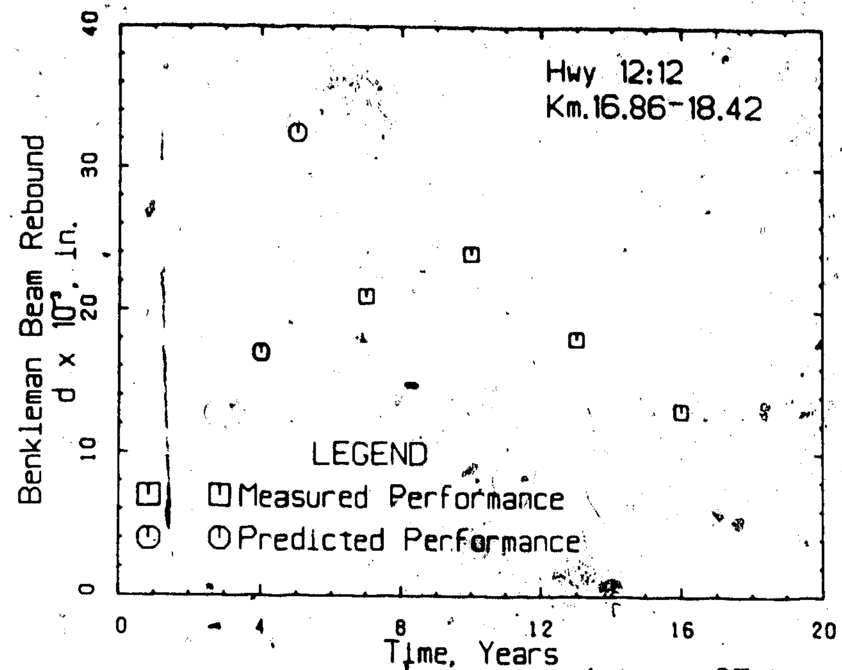

Figure 6.8 Comparison of Measured to Predicted Benkleman Beam Rebound Values -Hwy 12:12

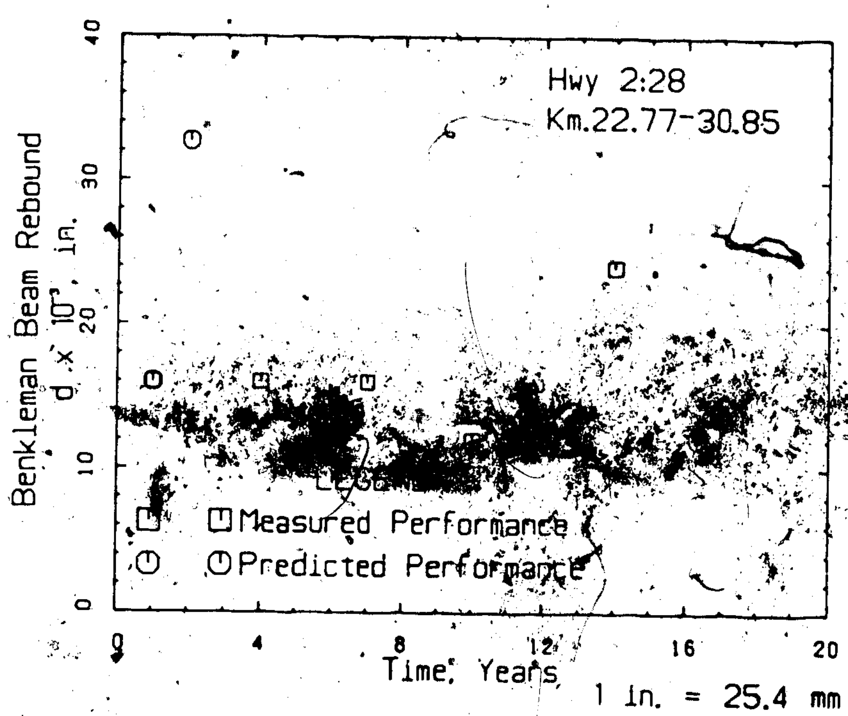

Figure 6.9 Comparison of Measured to Predicted Benkleman Beam Rebound Values -Hwy 2:28

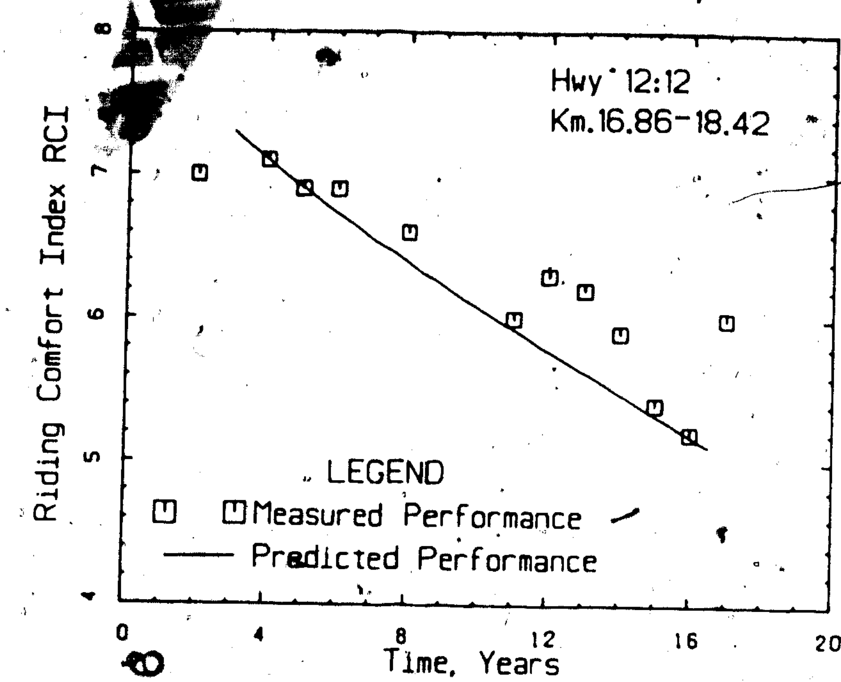

Figure 6.10 Comparison of Measured to Predicted Riding Comfort Index Values -Hwy 12:12

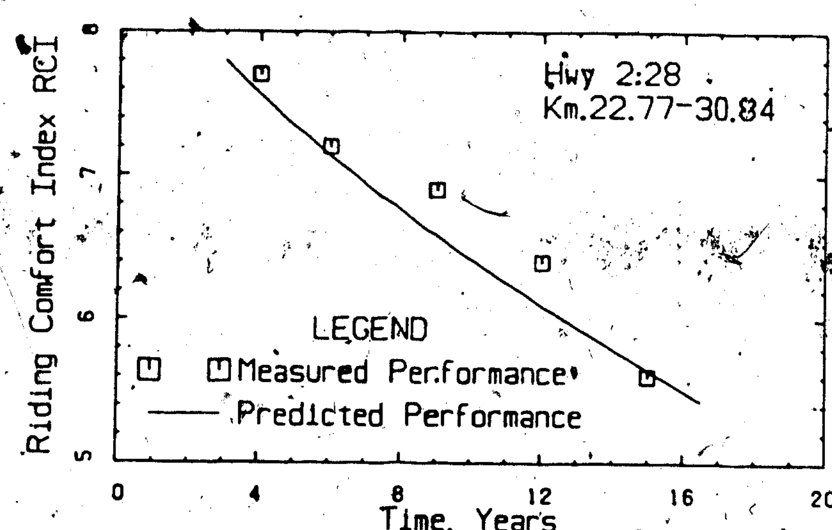

Figure 6.11 Comparison of Measured to Predicted Riding Comfort Index Values -Hwy 2:28

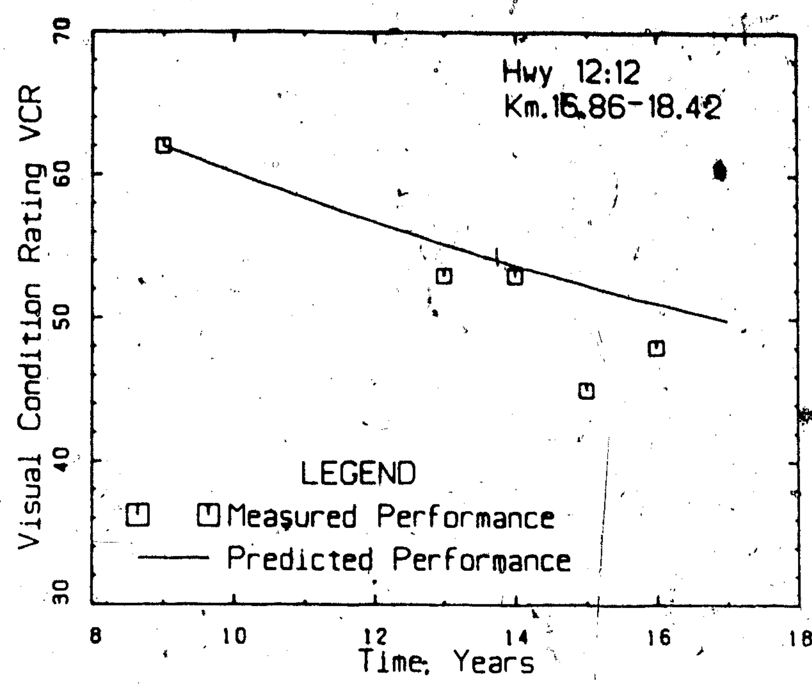

Figure 6.12 Comparison of Measured to Predicted Visual Condition Rating Values -Hwy 12:12

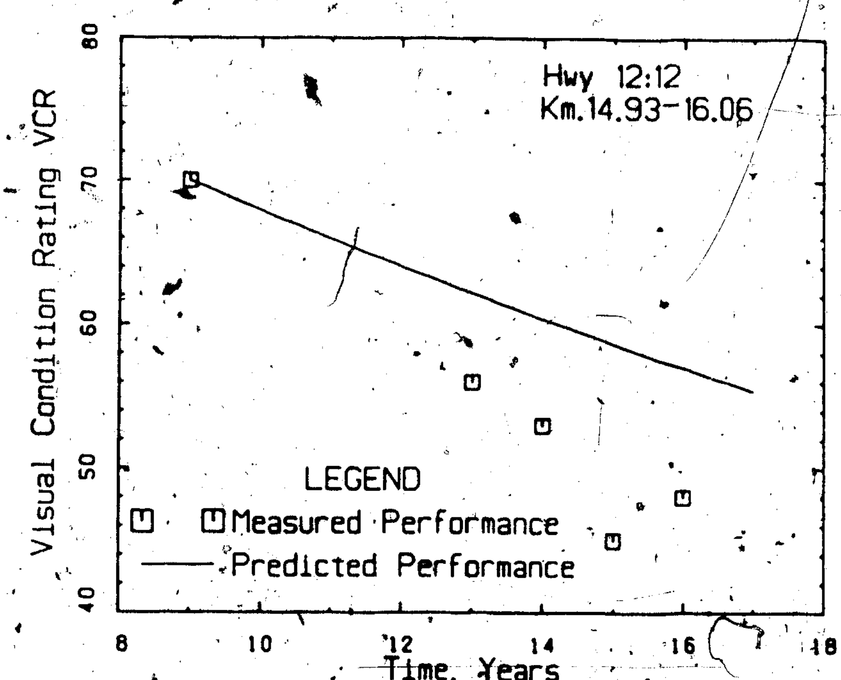

Figure 6.13 Comparison of Measured to Predicted Visual Condition Rating Values -Hwy 2:28

7. AN ECONOMIC ANALYSIS OF TYPICAL CTB PAVEMENTS IN ALBERTA

7.1 Introduction

From the preceding chapters, it is clear that the use of cement-treated base has in many instances provided an adequate alternative to the conventional granular base type of pavements in Alberta. However, in the over 25 years of its continous use, the selection of cement-treated base as a replacement for conventional granular base material for pavement construction has mainly been based on a subjective analysis, and in certain instances on a comparison of initial costs only. Evidently, an objective economic analysis would be beneficial to the pavement type selection process. It would ensure that the alternative chosen, while providing the required level of service, would also demand a reasonable amount of expenditure in its entire useful service life.

In this chapter, such an economic analysis of theoretical cement-treated base pavements and their equivalent conventional pavements with granular base course is undertaken. The intent is to compare the alternatives and highlight the economic factors which may contribute to the choice of cement-treated base pavements over other alternatives in the province. A life-cycle costing procedure is used in the analysis which compares the pavement alternatives on the basis of their present worths. This may be more complicated than simply comparing initial costs, but should provide more meaningful information.

As a result of the theoretical nature of the economic comparison; the unavailability of an efficient pavement management system to permit the accurate prediction of future maintenance and rehabilitation needs, vis-a-vis the amounts expended for such needs; and limited data in a number of instances, this study is not intended as an exhaustive discussion of the subject. It is more of a precursor to future and more detailed economic studies on the use of cement-treated base in Alberta. As part of the study, a number of non-monetary or subjective factors which may sometimes override the economic considerations are discussed.

7.2 The Pavement Types Compared

Basically in Alberta, cement-treated base (CTB) pavements have been built as replacement for conventional granular base course (GBC) pavements. Consequently, although other types of pavements such as full-depth asphalt and asphalt stabilized granular base course pavements have also been built in the province, only CTB and conventional GBC pavements are considered in this economic analysis. The analysis is undertaken for pavements of the two structural types usually identified in the province on the basis of the cumulative traffic carried, (cf. section 4.4.2, *ante*).

For the light-trafficked roads which carry up to 500 000 cumulative ESALs during their useful service lives, a conventional GBC type of pavement provided will usually comprise of 250 mm (10 in.) of a granular base course layer, 50 mm (2 in.) of a toplift of asphalt stabilized (granular) base course, ASBC, and a surfacing of 50 to 100 mm (2 to 4 in.) asphalt concrete, ACP. The 250-mm (10 in.) granular base course is replaced with a compacted soil-cement layer 150 mm (6 in.) thick to give the equivalent CTB pavement.

On the other hand, the conventional type of pavement for the heavy-trafficked roads usually consists of 300 mm (12 in.) of granular base course and 50 mm of an ASBC toplayer, all of which are surfaced with 100 mm (4 in.) of ACP. A 200 mm (8 in.) compacted soil-cement layer replaces the granular base course for the equivalent CTB pavement. For the purposes of this study, 2-lane undivided roads with lane widths of 3.75 m (12 ft) and 3.0-m (10-ft) shoulders were considered (Figure C.1).

7.3 Procedure of Economic Analysis

Life-cycle costing was selected as the method of analysis for comparing the pavement alternatives. According to a recent survey of current practice by Peterson (1985), the life-cycle costing method is the most widely used economic analysis procedure in North America. It involves a comparison of all the differential costs over a chosen analysis period incurred in the provision of the alternate pavement types. Taking into account the time value of money, the cost are transferred to a common time basis using a discounted cash flow analysis method, for comparison. By preference, the present-worth method of analysis, which again according to the survey by Peterson (1985) is the most popular method in North America, was used in this study. The economic analysis of the pavements was conducted by comparing costs per 2 lane kilometer for the equivalent theoretical CTB and GBC pavements for the two traffic categories in the province.

7.4 Factors Considered in the Analysis

A number of factors had to be considered in the analysis with regards to the input parameters. These factors are discussed in this section.

7.4.1 Analysis Period

A look at the performance histories of pavements in Alberta indicates that, GBC pavements usually have average useful service lives between 15 and 20 years. Comparatively, CTB pavements in the province have useful service lives which range from 10 to 15 years on the average. Consequently, for design purposes the time to first rehabilitation after construction, is often taken as 20 years for conventional GBC pavements and 15 years for CTB pavements. On this basis, 20 years was chosen as one analysis period for investigation. Over such a period, the CTB pavements would require rehabilitation (overlay) in the 15th year of service, while the GBC pavements would require none.

Again from existing performance data on rehabilitated pavements in the province, the life to second resurfacing of both CTB and GBC pavements is in the vicinity of 10 years. This is because, in general, the second lives of the pavements are more affected by external factors such as environmental and traffic conditions, than the characteristics of the pavement foundation materials. With this in mind and to allow a comparison involving the rehabilitation of both types of pavements, a 25-year analysis period was also selected for investigation. For a CTB pavement rehabilitated in the 15th year of service, its second life would be expended at the end of this 25-year period; and a GBC pavement overlaid in the 20th year of service would be half way through its second life at the end of the same period.

7.4.2 Cost Factors

In general, the cost factors considered in an economic analysis of pavements include engineering costs, initial construction cost, maintenance costs, rehabilitation costs, user costs and salvage value, with consideration only given to differential costs in practice. However, due to limited and sometimes unavailable data certain costs were not considered in the analysis. Following are the pertinent factors arising from a consideration of the various factors:

1. Most engineering costs, such as for the preliminary design of a roadway, the purchase of the right of way and the final geometric design of a road are the same for the different types of pavements, and are therefore not considered in this analysis. Costs associated with the location and selection of the aggregates or sand for base construction, the design of the base course and quality control may however be different for the alternatives considered. But since no reliable cost data for such parameters was available, they were also not considered in the analysis.

2. By the nature of the structural designs of the equivalent CTB and GBC pavements compared, the initial construction costs for the ASBC and ACP toplifts were omitted from the analysis, as they are the same for the equivalent pavements. Only the initial construction costs associated with the base courses needed to be included in the analysis.

3. No reliable maintenance cost data is presently available in Alberta on the different pavement types. In fact, there seems to be a general lack of maintenance related data in the province. As a result no projections of future maintenance measures and their costs could be included in the analysis. A meticulous compilation and analysis of maintenance cost data on pavements in the province is required before such costs can be include in future analysis.

4. From existing performance history data, information is available on the types, amounts and times of the first rehabilitation measures usually carried out on CTB and GBC pavements in the province. Basically, rehabilitation comprises of resurfacing with a layer of 50 to 100 mm of AC at the end of the useful service life of a pavement. For this analysis, a CTB pavement is assumed to require resurfacing with 50 mm (2 in.) of AC in the 15th year of service as rehabilitation, and therefore the resulting cost is included in the analysis.

5. User costs, or more accurately denial-of-use costs, may be significantly different for CTB and GBC pavements in the province, especially in the period immediately following completion of the base course construction. The base course of a CTB pavement requires at least 5 days to cure, and a further 5-day period before placement of the toplift of ASBC can begin. During this time traffic is restricted on the road, and depending on the nature of the road, this may bring about substantial denial-of-use cost losses. There is no need for such traffic restrictions on GBC pavements between the completion of the base course construction and the beginning of the ASBC construction, since the latter activity can immediately follow the former.

In any case, just as the user costs associated with delays during maintenance and rehabilitation works and higher vehicle operating cost due to increasing pavement roughness, data and methods for evaluating such costs are not available in the province to permit their inclusion in the analysis.

6. Salvage value comprises the cost of reusable materials in a pavement at the end of [illegible] useful service life. However, since it is difficult to project the cost of some pavement material 20 years from now after continous use, salvage value is often not considered in such analyses. On the other hand, the service life of some construction measure such as rehabilitation, may extend well past the life-cycle period being considered. In such instances, it is appropriate to include the residual value of the measure in an analysis. In this analysis, the residual value of any rehabilitation measure is taken as the product of the fraction of the useful life to next rehabilitation remaining at the end of a particular analysis period, and the cost of the initial rehabilitation.

7.4.3 Discount Rate

It is necessary to consider the time value of money in an economic analysis which involves future outlays of money. As pointed out in the RTAC (1977) manaul on pavement management, it is the practice by most agencies to use a discount rate set mainly by departmental policy, for transforming cash flows to a common basis, for an economic comparison of pavement alternatives. According to Peterson

(1985), there is a general agreement among highway engineers to use a real discount rate nominally equal to the prevailing market rate minus the current inflation rate for such transformations using constant dollars[20].

Peterson (1985) quotes the AASHTO Manual on User Benefit Analysis (Red Book) (1977), and more recently Oglesby and Hicks (1982), as recommending a minimum discount rate of 4 percent for low risk investments such as pavements; and reports of Epps and Wootan (1981) recommending a discount rate of 4 percent based on their determination that, since 1966 the real long-term rate of return on capital has been between 3.7 and 4.4 percent. The RTAC (1977) manual also reports of the use of rates up to 10 percent in Canada, with most values in the range of 4 to 8 percent. On these basis, discount rates of 2, 4 and 6 percent were selected for investigation in the economic analysis.

7.5 Life-Cycle Costing

As is evident from above, the costs to consider in the life-cycle economic comparisons varies for the two alternatives and for the analysis periods of 20 and 25 years selected for investigation. For an economic analysis over a 20-year period, the costs to consider for a CTB pavement are the initial base course construction cost, the

[20] For calculations in constant dollars, all the prices used in an economic analysis, including future prices, are taken as those at a chosen base year with no correction for inflation.

rehabilitation cost in the 15th year of service and the residual value of the rehabilitation measure at the end of the period. However, only the initial cost of the base course construction needs to be considered for the equivalent GBC pavement.

Over a 25-year analysis period, the differential costs to consider for a CTB pavement are the initial base construction cost and the cost of rehabilitation after 15 years of service. On the other hand, the initial base construction cost, the rehabilitation cost in the 20th year of service and the residual value of the rehabilitation measure at the end of the 25-year period are the costs considered for the equivalent GBC pavement for the same analysis period.

In all cases average unit prices paid by Alberta Transportation for the various items in the highway and road contracts undertaken in the province during the period from January 1 to December 31, 1985 were used in the analysis. The prices are given in Table C.1 of Appendix C.

7.5.1 Economic Formula for Net Present-Worth Cost

The basic economic formula used to determine the net present-worth of all costs and benefits over an analysis period for the equivalent theoretical CTB and GBC pavements is given by:

$$NPW = IC + \text{[illegible]}(PWF,r,n_1) - SV(PWF,r,n_2) \qquad (7.1)$$

where

NPW = net present-worth of cost per 2 lane kilometer

IC = initial construction cost of base course

RH = rehabilitation cost in n_1th year of service

SV = residual value of rehabilitation in n_2th year of service

(PWF,r,n) = single payment present-worth factor for an amount in the n_ith year, at a discount rate of r percent per year.

Using Equation (7.1), and for analysis periods of 20 and 25 years, the net present-worth costs of the equivalent pavements were calculated as outlined in Appendix C, for discount rates of 2, 4 and 6 percent, for both light-trafficked and heavy-trafficked roads.

7.5.2 Net Present-Worth Cost Comparisons

For each type of road by traffic-load classification, and for a particular analysis period, two equations giving the net present-worth costs were obtained at each discount rate, one for the theoretical CTB pavement and the other for an equivalent GBC pavement, with the variables y and z, respectively. The variable y is the haul distance to the source of the sand for construction of the CTB pavement; and similarly the variable z is the haul distance to the source of the granular base material for construction of the GBC pavement.

In each case, from the two equations, the minimum haul distance H_{GBC} (= z) beyond which a CTB pavement may be more economical than an equivalent GBC pavement, for the hypothetical situation of a zero haul distance to the source of sand material for CTB construction (y = 0), was determined. This parameter H_{GBC} was the major parameter considered in the economic comparisons of this study. For purposes of completion, however, the equal haul distance H_{EQ} to both the granular base material and sand sources, beyond which a CTB pavement may be more economic than an equivalent GBC pavement, was also determined.

As an illustration, the process of an economic comparison between a CTB pavement for a light-trafficked road and the equivalent GBC pavement it may replace, is presented in the following paragraphs for an analysis over a 20-year period, at a discount rate of 2 percent.

Net Present-Worth Cost for the Light-Trafficked Pavements

From Equation (7.1) and as outlined in Appendix C, the net present-worth cost for the theoretical CTB pavement of the light-trafficked category, over an analysis period of 20 years and at a discount rate of 2 percent, is given by:

$$NPW_{CTB} = 91\ 395.93 + 573.53y \qquad (7.2)$$

where y is the haul distance from a CTB plant situated at the sand source to the road construction site.

From Appendix C, the net present-worth cost of the equivalent GBC pavement of the light-trafficked category, over an analysis period of 20 years and at a discount rate of 2 percent, which is equal to the initial base construction cost, is given by:

$$NPW_{GBC} = 47\,865.22 + 1\,066.33z \qquad (7.3)$$

where z is the haul distance from the granular base material source to the road construction site.

At $y = 0$, that is where the sand for CTB construction is available adjacent to a project, from Equations (7.2) and (7.3), $H_{GBC} = z = 40.82$ km. Also solving the equations simultaneously, $H_{EQ} = 88.83$ km. A graphical solution shown in Figure 7.1 further illustrates what H_{GBC} in particular, and H_{EQ}, stand for. In this case, according to an economic comparison on the basis of life-cycle net present-worth costs, if sand for CTB construction can be found adjacent to the project, a haul distance of over 40.82 km to the nearest source of conventional granular base material, will likely make the CTB pavement alternate more economic in comparison to an equivalent GBC pavement.

Similarly, at discount rates of 2, 4 and 6 percent, and for the 20- and 25-year analysis periods, H_{GBC} and H_{EQ} were determined for the CTB pavements and their equivalent GBC pavements, for both the light-trafficked and heavy-trafficked roads. The summary of the results obtained are presented in Table 7.1.

7.6 Discussion of Results

It should be noted that since the preceding analysis was predicated on a number of assumptions, the specific findings from a comparison of the theoretical CTB and GBC pavements are not necessarily representative of all situations that may arise in the province. Such a comparison only gives an indication of the parameters that may influence the economic considerations of any specific case. Of particular interest is the distance H_{GBC}, the minimum haul distance to the source of granular material for a project, beyond which use of adjacent sand for CTB construction may provide a more economic pavement in comparison to the conventional GBC pavement alternate.

As indicated in Table 7.1, at discount rates of 2 to 6 percent and at prevailing highway contract prices in Alberta, it may be more economical to substitute the 250-mm (2-in.) granular base course of the light-trafficked roads with 150 mm of compacted CTB, when haul distances to the material for the GBC pavements (H_{GBC}) are in excess of 33 to 40 km, and sand for CTB is available nearby. H_{GBC} increases to between 45 and 60 km for the heavy-trafficked pavements, with their 300-mm thick granular base course as opposed to a 200-mm thick compacted CTB. The results also indicates that H_{GBC} decreases appreciably as the discount rate used increases. This, especially for the heavy-trafficked pavements, means that the viability of CTB as an economic replacement for conventional GBC pavements is further increased at higher discount rates.

From Table 7.1, it is evident that H_{GBC} is approximately the same at equal discount rates, for the analysis periods of 20 and 25 years investigated, the use of which introduces different cost factors in the economic analysis of the pavements. To an extent, this indicates the values of H_{GBC} arrived at are reasonable estimates for the specific cases investigated. Since in certain parts of Alberta, especially in the south-eastern sector, it is not uncommon to have sources of conventional base material over 50 km away from a project, while sands suitable for CTB are virtually located next to the same project, it may therefore be worthwhile on the basis of the results of this study to consider the use of CTB in place of the usual granular base course material. Undoubtedly, an economic analysis comparison such as described above, is a necessary part of the pavement type selection process.

7.7 Additional Factors Favorable to CTB Construction

It is often the case that additional factors, mostly subjective or non-monetary, may favor the construction of a CTB pavement in place of a conventional GBC pavement. Following are some of such factors which arise in the specific case of the practice in Alberta:

a. conservation of available but limited granular material resources for future considerations
b. efforts to enhance the development and stable use of an alternative type of pavement construction material; in this case soil-cement
c. efforts to keep certain local industries in business as a contribution to the general economy
d. the hierarchical importance of a particular project in the overall highway system, with reference to the desire to provide the best system under the constraints of limited resources.

Obviously, it is necessary to also consider such factors in conjuction with an objective economic evaluation in the final pavement type selection process.

7.8 Summary

The life-cycle cost analysis described above illustrates how economic comparisons may be very beneficial in the determination of which pavement type of the alternatives of CTB and conventional GBC pavements will be suitable for a particular project. Although a number of assumptions had to be made in the analysis, the theoretical pavements compared are to a considerable degree representative of the pavement types used in the province. Consequently, the findings of the analysis may to an extent be reflective of practice in Alberta. It is therefore important to include an economic analysis in the pavement type selection process in the province. For projects in known areas of abundant sand supplies, such as the south-eastern sector of the province, such methods may have far-reaching beneficial economic implications.

Table 7.1 Summary of Results of Economic Analysis

Pavement Type	Analysis Period, years	Discount Rate, r	H_{GBC}, km	H_{EQ}, km
Light-Trafficked	20	2	40.82	88.33
		4	37.48	81.11
		6	34.72	75.13
Heavy-Trafficked	20	2	58.42	145.62
		4	53.16	132.51
		6	48.81	121.66
Light-Trafficked	25	2	39.50	85.47
		4	35.77	77.41
		6	33.06	71.54
Heavy-Trafficked	25	2	56.33	140.42
		4	50.47	125.80
		6	46.20	115.15

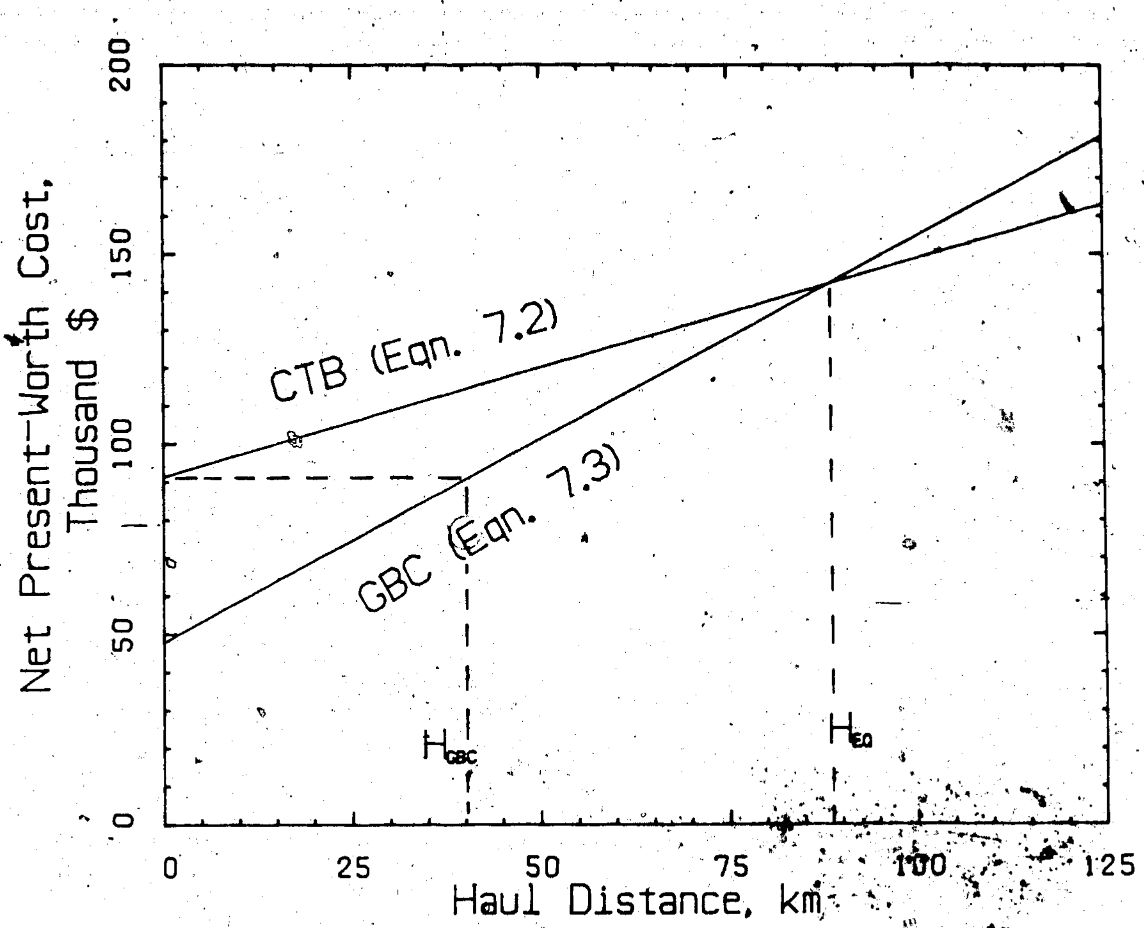

Figure 7.1 Comparison of Net Present-Worth Costs of Equivalent Theoretical CTB and GBC Pavements

8. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

This study covered the subject of cement-treated base practice for highway pavements in the province of Alberta. Specifically, the mix design method,the structural design procedure, and the construction practices undertaken by Alberta Transportation in the provision of highway flexible pavements with cement-treated base as base course were presented. In addition, the performance of typical cement-treated base pavements representing the types usually used in the province were investigated, and an economic analysis of theoretical pavements undertaken to highlight the economic factors which may influence the selection of cement-treated base over other alternatives.

These specific studies were conducted against a background of a preceding perspective literative review on the evolvement and development of cement-treated base as a pavement construction material. This review dealt with a wide range of subjects including the attributes required of the components of cement-treated base, and the engineering properties and characteristics of this construction material, composed mainly of marginal quality aggregates or soils otherwise considered unsuitable for base course construction. Also discussed were the various mix design methods, structural design procedures, construction practices and suggested remedies for the major problem of cracking often encountered with the use of cement-treated base material.

The broad literature review and synthesis of the practice in the province, presented the opportunity to review and draw valid conclusions on its use in Alberta in the last 25 years, as well as make recommendations on the future research work required to enhance its use.

8.2 Conclusion

Based of the findings of this study and the review of published literature, the following conclusions can be drawn on cement-treated base practice in Alberta:

1) Cement-treated base is a viable base course material for flexible highway pavements. It possesses the necessary engineering properties and characteristics that makes it comparable to other base course materials used in the province.
2) The current mix design procedure used in the province which comprises of freeze-thaw, wet-dry and unconfined compressive strength tests, provides adequate design parameters for use in construction. In view of the usual six months fallow period between construction seasons in the province, the lengthy time of 32 working days required for the procedure is not a limitation in major CTB practice in Alberta, especially when balanced against the higher accuracy of the mix design parameters

obtained thereof.

3) Notwithstanding the afore-mentioned advantages of the conventional CTB mix design practice in Alberta, it is still necessary to develop shortcut methods of design based on, for example, gradation and density considerations. Such methods would provide a quick means of determining cement requirements for new materials encountered on an ongoing project, or allow frequent checks to ascertain that the proper cement content is being used, for instance.

4) The structural design procedure used in Alberta, which basically comprises of the selection of one of two thicknesses, has generally provided CTB pavements with adequate load-carrying capabilities. However, there is much to be gained from a more rational approach to design which takes the engineering properties of CTB and findings from performance evaluations into consideration.

5) By the nature of the soil materials used in Alberta, namely sands from borrow pits, the construction practice involving plant-mixing and laying the CTB with automatic spreaders is most suitable. The process of dividing a day's production into units each of which have specific quality control tests conducted on them has also ensured the production of a high quality material in the province in general.

6) The evaluation of two representative projects does

not permit any sweeping statements on the performance of CTB pavements in Alberta. However, on the basis of the limited data, it is evident that some CTB pavements perform adequately at the required levels of service for periods comparable to the alternative types of pavements. Others, however, fail to meet expectations; but not because of a substandard nature of the material. Invariably, human error or the improper use of CTB is the usual source of its low quality in particular instances.

7) It is possible to develop performance prediction models for CTB pavements in the province which will reasonably predict future rehabilitation requirements. More data collection and compilation is however necessary before such improved models can be developed.

8) It is necessary to include a life-cycle cost analysis in the pavement type selection process. On the basis of the economic comparisons conducted in this study, no sweeping conclusions can be drawn on the economic advantages of CTB over the conventional granular base course it usually replaces in pavements in Alberta. However, the analysis indicates that on the basis of life-cycle cost comparisons, it is conceivable a CTB pavement may indeed be more economical to construct than a conventional granular base course pavement, where

haul distances to the granular material source are relatively high.

8.3 Recommendations

From the preceding discussions and conclusions, it is certain that there will be an increased use of CTB for pavement construction in the province in the future. Especially, with sources of the more conventional type of base course materials becoming depleted, Alberta Transportation may find it necessary to increase the use of CTB for base construction. Following are recommendations on future research work necessary to ensure a more successful use of the material:

1) Efforts to develop short-cut methods of mix design based on easily verifiable properties of the sands in the province, e.g. gradation and density considerations, should be increased.
2) Data collection and compilation on the various aspects of CTB practice should be increased appreciably. Especially, maintenance data, performance data, and cost related data should be seriously collected to ensure better appraisals of the CTB practice in Alberta.
3) Efforts should be made to increase the accuracy of existing performance prediction models. Such models would be of great help in scheduling maintenance and rehabilitation activities. Studies to determine the

nature of CTB with respect to the relationships between performance parameters such as Benkleman beam rebound, riding comfort index and visual condition rating; and traffic loading, environmental and material characteristics will be highly beneficial to the development of such better performance prediction models.

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APPENDIX A

SUMMARY OF QUALITY TESTS FOR THE DETERMINATION OF THE SUITABILITY OF SOILS FOR CTB

APPENDIX A

SUMMARY OF QUALITY TESTS FOR THE DETERMINATION OF THE SUITABILITY OF SOILS FOR CTB

A.1 Introduction

This appendix contains a summary of the preliminary tests conducted by Alberta Transportation on soils delivered for cement-treated base mix design, to determine if they are in the first place suitable for use. Depending on the results of the visual test conducted on samples of the soils, any number of these tests may be required for further proof of the suitability of the soils.

A.2 Color Test

In this test a volume calibrated bottle is filled to the 150 ml mark with the dry sand being tested. A 3 percent solution of sodium hydroxide is added to bring the total volume up to 200 ml, with the rapid agitation of the bottle to ensure complete mixing. The solution is allowed to settle with the bottle placed in semi-darkness for 24 hours. After 24 hours, the color of the solution is compared to 5 shades of color on a standard glass color plate. The colors are numbered from 1 the lightest, to 5 the darkest. A number of 3 or more for the solution is indicative of excessive organic impurities.

A.3 Silt Band Test

The silt band test is conducted with the solution used for the color test described above. During the 24-hour period the solution is allowed to settle. The heavier sand particles settle to the bottom while the finer silt and clay particles do so more slowly and settle at the top. The thickness of the fine sediment on top of the sand layer in millimeters is the silt band. This silt band is expected not to be too narrow nor too wide.

A.4 Coal Test

The coal test involves pouring a 300-gm split sample of the aggregates passing the 1 250 μm sieve size into a heavy liquid with a relative density of approximately 2.0. The mixture is stirred and the lightweight materials which float to the top skimmed with a 315 μm sieve strainer, the mixing and skimming continuing till no lightweight material floats to the top. The weight of lightweight material expressed as a percentage of the initial dry weight of the 300-gm sample is then determined.

A.5 Atteberg Limits Tests

These tests are conducted on aggregates passing the 5 000 μm sieve but retained on the 425 μm sieve to determine the liquid limit and plastic limit, and consequently the plasticity index. The tests are the same as the standard ASTM D423 and ASTM D424 tests for the liquid limit and, plastic limit and plasticity index of soils, respectively, commonly used in characterizing soils.

APPENDIX B

MIX DESIGN DATA SHEETS

Table B.1 Typical Data Sheet for Mix Design Parameters

Alberta TRANSPORTATION

SUMMARY OF TESTS ON CEMENT STABILIZED MIXTURES
ASTM D1633 ASTM D559
ASTM D560

PIT NAME MACAHONIC
LOCATION NE 29 & SE 32-50-16-4
PROJECT SR 855
DATE 85.02.15
DESIGN No. SAS A-730

GRADATION

SIEVE SIZE - µm	% PASSING
20 000	99.9
16 000	
10 000	99.8
5 000	99.5
2 500	
1 250	96
630	88
315	58
160	24.3
80	12.2
50	Est. 7
5	

COMPRESSIVE STRENGTH *

CEMENT CONTENT % by wt.	Age when tested (days): 7 CONTROL MPa	7 F-T MPa	28 CONTROL MPa	
5	2.11	2.53	2.71	
6	-	3.26	-	
7	3.38	4.24	4.62	
8	-	5.06	-	
9	4.70	6.19	6.76	

* SPECIMENS SATURATED IN WATER BEFORE TESTING

TEST DATA FROM WET-DRY AND FREEZE-THAW SPECIMENS

CEMENT CONTENT % by wt.	TOTAL SOIL-CEMENT LOSS %: WET-DRY	FREEZE-THAW
5	3.5	3.0
6	-	1.6
7	1.2	1.1
8	-	0.9
9	1.0	0.8

Recommended cement content 6.0 % by wt.

Laboratory optimum moisture content ** 10.5 % by wt.
Laboratory maximum dry density ** 1923 Kg/m³
Tests made on total sample using 20 000 µm maximum size material

** moisture density tests made during construction govern field control

SOIL CONSTANTS
LL
PI Trace (low)

UNIFIED SOIL CLASSIFICATION (MODIFIED BY PFRA – SIMILAR TO ASTM D 2487)
SMd

AASHTO SOIL CLASSIFICATION
A-3(0)

Remarks: P.C.A. L/D ratio used. After 12 cycles freeze-thaw 5% fairly good, 6 and 7% good, 8 and 9% very good condition. After 12 cycles wet-dry 5% fairly good, 7% good, 9% very good condition. Moistures at time of testing vary from 4.1 to 15.1% (average = 6.9%). Sands from all 32 test pits were combined and used in this design. Coefficient of uniformity = 4.3.

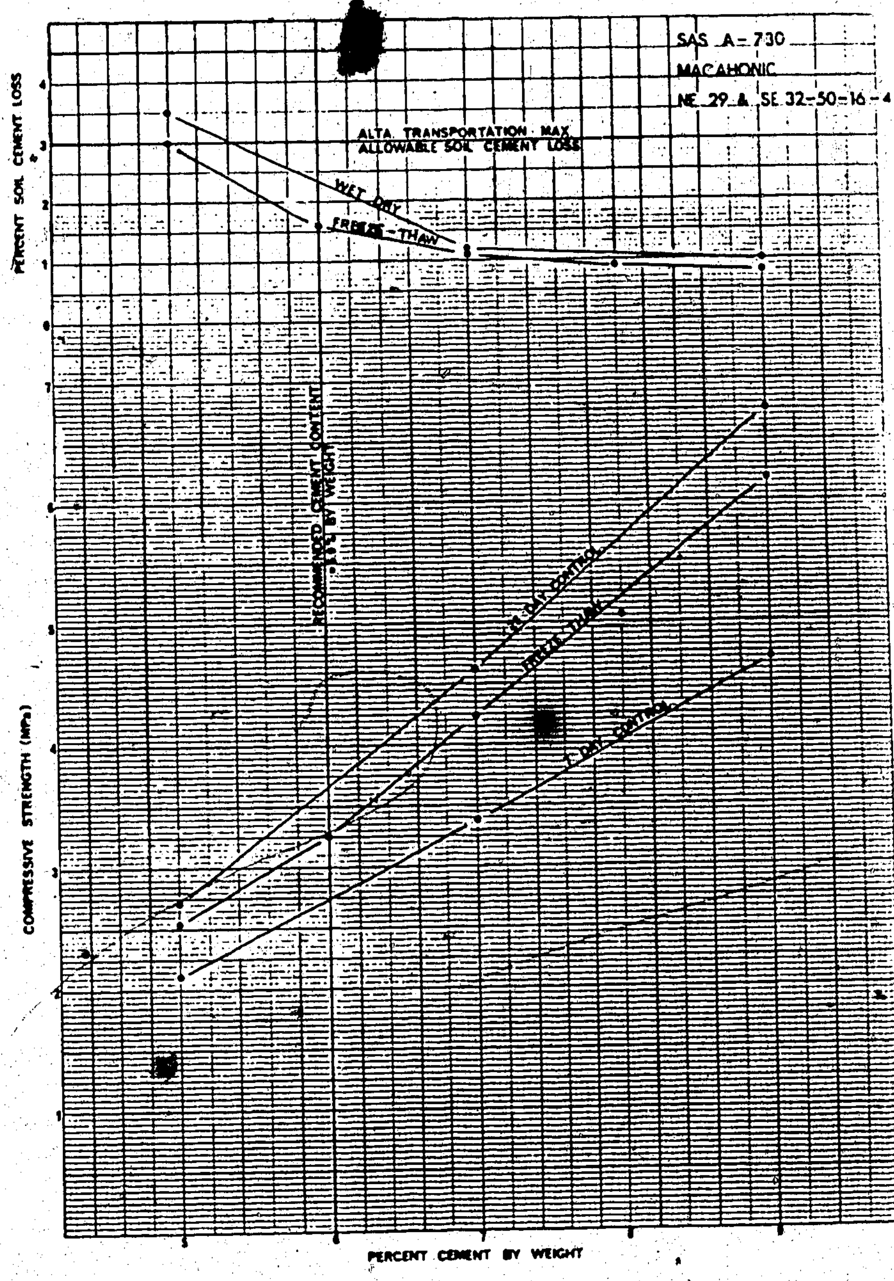

Figure B.1 Graphs of Percent Soil-Cement Loss, and Compressive Strength, against Cement Content

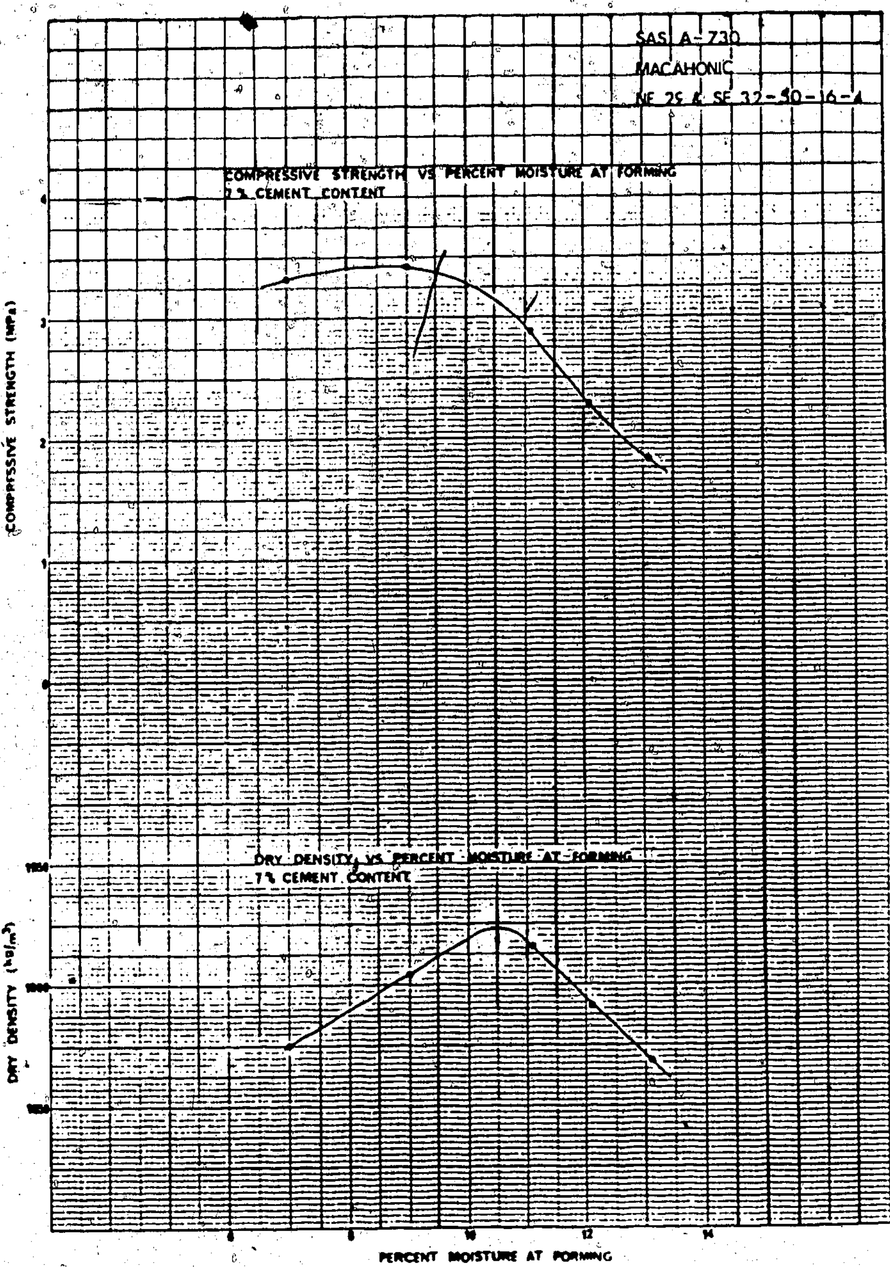

Figure B.2 Graphs of Compressive Strength, and Dry Density, against Moisture Content

APPENDIX C

DETAILS OF COST ANALYSIS OF EQUIVALENT CTB AND GBC PAVEMENTS

APPENDIX C

DETAILS OF COST ANALYSIS OF EQUIVALENT CTB AND GBC PAVEMENTS

C.1 Introduction

The details of the cost analysis of the equivalent theoretical CTB and GBC pavements, of both the light-trafficked and heavy-trafficked categories, are presented in this appendix. The pavements were compared on the basis of their life-cycle net present-worths. By the nature of the structures of these two pavement types in Alberta, the basic economic formula for the net present-worth cost is given by:

$$NPW = IC + RH(PWF,r,n_1) - SV(PWF,r,n_2) \qquad (C.1)$$

where

NPW = net present-worth of cost per 2 lane kilometer

IC = initial construction cost of base course

RH = rehabilitation cost in n_1th year of service

SV = residual value of rehabilitation in n_2th year of service

(PWF,r,n) = single payment present-worth factor for an amount in the n_ith year, at a discount rate of r percent per year.

The pavements were compared over analysis periods of 20 and 25 years for discount rates of 2, 4 and 6 percent. On the basis of existing historical performance data in the province, the CTB and GBC pavements are assumed to require rehabilitation in their 15th and 20th years of service, respectively. The light-trafficked roads are overlaid with 50 mm of ACP and the heavy-trafficked roads with 100 mm ACP at the end of their respective first useful service lives, which in both cases give the pavements an average useful service life to second resurfacing of approximately 10 years in the province.

Consequently, for an economic analysis over a period of 20 years, the costs to consider are the initial construction cost, IC, the rehabilitation cost in the 15th year of service, RH_{15}, and the residual value at the end of the analysis period, SV_{20}, for a CTB pavement; and only the initial construction cost, IC, for an equivalent GBC pavement. On the other hand, the intial construction cost, IC, and the rehabilitation cost in the 15th year of service, RH_{15}, for a CTB pavement; and the initial construction cost, IC, the rehabilitation cost in the 20th year of service, RH_{20}, and the residual value at the end of the analysis period, SV_{25}, for an equivalent GBC pavement are pertinent to the calculation of the net present-worth cost of the pavements over a 25-year analysis period.

C.2 General Data

Alberta Transportation has compiled data on the unit prices of the various items of the highway and road contracts undertaken in the province in the period from January 1 to December 31, 1985. The average unit prices of the items from this data base pertinent to CTB and GBC construction are presented in Table C.1, and are the prices used for this analysis. The specific aggregate and sand types selected are typical of practices in the province. (In fact, the aggregate and sand types chosen for this analysis, are similar to those used for the illustrative project in Chapter 5).

Following are average values assumed for some of the material parameters typical of practice in the province and used in the analysis:

1. Compacted density of CTB layer = 2.15 tonne/m^3
2. Compacted density of GBC layer = 2.33 tonne/m^3
3. Compacted density of ACP layer = 2.40 tonne/m^3
4. Cement content of CTB by wt. of dry aggregate = 8%
5. Asphalt content of ACP by wt. of dry aggregate = 6%

In Figure C.1, is a cross-sectional sketch of the pavement used for the economic analysis, where B is the thickness of the base course, P is the thickness of the ACP, and R is the thickness of the overlay placed as a pavement rehabilitation measure where necessary. All the pavements are assumed to have side slopes of 6:1, and top-widths of 13.5 m (45 ft) made up of two 3.75-m (12-ft) lanes and two 3-m (10-ft) shoulders.

C.3 Comparison of Net Present-Worth Costs of Light-Trafficked Pavements Over a 20-Year Analysis Period

In this section, the net present-worth costs of the equivalent theoretical CTB and GBC pavements, typical of a light-trafficked road, are calculated and compared over a 20-year analysis period.

C.3.1 CTB Pavement Net Present-Worth Cost

For a light-trafficked CTB pavement, B is 150 mm, P is 50 mm and the thickness of the overlay placed in the 15th year of service R is 50 mm (Figure C.1). The net present-worth cost per 2 lane kilometer for an analysis period of 20 years is calculated as follows:

Material Quantities

Wt. of compacted CTB base course = 2 340 x 2.15 = 5031 T

Wt. of cement at 8% content = 372.67 T

Wt. of 50-mm ACP overlay = 675 x 2.40 = 1 620 T

Wt. of asphalt in overlay at 6% content = 91.70 T

Initial Construction Cost

Description	Quantity	Unit Price	Amount
Cement	372.67 T	$124.00	$46 211.08
CTB from pit	5 031.00 T	4.492	22 599.25
Haul CTB BLF	5 031.00 T	0.801	4 029.83
Haul CTB Haul	5 031.00y T KM	0.114	573.53y
Fog Seal Coat	13 500 SQ M	0.024	324.00

Total IC = $73 164.16 + 573.53y

where y is the haul distance from the CTB production plant to the construction site.

Rehabilitation Cost

Description	Quantity	Unit Price	Amount
Asphalt	91.70 T	$288.00	$26 409.60
ACP from pit	1 620.00 T	10.496	17 003.52
Haul ACP BLF	1 620.00 T	0.754	1 221.48
Haul ACP Haul	1 620.00q T KM	0.112	181.44q
Tack Coat	13 500 SQ M	0.016	216.00

Total RH_{15} = 44 850.60 + 181.44q

where q is the haul distance from the ACP production plant situated at an aggregate source to the construction site.

Residual Value

From the assumptions made in section C.1, *ante*, the residual value of the CTB pavement at the end of the 20-year analysis period, SV_{20}, may be assummed to be half the cost of resurfacing undertaken in the 15th year, since the average second lives of CTB pavements in the province is about 10 years. Therefore, at the end of the analysis period,

$$\text{Total } SV_{20} = 22\,425.30 + 90.72q$$

Substituting IC, RH_{15} and SV_{20} in Equation (C.1), the net present-worth cost of the light-trafficked CTB pavement is given by:

$$NPW_{CTB/20} = [73\,164.16 + 573.50y] + [(44\,850.60 + 181.44q) \times (PWF,r,15)] - [(22\,425.30 + 90.72q) \times (PWF,r,20)] \quad (C.2)$$

@ r = 2%, (PWF,2,15) = 0.7430, (PWF,2,20) = 0.6730 and

$$NPW_{CTB/20} = 91\,395.93 + 573.53y \quad (C.3)$$

@ r = 4%, (PWF,4,15) = 0.5553, (PWF,4,20) = 0.4564 and

$$NPW_{CTB/20} = 87\,834.79 + 573.53y \quad (C.4)$$

@ r = 6%, (PWF,6,15) = 0.4173, (PWF,4,20) = 0.3118 and

$$NPW_{CTB/20} = 84\ 888.11 + 573.53y \qquad (C.5)$$

where the terms in q in Equations (C.3), (C.4) and (C.5) (73.76, 59.35 and 47.43, respectively) are by comparison negligible.

C.3.2 GBC Pavement Net Present-Worth Cost

For a light-trafficked GBC pavement, B is 250 mm and P is 50 mm (Figure C.1). No rehabilitation is required during the 20-year analysis period and the net present-worth cost per 2 lane kilometer is calculated as follows:

Material Quantities

Wt. of compacted granular base course = 4 050 x 2.33
= 9 436.50 T

Initial Construction Cost

Description	Quantity	Unit Price	Amount
GBC from pit	9 436.50 T	4.268	$40 274.98
Haul GBC BLF	9 436.50 T	0.760	7 171.74
Haul GBC Haul	9 436.50z T KM	0.113	1 066.33z
Prime Coat	13 500 SQ M	0.031	418.50

Total IC = $47 865.22 + 1 066.33z

where z is the haul distance from the GBC source to the road construction site.

Substituting IC into Equation (C.1), since RH = 0 and SV = 0, the net present-worth costs for the light-trafficked GBC pavement @ r = 2, 4 and 6 percent are all given by:

$$NPW_{GBC/20} = 47\ 865.22 + 1\ 066.33z \qquad (C.6)$$

C.3.3 Net Present-Worth Cost Comparisons

In order to permit comparison of the net present-worth costs of the equivalent theoretical pavements in the most visible manner, the minimum haul distance to the granular base course material source H_{GBC} beyond which a CTB pavement may be more economic than an equivalent GBC pavement is determined for each discount rate, for the hypothetical situation of a zero haul distance to the source of sand material for CTB construction. For comparative purposes, the equal haul distance H_{EQ} to both the granular base course material and sand sources, beyond which a CTB pavement may be more economic than an equivalent GBC pavements, is also determined.

For example, at a discount rate of 2 percent, from Equations (C.3) and (C.6), H_{GBC} (= z) is equal to 40.82 km when y = 0. Also solving the equations simultaneously, H_{EQ} (= y = z) is equal to 88.33 km. Following are H_{GBC} and H_{EQ} at the various discount rates, r, for the equivalent pavements (together with the equations used to determine the haul distances):

Equations	r	H_{GBC}	H_{EQ}
(C.3), (C.6)	2	40.83	88.33
(C.4), (C.6)	4	37.48	81.11
(C.5), (C.6)	6	34.72	75.13

C.4 Comparison of Net Present-Worth Costs of Heavy-Trafficked Pavements Over a 20-Year Analysis Period

As in the preceding section, the net present-worth costs over an analysis period of 20 years for the equivalent theoretical CTB and GBC pavements typical of heavy-trafficked roads in the province are calculated and compared in this section.

C.4.1 CTB Pavement Net Present-Worth Cost

For a heavy-trafficked CTB pavement B = 200 mm, P = 100 mm and the thickness of the overlay placed in the 15th year of service R = 100 mm (Figure C.1). The net present-worth cost per 2 lane kilometer for an analysis period of 20 years is calculated as follows:

<u>Material Quantities</u>

Wt. of compacted CTB base course = 3 300 x 2.15 = 7 095 T

Wt. of cement at 8% content = 525.56 T

Wt. of 100-mm ACP overlay = 1 350 x 2.40 = 3 240 T

Wt. of asphalt in overlay at 6% content = 183.40 T

Initial Construction Cost

Description	Quantity	Unit Price	Amount
Cement	525.56 T	$124.00	$65 169.44
CTB from pit	7 095.00 T	4.492	31 870.74
Haul CTB BLF	7 095.00 T	0.801	5 683.10
Haul CTB Haul	7 095.00y T KM	0.114	808.83y
Fog Seal Coat	13 500 SQ M	0.024	324.00

Total IC = $103 047.28 + 808.83y

where y is the haul distance from the CTB production plant to the construction site.

Rehabilitation Cost

Description	Quantity	Unit Price	Amount
Asphalt	183.40 T	$288.00	$52 819.20
ACP from pit	3 240.00 T	10.496	34 007.04
Haul ACP BLF	3 240.00 T	0.754	2 442.96
Haul ACP Haul	3 240.00q T KM	0.112	362.88q
Tack Coat	13 500 SQ M	0.016	216.00

Total RH_{15} = 89 485.20 + 362.88q

where q is the haul distance from the ACP production plant situated at an aggregate source to the construction site.

Residual Value

As in the case of the light-trafficked roads, the residual value of the heavy-trafficked pavement at the end of the 20-year analysis period, SV_{20}, may be assummed to be half the cost of the overlay construction in the 15th year, since the pavements have an average second life of about 10 years in the province. Therefore, at the end of the 20-year analysis period,

$$\text{Total } SV_{20} = 44\ 742.60 + 181.44q$$

Substituting IC, RH_{15} and SV_{20} in Equation (C.1), the net present-worth cost of the heavy-trafficked CTB pavement for an analysis period of 20 years is given by:

$$NPW_{CTB/20} = [103\ 047.28 + 808.83y] + [(89\ 485.20 + 362.88q) \times (PWF,r,15)] - [(44\ 742.60 + 181.44q) \times (PWF,r,20)] \tag{C.7}$$

@ r = 2%, (PWF,2,15) = 0.7430, (PWF,2,20) = 0.6730 and

$$NPW_{CTB/20} = 139\ 423.01 + 808.83y \tag{C.8}$$

@ r = 4%, (PWF,4,15) = 0.5553, (PWF,4,20) = 0.4564 and

$$NPW_{CTB/20} = 132\ 317.89 + 808.83y \tag{C.9}$$

@ r = 6%, (PWF,6,15) = 0.4173, (PWF,4,20) = 0.3118 and

$$NPW_{CTB/20} = 126\ 438.71 + 808.83y \tag{C.10}$$

where the terms in q in Equations (C.8), (C.9) and (C.10) (147.51, 118.70 and 94.86, respectively) are by comparison negligible.

C.4.2 GBC Pavement Net Present-Worth Cost

For a heavy-trafficked GBC pavement B = 300 mm and P = 100 mm (Figure C.1). The net present-worth cost per 2 lane kilometer over a life cycle of 20 years is calculated as follows:

Material Quantities

Wt. of compacted granular base course = 5 130 x 2.33
= 11 952.90 T

Initial Construction Cost

Description	Quantity	Unit Price	Amount
GBC from pit	11 952.90 T	4.268	$51 014.98
Haul GBC BLF	11 952.90 T	0.760	9 084.20
Haul GBC Haul	11 952.20z T KM	0.113	1 350.68z
Prime Coat	13 500 SQ M	0.031	418.50

Total IC = $60 517.68 + 1 350.68z

where z is the haul distance from the GBC source to the road construction site.

Since for a GBC pavement, RH = 0 and SV = 0 in a 20 year period, substituting IC in Equation (C.1) the net present-worth cost at r = 2, 4 and 6 percent are all given by:

$$NPW_{GBC/20} = 60\ 517.68 + 1\ 350.68z \qquad (C.11)$$

C.4.3 Net Present-Worth Cost Comparisons

The H_{GBC} and H_{EQ} for the heavy-trafficked equivalent pavements as determined from the respective equations are as follows:

Equations	r	H_{GBC}	H_{EQ}
(C.8), (C.11)	2	58.42	145.62
(C.9), (C.11)	4	53.16	132.51
(C.10), (C.11)	6	48.81	121.66

C.5 Comparison of Net Present-Worth Costs of Light-Trafficked Pavements Over a 25-Year Analysis Period

The net present-worth costs of the equivalent light-trafficked CTB and GBC pavements are calculated and compared in this section for an analysis period of 25 years.

C.5.1 CTB Pavement Net Present-Worth Cost

By inference from Equations (C.1) and (C.2), for a light-trafficked CTB pavement with B = 150 mm, P = 50 mm and R = 50 mm (Figure C.1), since $SV_{25} = 0$, the net present-worth cost over an analysis period of 25 years is given by:

$$NPW_{CTB/25} = [73\ 164.16 + 573.50y] + [(44\ 850.60 + 181.44q) \times (PWF,r,15)] \qquad (C.12)$$

@ r = 2%, (PWF,2,15) = 0.7430 and

$$NPW_{CTB/25} = 106\ 488.16 + 573.53y \qquad (C.13)$$

@ r = 4%, (PWF,4,15) = 0.5553 and

$$NPW_{CTB/20} = 98\ 069.70 + 573.53y \qquad (C.14)$$

@ r = 6%, (PWF,6,15) = 0.4173 and

$$NPW_{CTB/20} = 91\ 880.32 + 573.53y \qquad (C.15)$$

where the terms in q in Equations (C.13), (C.14) and (C.15) (134.81, 100.75 and 75.72, respectively) are by comparison negligible.

C.5.2 GBC Pavement Net Present-Worth Cost

From Equations (C.1), (C.2) and (C.6), since for the equivalent light-trafficked roads $RH_{20/GBC} = RH_{15/CTB}$ and $SV_{25/GBC} = SV_{20/CTB}$, the net present-worth cost over a 25-year period for a light-trafficked GBC pavement with B = 250 mm, P = 50mm and R = 50 mm (Figure C.1) is given by:

$$NPW_{GBC/25} = [47\ 865.22 + 1\ 066.33z] + [(44\ 850.60 + 181.44q) \times (PWF,r,20)] - [(22\ 425.30 + 90.72q) \times (PWF,r,25)] \quad (C.16)$$

@ r = 2%, (PWF,2,20) = 0.6730, (PWF,r,25) = 0.6100 and

$$NPW_{GBC/25} = 64\ 370.24 + 1\ 066.33z \quad (C.17)$$

@ r = 4%, (PWF,4,20) = 0.4564, (PWF,r,25) = 0.3751 and

$$NPW_{GBC/25} = 59\ 923.30 + 1\ 066.33z \quad (C.18)$$

@ r = 6%, (PWF,6,20) = 0.3118, (PWF,r,25) = 0.2330 and

$$NPW_{GBC/25} = 56\ 624.54 + 1\ 066.33z \quad (C.19)$$

where the terms in q in Equations (C.17), (C.18) and (C.19) (134.81, 100.75 and 75.72, respectively) are by comparison negligible.

C.5.3 Net Present-Worth Cost Comparisons

The H_{GBC} and H_{EQ} for the light-trafficked equivalent pavements as determined from the respective equations on the basis of a 20-year analysis period for the various discount rates are as follows:

Equations	r	H_{GBC}	H_{EQ}
(C.13), (C.17)	2	39.50	85.47
(C.14), (C.18)	4	35.77	77.41
(C.15), (C.19)	6	33.06	71.54

C.6 Comparison of Net Present-Worth Costs of Heavy-Trafficked Pavements Over a 25-Year Analysis Period

As in the preceding section, the net present-worth cost of the equivalent heavy-trafficked CTB and GBC pavements are determined and compared, for an analysis period of 25 years.

C.6.1 CTB Pavement Net Present-Worth Cost

From Equations (C.1) and (C.7), since $SV_{25/CTB} = 0$, the net present-worth cost of a heavy-trafficked CTB pavement with B = 200 mm, P = 100 mm and R = 100 mm (Figure C.9) is given by:

$$NPW_{CTB/25} = [103\ 047.28 + 808.83y] + [(89\ 485.20 + 362.88q) \times (PWF,r,15)] \qquad (C.20)$$

@ r = 2%, (PWF,2,15) = 0.7430 and

$$NPW_{CTB/25} = 169\ 534.78 + 808.83y \tag{C.21}$$

@ r = 4%, (PWF,4,15) = 0.5553 and

$$NPW_{CTB/25} = 152\ 738.41 + 808.83y \tag{C.22}$$

@ r = 6%, (PWF,6,15) = 0.4173 and

$$NPW_{CTB/25} = 140\ 389.45 + 808.83y \tag{C.23}$$

where the terms in q in Equations (C.21), (C.22) and (C.23) (269.62, 201.51 and 151.43, respectively) are by comparison negligible.

C.6.2 GBC Pavement Net Present-Worth Cost

For a heavy-trafficked GBC pavement, the cost of rehabilitation in the 20th year of service is assumed to be the same as the rehabilitation cost on an equivalent CTB structure after 15 years of service, and the residual value at the end of a 25-year period taken as half of this rehabilitation cost. Consequently, from Equations (C.1), (C.7) and (C.11), the net present-worth cost of the GBC pavement over a 25-year period is given by:

$$\begin{aligned} NPW_{25/GBC} = {} & [60\ 517.68 + 1\ 350.68z] + [(89\ 485.20 + 362.88q) \\ & \times (PWF,r,20)] - [(44\ 742.60 + 181.44q) \\ & \times (PWF,r,25)] \end{aligned} \tag{C.24}$$

@ r = 2%, (PWF,2,20) = 0.6730, (PWF,r,25) = 0.6100 and

$$NPW_{GBC/25} = 93\ 448.23 + 1\ 350.68z \qquad (C.25)$$

@ r = 4%, (PWF,4,20) = 0.4564, (PWF,r,25) = 0.3751 and

$$NPW_{GBC/25} = 84\ 575.78 + 1\ 350.68z \qquad (C.26)$$

@ r = 6%, (PWF,6,20) = 0.3118, (PWF,r,25) = 0.2330 and

$$NPW_{GBC/25} = 77\ 994.14 + 1\ 350.68z \qquad (C.27)$$

where the terms in q in Equations (C.25), (C.26) and (C.27) (133.54, 97.56 and 70.87, respectively) are by comparison negligible.

C.6.3 Net Present-Worth Cost Comparisons

The H_{GBC} and H_{EQ} for the heavy-trafficked equivalent pavements as determined from the respective equations for the 25-year analysis are as follows:

Equations	r	H_{GBC}	H_{EQ}
(C.21), (C.25)	2	56.33	140.42
(C.22), (C.26)	4	50.47	125.80
(C.23), (C.27)	6	46.20	115.15

Table C.1 Average Unit Prices for Some Items of Alberta Transportation CTB Highway and Roads Contracts

Item Number	Item Description	Unit	Unit Price, $ -1984	Unit Price, $ -1985
B510	GBC from pit; Desg. 2, Class 16	tonne	3.632	4.268
B570	Haul of GBC; BLF	tonne	0.743	0.760
B571	Haul of GBC; Haul	t km	0.114	0.113
B660	CTB from pit, Desg. 7, Class 40	tonne	3.687	4.492
B670	Haul CTB; BLF	tonne	0.827	0.801
B671	Haul CTB; Haul	t km	0.121	0.114
B682	Broom and Apply Fog Coat Seal	sq m	0.030	0.024
B686	Broom and Apply Prime Coat	sq m	0.026	0.031
B688	Broom and Apply Tack Coat	sq m	0.015	0.016
P104	ACP from pit; Desg. 1, Class 16	tonne	9.149	10.496
P130	Haul ACP; BLF	tonne	0.741	0.754
P131	Haul ACP; Haul	t km	0.112	0.112

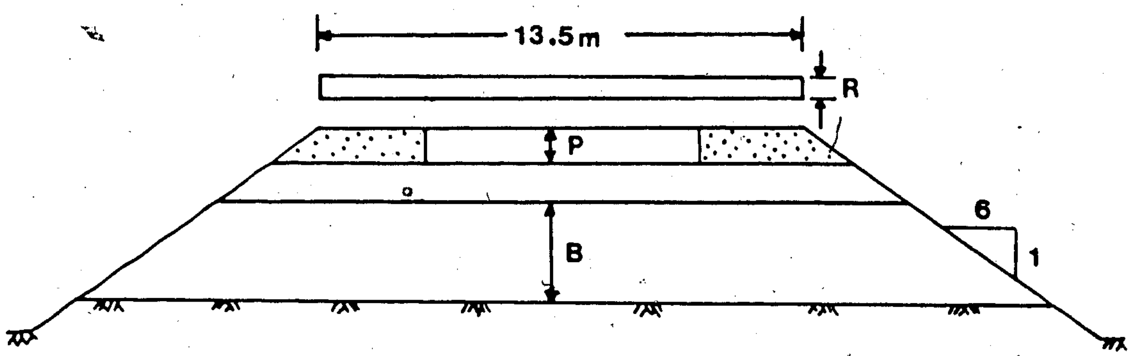

Figure C.1 Pavement Cross-Section Used for Economic Analysis