University of Alberta

Material Properties for Implementation of Mechanistic-Empirical Rigid Pavement Design Procedure (Design Guide 2002) in Alberta

by

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ABSTRACT

Due to increasing cost of pavement rehabilitation and maintenance, highway agencies put more efforts in finding the best alternatives for paving their roads. Portland Cement Concrete Pavements (PCCP) have higher initial cost, however with longer service lives and lower needed repairs could be an attractive choice for paving high traffic roads.

This thesis presents reviews and conducts sensitivity analyses on the existing and new rigid pavement design methods (AASHTO, PCA, and Design Guide 2002) and compares the influences of different design inputs in performance of rigid pavements during its service life. A typical pavement concrete from a local project in Edmonton was tested based on Design Guide 2002 (DG 2002) requirements to facilitate the implementation of DG 2002 for the future projects in Alberta and results were used for redesign of the PCCP using existing rigid pavement design methods.

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CHAPTER 1: INTRODUCTION

1.1 Background

Pavements are one of the most important components of our infrastructure. The design, maintenance, and rehabilitation of pavements are one of the main concerns of highway agencies. For example, the Texas Department of Transportation (TxDOT) spends more than 50 percent of its annual construction and maintenance budget on pavements (TxDOT 2004). Therefore, pavements need to be properly designed using an analytical process with accurate design inputs.

Pavements are designed to withstand heavy traffic loads, extreme environmental conditions and must provide safe, comfortable and smooth rides for users.

Generally, there are two types of pavements: flexible (surfaces paved with asphalt concrete) and rigid (surfaces paved with cement concrete). Traditionally, rigid or Portland Cement Concrete Pavements (PCCP) have been designed according to American Association of State Highway and Transportation Officials (AASHTO) or Portland Cement Association (PCA) methods. These methods were developed based on much lower traffic loads than we have today, old paving materials and technologies, and lower expected service lives. As a result they do not reflect the performance of PCCP over its service life accurately.

Existing pavement design procedures mainly rely on empirical approaches and do not consider mechanistic design concepts. Therefore, existing rigid pavement design methodologies do not capture the actual behavior of PCCP (Guclu and Ceylan 2005).

The AASHTO empirical rigid pavement model for the performance of Jointed Plain Concrete Pavements (JPCP) and Jointed Reinforced Concrete Pavements (JRCP) could not predict the performance of PCCP under various traffic and environmental conditions. It has been modified and extended to make possible the estimation of allowable axle load applications to a given terminal serviceability level for conditions of concrete strength, subgrade k-value, and concrete modulus of elasticity different than those of the AASHTO Road Test. The AASHTO design methodology has also been extended to accommodate the conversion of mixed axle loads to equivalent 80-kN (18-kip) through the use of load equivalency factors (Huang 1993).

The PCA's concrete pavement design procedure for roads and streets evaluates a candidate pavement design with respect to two potential failure modes: fatigue and erosion. The procedure was developed using the results of finite element analyses of stresses induced in concrete pavements by joint, edge, and corner loading. The analyses take into consideration the degree of load transfer provided by dowels or aggregate interlock and the degree of edge support provided by a concrete shoulder. For each load level considered, the expected number of load repetitions over the design life is expressed as a percentage of the allowable repetitions of that load level with respect to both fatigue and erosion. An adequate thickness is one for which the sum of the contributions of all axle load levels to fatigue and erosion damage is less than 100 percent (Huang 1993). This model is a mechanistic based one; however, it does not consider important input factors such as the thermal properties of concrete, which have an important role in the performance of PCCP.

Due to limitations in existing pavement design methods, a new mechanistic-empirical design method has been developed to overcome the deficiencies of traditional pavement design methods. The new mechanistic-empirical pavement design procedure, is named Design Guide 2002 (DG 2002) in this thesis, was developed under National Cooperative Highway Research Program (NCHRP) 1-37A (United States). An evaluation version of the DG 2002 is available on the Transportation Research Board webpage (http://www.trb.org/mepdg/). DG 2002 is a pavement performance evaluation tool which, based on traffic, climate, and pavement materials, predicts the distress (roughness, cracking, faulting, etc) level at any age during the design life of a project. The DG 2002 is structured in three levels and allows the user to use input measured values (Level 1), derived values (Level 2), or typical values (Level 3).

The DG 2002 for rigid pavement brings improvements in the following areas (Hall 2000):

- Consideration of climatic effects such as curling temperature and warping moisture on concrete pavement behavior,
- Effects of subsurface drainage on concrete pavement performance,
- The influence of the properties of aggregate on concrete pavement joint and crack formation and behavior,

- The influence of base types and their properties on concrete pavement performance, and
- Methods for the design of doweled transverse joints.

DG 2002 is a complex pavement model which requires comprehensive traffic, materials, and environmental inputs. This method is in final evaluation stage and has not been implemented by any agency in North America.

Given the high number of input parameters in the DG 2002, it is important for users to know which input parameters have the highest effect on pavement performance. In addition, DG 2002 was designed and calibrated based on global findings from the Long Term Pavement Performance (LTPP) program. Therefore, to implement DG 2002, local traffic, materials, and climate calibrations need to be considered.

1.2 Research scope and objectives

The main objective of this research is to facilitate the implementation of the DG 2002 rigid pavement design for Alberta Infrastructure & Transportation (AI&T). To achieve the objective of this study, two main tasks were conducted:

1. A sensitivity analysis was conducted on two existing rigid pavement designs including AASHTO and PCA methods as well as DG 2002, to compare the significance of design input factors among them.
2. A typical PCCP from a project in Edmonton, Alberta was characterized, and testing results were compared with models and suggested values from the DG 2002.

The study was focused on the material characterization of DG 2002 and did not study environmental and traffic factors.

1.3 Thesis structure

The thesis is structured in five chapters and one appendix, revealing all aspects of the research.

Chapter 1 presents general information about the research objectives and the structure of the report. Chapter 2 highlights the existing practices in rigid pavement design. This chapter introduces and compares rigid and flexible pavements and explains the advantages and disadvantages of these pavements. In addition, this chapter describes the AASHTO and PCA rigid pavement design methods. Chapter 2 also includes a review of Joint Plain Concrete Pavement (JPCP) design using the DG 2002 and discusses issues regarding the implementation of the DG 2002. Chapter 3 presents a comparison between rigid pavement design methods (AASHTO, PCA, and DG 2002) by conducting sensitivity analyses on them. Chapter 4 introduces the first Portland Cement Concrete Pavement (PCCP) project in Edmonton, the Anthony Henday Drive project, and presents Portland cement concrete test results from this project and how they could be used to calibrate the DG 2002 for Alberta. Chapter 5 presents conclusions and findings from this study and recommendations for future studies in this area. Details concerning testing data are included in the Appendix.

CHAPTER 2: RIGID PAVEMENT DESIGN METHODS

2.1 Introduction

Pavements are one of the most important components of our infrastructure. Pavements are designed to withstand heavy traffic loads, extreme environmental conditions, and must provide smooth rideability to users. Pavement must be designed to perform at an acceptable serviceability level with the lowest total costs during its service life.

Generally, there are two types of pavements: flexible (i.e., surface paved with asphalt concrete) and rigid (i.e., surface paved with cement concrete). The main difference between these pavements is the way in which they distribute traffic loads. In concrete pavements, concrete slabs provide the major portion of pavement's structural capacity. As can be observed in Figure 2-1, rigid pavement, due to the high stiffness of concrete, tends to distribute traffic loads over a relatively wide area of subgrade. Flexible pavement, built using asphalt mixture, distributes traffic loads more locally due to reduced stiffness of asphalt concrete (Huang 1993). Therefore, flexible pavements will usually require more layers and greater thickness of materials to reduce stresses on the same subgrade.

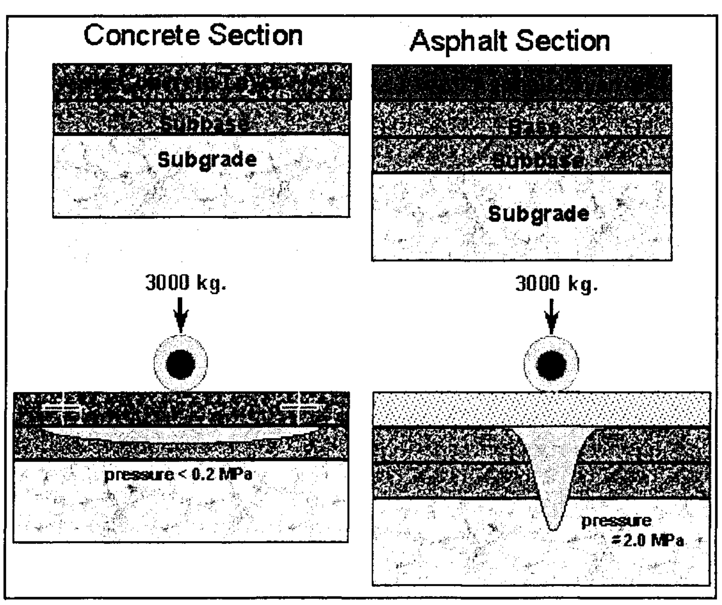

Figure 2-1: Rigid and Flexible Pavements

2.1.1 Rigid pavement: Advantages and disadvantages

There is a growing competition between asphalt and concrete industries in the area of pavement construction. Each industry highlights the advantages of its materials and provides case studies that the total costs of pavements during their service lives are less than those projects using the other movement material. Some comparisons between these two types of pavements are:

Construction and maintenance costs and service life

Embracher et al. (2001), in a Life Cycle Cost Analysis (LCCA) compared PCC and Asphalt Concrete (AC) pavements from Minnesota. They found that PCC pavements were clearly more cost-effective. For example, in Olmsted County, the Equivalent Unit Annual Costs (EUAC) of PCC and AC sections studied were $574 and $597 (US Dollars), respectively, per lane mile per year per million vehicles.

A LCCA study performed by the American Concrete Pavement Association (ACPA) in Utah compared PCC and AC pavements using EUAC. It was found that EUAC range from $9,510 to $10,047 for PCC pavements and from $11,827 to $13,881 for AC pavements. The analyzed period was 30 years; the EUACs for AC pavements were 38% greater than those for PCCP. The same study compared the service lives of PCC and AC pavements and found that the average life of PCCP is 2.5 times greater than the service life of AC pavements (31.4 versus 12.6 years, respectively) (ACPA 2005).

A life cycle cost report by ERES Consultants Inc. indicates that the expected life of an asphalt road is 17 years compared to 34 years for concrete. The report also indicates that asphalt highways require maintenance activities every three to five years and that major rehabilitation becomes increasingly frequent after the initial 17 year overlay. Concrete, on the other hand, requires its first minor maintenance after 12 years and will require a retexturing of the concrete surface after 18 years (ERES 1998).

The Minnesota Asphalt Pavement Association reported that the average service life of pavement with Hot Mix Asphalt (HMA) is 20 to 25 years. An asphalt overlay lasts another 15 years, so a total of at least 35 years service life can be achieved (Holt 2002).

Environmental aspects

In a five-year study performed by Nova Scotia Transportation and Public Works (1999), the roadside noise levels of concrete pavement were, on average, two to four decibels (dBA) higher than the asphalt pavement. To put this into perspective, normal conversation registers at 60 to 70 decibels and a human whisper registers at 20 decibels (Nova Scotia 1999). The Synthesis 268 study of NCHRP (1998) found that when dense-graded asphalt and PCC pavement are compared, the dense-graded is quieter by two to three dBA (NCHRP 1998).

In terms of energy and pollution, concrete pavements consume more energy and are more polluted due to the production of Portland cement. One tonne of cement consumes about four GJ of energy (equivalent to 131 cubic meters of natural gas) in electricity, process heat, and transport, produces approximately one tonne of CO_2 and produces about 3 kg of NO_X, an air contaminant that contributes to ground-level smog. In manufacturing 1.56 billion tonnes of Portland cement each year worldwide, an equivalent amount of CO_2 is released into the air (Eco-Smart website). Cement manufacturing is also a source of greenhouse gas emissions, accounting for approximately 7% to 8% of CO_2 globally (Mehta 1998).

User costs

Embacher et al. (2001) reported that concrete pavement provides fuel savings for heavy vehicles. Heavy trucks get up to 19% better mileage on concrete. Zaniewski (1981) also performed a comprehensive study of the relationship between highway design and vehicle operating costs, which considered several cost components, one of which was fuel consumption. Based on this analysis, it was found that the savings in fuel consumption for heavy vehicles travelling on concrete versus asphalt pavements was up to 20% greater on concrete.

Heavy vehicles cause greater deflection on flexible pavements than on rigid pavements. This increased deflection of flexible pavements absorbs part of the vehicle energy that would otherwise be available to propel the vehicle. Thus, it has been concluded that more energy and therefore more fuel is required to drive on flexible pavements. Concrete's rigid design reduces road deflection and its corresponding fuel consumption. It is estimated that concrete pavements improve fuel consumption for heavy vehicles by 11% (NRCC 2000).

Another study by Wisconsin Department of Transportation showed that the roughness of the asphalt pavement is more than double that of the concrete after five years of service (i.e., 6.8 mm/100 meters on concrete versus 16.2 mm/100 meters on asphalt). This result highlights the enhanced ride comfort and quality provided by PCC compared to AC pavements (WDOT 1997). From the driver's perspective, then, concrete pavements reduce the fuel consumption for heavy vehicles, reduce the delay due to less maintenance works, eliminate load spring restrictions, and improve visibility during night time driving.

It can be observed that each pavement type has its own advantages and disadvantages. While considering only environmental conditions or initial cost of construction are in favour of flexible pavements, by increasing cost of maintenance and repair for pavement, it seems that more agencies are looking to a more durable, lower total cost and high service life for pavements. Therefore, it is predicted that application of PCCP is increasing in pavement construction. A comprehensive life cycle analysis for these two common types of pavements could depend on many other factors and must be addressed at a project level for specific conditions and not at the network level.

2.1.2 Existing practices on rigid pavement design

The theory of pavement thickness design was advanced by the work of Westergaard in 1926. He presented equations for determining stresses and deflections in concrete pavements due to loads applied at the interior of the slab and at the free edges and corners. Factors such as the size and weight of loads, subgrade reaction, concrete thickness, modulus of elasticity, and Poisson's ratio were included. Engineers have used these equations, which permitted the determination of pavement thickness for any specified condition of loading, for many years (Yoder and Witczak 1975).

In 1933, the Portland Cement Association (PCA) introduced fatigue concepts to rigid pavement design, based on results from the Bates Test Road, in which the number of wheel load repetitions causing slab failure was related to the computed stress level (Huang 1993). Equations 2-1 and 2-2 approximated the computations of the Westergaard theory based on behaviours gleaned from the Bates Test Road sections.

Case I - Protected corners (smooth longitudinal edge bars)

$$S = \frac{1.92 \times W}{d^2} \qquad \text{Equation 2-1}$$

Case II - Unprotected corners (no edge bars)

$$S = \frac{2.4 \times W}{d^2} \qquad \text{Equation 2-2}$$

Where:

S = allowable stress for concrete (psi)

W = wheel load (lbs)

d = slab thickness (in.).

During the 1950s, many highway agencies developed their own rigid pavement design procedures. The AASHTO Interim Guide for Design of Pavement Structures based on the results of the AASHTO road test was published in 1972. (By this time, AASHO had changed its name to AASHTO, the American Association of State Highway and Transportation Officials). The concept of "Equivalent Single Axle Loads," introduced by engineers at the road test, was included to simplify the handling of axle loads of mixed magnitudes.

In 1984, PCA procedures were revised with a comprehensive analysis of concrete stresses and deflections by a finite-element computer program. The program modelled the conventional design factors of concrete properties, foundation support, and loadings, as well as joint-load transfers by dowels or aggregate interlock and concrete shoulders, for axle-load placements on the slab interior, edge, joint, and corner (Huang 1993).

A revision of the AASHTO Guide for Design of Pavement Structures was published in 1986. It retained the basic algorithms developed from the AASHO Road Test, as used in the Interim Guide, but was expanded to include many new considerations, such as reliability concepts, improved material characterization, drainage and environmental conditions, tied concrete shoulders or widened lanes, life cycle cost analysis, and pavement management considerations (Yoder and Witczak 1975).

2.1.3 AASHTO rigid pavement design method

The AASHTO 1993 design equation for rigid pavement is (Equation 2-3):

$$\mathrm{Log_{10}ESAL} = Z_R \times S_0 + 7.35 \times [\log_{10}(D+1)] - 0.06 + \frac{\log_{10}\dfrac{P_I - P_T}{4.5 - 1.5}}{1 + \dfrac{1.624 \times 10^7}{(D+1)^{8.46}}} + \quad \text{Equation 2-3}$$

$$+ (4.22 - 0.32 \times P_T) \times \log_{10}\left[\frac{S'_C \times C_D \times (D^{0.75} - 1.132)}{215.63 \times J \times (D^{0.75} - \dfrac{18.42}{(E_C/K)^{0.25}}}\right]$$

Where:

D = the required depth of the concrete slab (in.)

ESAL = accumulated 18-kip (80-kN) Equivalent Single Axle Loads over the life of the project

Z_R = standard normal deviation for a given reliability (reliability = 95% => Z_R = -1.645)

K = modulus of subgrade reaction (pci)

S_0 = overall standard deviation

P_I = initial serviceability

P_T = terminal serviceability

S'_C = modulus of rupture of concrete (psi)

E_C = modulus of elasticity of concrete (psi)

C_D = drainage coefficient

J = joint transfer factor.

As the latest revision to the AASHTO rigid pavement design method, the empirical AASHTO 1993 model for designing PCC pavements predicts the log of the number of axle load applications (log W) as a function of the slab thickness, axle type (single or tandem) and weight, and terminal serviceability (AASHTO 1993).

2.1.4 PCA rigid pavement design method

The PCA's concrete pavement design procedure for roads and streets considers two potential failure modes for rigid pavements: fatigue and erosion. The fatigue criteria retain pavement stresses due to repeated loads within safe limits, and the erosion criteria limit the effects of pavement deflections at edges, joints, and corners. The analyses take into consideration the degree of load transfer provided by dowels or aggregate interlock and the degree of edge support provided by a concrete shoulder. Warping and curling of concrete are assumed to cancel each other out (Huang 1993).

The PCA method could be applied to the following types of pavements: plain, plain doweled, reinforced, and continuously reinforced.

The input parameters in the analysis are:

- Type of joint and shoulder,
- Concrete modulus of rupture (M_R) at 28 days,

- k-value of the subgrade or, subgrade and subbase combination,
- Load safety factor (LSF),
- Axle-load distribution, and
- Expected number of axle-load repetitions during design period (PCA 1995).

PCA is a pavement analysis tool for which different slab thicknesses are tested in order to obtain distress (erosion and fatigue) levels of less than 100%. The fatigue and erosion analyses are performed by calculations on a worksheet. This worksheet organizes the calculations by traffic characteristics (single and tandem axles) and by types of pavement distress (fatigue and erosion). The design inputs are included at the top of the worksheet and the calculations are organized as presented in the seven columns of Table 2-1.

Table 2-1: Example of PCA rigid pavement design calculation

PCAPAV Rigid Pavement Design Method				Y	N
Trial Thickness	9.5	in	Doweled Joints	x	
Subbase-subgrade, k	130	pci	Concrete Shoulder		x
Modulus of Rupture, M_R	650	psi	Design Period	20	years
Load Safety Factor, LSF	1.2		Subbase	4 in	untreated

			Fatigue Analysis		Erosion Analysis	
Axle Load, kips	Multiplied by LSF	Expected Repetitions	Allowable Repetitions	Fatigue, %	Allowable Repetitions	Damage, %
1	2	3	4	5	6	7
8. Equivalent Stress		206.0	10. Erosion Factor		2.590	
9. Stress Ratio Factor		0.317				
Single Axles						
12	14.4	1,835,373	Unlimited	0.0	Unlimited	0.0
14	16.8	586,278	Unlimited	0.0	Unlimited	0.0
16	19.2	422,102	Unlimited	0.0	Unlimited	0.0
18	21.6	307,065	Unlimited	0.0	64,000,000	0.5
20	24.0	235,507	Unlimited	0.0	23,000,000	1.0
22	26.4	106,884	Unlimited	0.0	11,000,000	1.0
24	28.8	64,312	1,200,000	5.4	5,900,000	1.1
26	31.2	30,118	230,000	13.1	3,500,000	0.9
28	33.6	14,719	77,000	19.1	2,200,000	0.7
30	36.0	6,341	27,000	23.5	1,500,000	0.4
11. Equivalent Stress		192	13. Erosion Factor		2.790	
12. Stress Ratio Factor		0.295				
Tandem Axles						
16	19.2	1,355,300	Unlimited	0.0	Unlimited	0.0
20	24.0	1,226,224	Unlimited	0.0	Unlimited	0.0
24	28.8	983,923	Unlimited	0.0	Unlimited	0.0
28	33.6	1,654,666	Unlimited	0.0	92,000,000	1.8
32	38.4	929,801	Unlimited	0.0	24,000,000	3.9
36	43.2	884,964	Unlimited	0.0	9,500,000	9.3
40	48.0	372,509	Unlimited	0.0	4,600,000	8.1
44	52.8	124,774	Unlimited	0.0	2,500,000	5.0
48	57.6	42,799	Unlimited	0.0	1,500,000	2.9
52	62.4	21,286	1,100,000	1.9	920,000	2.3
		TOTAL, %	63.0		38.8	

Columns 1 (axle loads, kip) and 3 (expected repetitions) of the table are input data related to traffic. Column 2 (axle loads multiplied by the LSF) of the table is obtained by multiplying column 1 by the LSF. For each type of axle (single and tandem), three elements are determined: equivalent stress, erosion factor, and stress ratio factor. The equivalent stress and erosion factors are interpolated between the values from each of the tables (similar to Tables 2-2 and 2-3), based on slab thickness and k value. Tables 2-2 and 2-3 present only an extract from the PCA method allowing a selection between more slab thicknesses and more modulus values for subgrade-subbase.

Table 2-2: Equivalent stress - no concrete shoulder (single axle / tandem axle)

Slab thickness, in	9.5	
k of subgrade-subbase, pci	100	150
Equivalent stress	215/205	200/183

Table 2-3: Erosion factor-doweled joints, no concrete shoulder (single axle/tandem axle)

Slab thickness, in	9.5	
k of subgrade-subbase, pci	100	150
Erosion factor	2.60/2.81	2.58/2.74

The stress ratio factor is calculated by dividing the equivalent stress to the flexural strength of concrete.

Columns 4 (allowable repetitions for fatigue analysis) and 6 (allowable repetitions for erosion analysis) from Table 2 are extracted from nomographs presented in Figure 2-2, based on the axle loads and stress ratio factor and erosion factor, respectively.

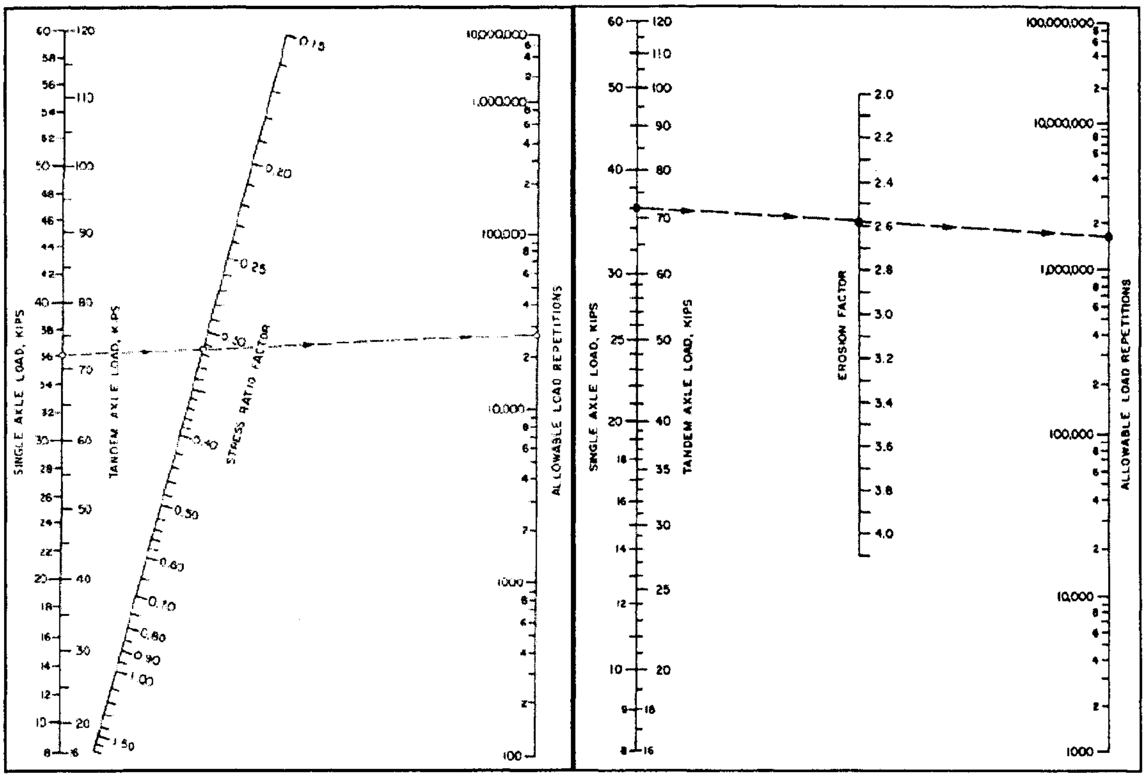

Figure 2-2: Allowable number of load repetitions (PCAPAV 1990)

Columns 5 (fatigue level, %) and 7 (erosion level, %) are calculated by dividing Column 3 to Columns 4 and 6. The totals from Columns 5 and 7 give the percentage of fatigue and erosion attained at the end of the design period. If both distresses are less than 100%, then the trial analysis is validated as acceptable. If at least one of the two distresses evaluated are greater than 100%, then some input parameters must be adjusted and another trial analysis needs to be performed in order to have both total distress levels at less than 100%.

A comparison between PCA and AASHTO pavement design methods is not easy, due to the fact that AASHTO is an empirically based method while the PCA method is based on stress calculation on slabs. Another main difference between the two methods is that the AASHTO method equates traffic with the 18-kip (80-kN) single axle load and does not differentiate the failing distress, while the PCA method uses actual single and tandem axle loads and considers two failing distresses for rigid pavement, namely, erosion and fatigue (Huang 1993).

2.1.5 Problems in existing PCCP design methods

Due to the fact that the climatic conditions, traffic levels, material types and subgrade conditions are all "built into" existing design procedures, they have now become outdated because of the significant changes in traffic loads, tire pressures, tire types, the use of new materials, new construction procedures and equipment, and other parameters that affect pavement performance. The AASHTO rigid pavement design method is empirically based. The design equation was obtained based on specific field experiments with specific materials and in specific (materials, weather, and traffic) conditions. In addition, no continuously reinforced concrete pavements (CRCP) were constructed at the Road Test. Furthermore, all CRCP design procedures are extrapolations to the jointed concrete procedure. For these and other reasons, the current AASHTO Guide is currently being replaced by a mechanistic-empirical procedure, which is based on mechanistic pavement performance and findings from the LTPP field studies.

The PCA design method is a mechanistic-based pavement analysis method and incorporates more detailed traffic data than AASHTO method; however, it still lacks important pavement characteristics, material, and climatic data.

2.1.6 The need to have a performance-based PCCP design (DG 2002)

The main reasons needed for a better pavement design method are as follows:

- Traffic loads have been increased significantly,
- New materials and construction methods have been developed,
- Long Term Pavement Performance (LTPP) and Strategic Highway Research Program (SHRP) studies provided significant field performance data during the last ten years, and
- Service life expectancy of pavements has increased.

One significant development towards improving design technology is the increased attention and effort presently given to the development of "mechanistic" designs. In addition, the availability of faster and higher computing power, makes it possible to conduct complicated pavement performance predictions. Therefore, after several years of research into pavement projects such as LTPP, it was decided to develop a mechanistic-empirical pavement design method. A comprehensive study was conducted for more than five years under the NCHRP 1-37A and a new pavement design guide (DG 2002) was developed in the USA.

The new 2002 Design Guide (DG 2002) is a mechanistic-empirical pavement design tool based on pavement performance criteria, allowing the user to design the pavement (rigid and flexible, new and rehabilitated) to correspond to specific conditions of traffic, climate, and materials. The guide was developed as part of the NCHRP 1-37A, initiated in 1996, and is structured in three design levels, allowing the user to input measured values (Level 1), derived values (Level 2), or typical values (Level 3).

Design Level 1 would be implemented only on a limited number of high traffic highways, due to the excessive cost of material characterization and traffic data collection. Design Level 2 would be used for projects on primary and selected secondary routes. The Design Level 1 requires real measurement data inputs while for Design Levels 2 and 3 only certain models and typical values are required. This type of structure gives agencies more flexibility in designing their projects.

2.2 Review of the DG 2002 for design of JPCP

2.2.1 Introduction

The Design Guide 2002 is based on a mechanistic-empirical design procedure, which calculates, mechanistically, pavement responses such as stresses, strains, and deflections. The cumulative damage is then determined and a time evolution of the distress is displayed for the design life of the project. The design procedure is presented in Figure 2-3 (DG 2002, User's Guide).

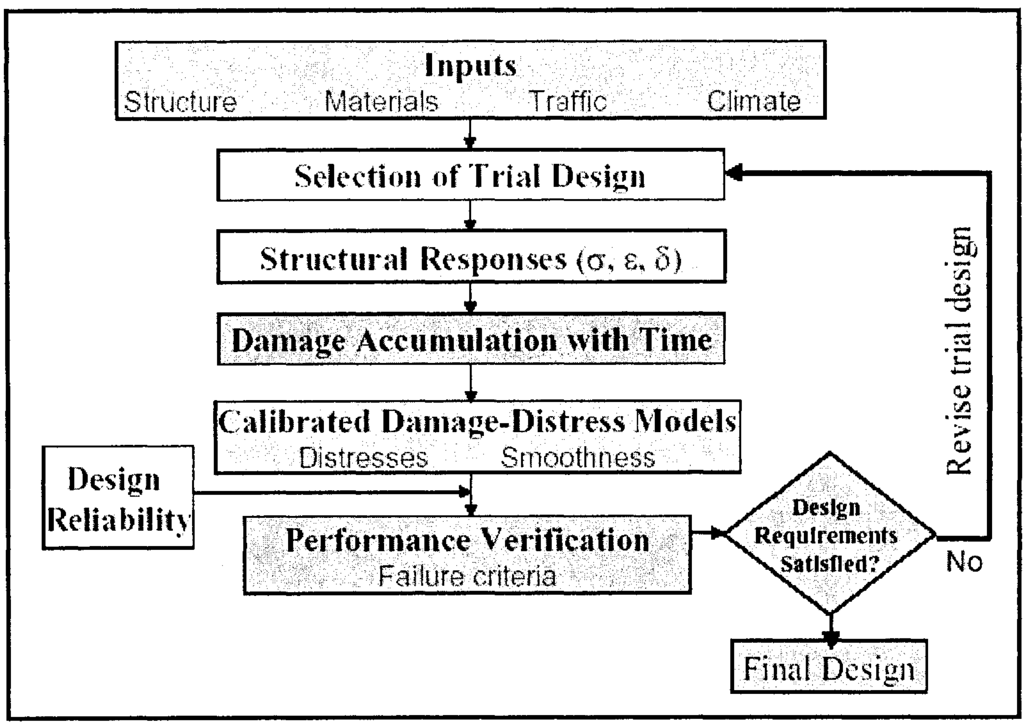

Figure 2-3: The DG 2002 procedure

Software and technical materials related to the DG 2002 can be found on TRB webpage (http://www.trb.org/mepdg/). Currently, the DG 2002 is offered as a free evaluation trial program, allowing agencies to use, test, and calibrate it to local conditions.

The principles of the Design Guide 2002 development include the following:

- Applies validated, state-of-the-practice technologies,
- Provide designers with the versatility to consider local design options,

- Provides an equitable design basis from the standpoint of pavement type selection,
- Addresses both new and rehabilitation design issues,
- Is user-friendly, and
- Has three hierarchical levels of design input, which allows the designer to match the level of effort to the importance of the project.

The benefits of the mechanistic-empirical procedure are:

- The consequences of non-traditional loading conditions can be evaluated. For example, the damaging effects of increased loads, high tire pressures, and multiple axles can be modelled.
- Better use of available materials can be made. For example, the use of stabilized materials in both rigid and flexible pavements can be simulated to predict future performance.
- Improved procedures to evaluate premature distress can be developed to analyze why certain pavements exceed their design expectations. In effect, better diagnostic techniques can be developed.
- Seasonal effects, such as thaw weakening, can be included in estimates of performance.
- Consequences of subbase erosion under rigid pavements can be evaluated, and
- Methods can be developed to better evaluate the long-term benefits of providing improved drainage in the roadway section.

Pavement design using the Design Guide is an iterative process and includes the following steps:

- The designer inputs a trial design,
- The software estimates the damage and key distresses over the design life, and
- The design is verified against the performance criteria at a desired level of reliability. The design may be modified as needed to meet performance and reliability requirements.

2.2.2 Distress prediction models for design of new JPCP using DG 2002

There are three critical distresses for PCCP in the DG 2002: roughness, transverse cracking, and faulting. Each distress has a prediction model which estimates the level of the distress at any age of the pavement.

Roughness prediction model for new JPCP

The DG 2002 uses International Roughness Index (IRI) in its roughness prediction model. Pavement roughness represents a deviation of pavement surface from a true planar surface with the characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads, and drainage. Pavement roughness is an indicator of pavement performance and is associated with driving comfort, vehicle operating costs, and safety. Roughness is expressed using the International Roughness Index (IRI) and is measured in mm/m or m/km. The model developed for smoothness predicting of JPCP pavements is as follows (Equation 2-4):

$$IRI = IRI_I + 0.013 \times TC + 0.007 \times SPALL + 0.005 \times PATCH + 0.0015 \times TFAUL + 0.4 \times SF$$

Equation 2-4

Where:

IRI = smoothness at a specific age (m/km)

IRI_I = initial smoothness measured as IRI (m/km)

TC = percentage of slabs with transverse cracking (all severities) (%)

SPALL = percentage of joints with spalling (all severities) (%)

PATCH = pavement surface area with flexible and rigid patching (all severities) (%)

TFAULT = total joint faulting cumulated per km (mm)

SF = site factor = Age×(1+FI)×(1+P200)/1000000

Age = pavement age (years)

FI = freezing index (°C days)

P200 = percent subgrade material passing the 0.075-mm sieve (%) (DG 2002, Appendix PP).

The roughness at each specific age is a complex element, which depends upon several other elements such as initial roughness, other rigid pavement distresses (transverse cracking, spalling, patching, and faulting), subgrade material properties, climate data, and the age of the pavement.

Transverse cracking prediction model for new JPCP

Transverse cracking is measured in terms of the percentage of slabs cracked and is expressed as the ratio of the number of slabs transversely cracked (cracks that are predominantly perpendicular to the pavement centerline) over the total number of slabs in the section (which is 100%). The general expression for fatigue damage accumulations considering all critical factors for JPCP transverse cracking is as follows (Equation 2-5):

$$FD = \sum \frac{n_{i,j,k,l,m,n}}{N_{i,j,k,l,m,n}} \qquad \text{Equation 2-5}$$

Where:

FD = total fatigue damage (top-down or bottom-up) (%)

$n_{i,j,k,\ldots}$ = applied number of load applications at conditions i, j, k, l, m, and n

$N_{i,j,k,\ldots}$ = allowable number of load applications at conditions i, j, k, l, m, and n

i = age (accounts for change in PCC modulus of rupture, layer bond condition, and deterioration of shoulder)

j = month (accounts for change in base and effective dynamic modulus of subgrade reaction)

k = axle type (single, tandem, and tridem for bottom-up cracking; short, medium, and long wheelbase for top-down cracking)

l = load level (incremental load for each axle type)

m = temperature difference between the top and the bottom of the slab (°C)

n = traffic path (a strip considered close to the pavement edge).

The number of load applications ($n_{i,j,k,l,m,n}$) is the actual number of type "k" axles of load level "l" passing through traffic path "n" under each condition (age, season, and temperature difference). The allowable number of load applications is the number of load cycles at which fatigue failure is expected (corresponding to 50 percent slab cracking) and is a function of the applied stress and PCC strength. The allowable number of load applications is determined using the following field-calibrated fatigue model (Equation 2-6):

$$\text{Log}(N_{i,j,k,l,m,n}) = C_1 \times \left(\frac{MR_i}{\sigma_{i,j,k,l,m,n}} \right)^{C_2} + 0.4371 \qquad \text{Equation 2-6}$$

Where:

$N_{i,j,k,\ldots}$ = allowable number of load applications at conditions i, j, k, l, m, and n

MR_i = PCC modulus of rupture at age i (psi)

$\sigma_{i,j,k,\ldots}$ = applied stress at condition i, j, k, l, m, and n

C_1 = calibration constant = 2.0

C_2 = calibration constant = 1.22.

The fatigue damage calculation is a simple process of summing the damage from each damage increment; however, a numerical integration scheme is used to determine accurately the effects of traffic wander (DG 2002, Appendix KK).

Faulting prediction model for new JPCP

Faulting, which is the difference in elevation across a joint or crack, is measured in millimeters. The mean transverse joint faulting is predicted using an incremental approach. A faulting increment is determined each month. The current faulting level affects the magnitude of the increment. The faulting each month is determined as a sum of the faulting increments of all the previous months in the pavement life since the traffic opening using the following model (Equations 2-7, 2-8, 2-9, and 2-10):

$$\text{Fault}_m = \sum_{i=1}^{m} \Delta\text{Fault}_i \qquad \text{Equation 2-7}$$

$$\Delta\text{Fault}_i = C_{34} \times (\text{FAULTMAX}_{i-1} - \text{Fault}_{i-1})^2 \times DE_i \qquad \text{Equation 2-8}$$

$$\text{FAULTMAX}_i = \text{FAULTMAX}_0 + C_7 \times \sum_{j=1}^{m} DE_j \times \log(1 + C_5 \times 5.0^{EROD})^{C_6} \qquad \text{Equation 2-9}$$

$$FAULTMAX_0 = C_{12} \times \delta_{curling} \times \left[\log(1 + C_5 \times 5.0^{EROD}) \times \log(\frac{P_{200} \times WetDays}{p_s}) \right]^{C_6}$$

Equation 2-10

Where:

$Fault_m$ = mean joint faulting at the end of month m (in.)

$Fault_i$ = mean joint faulting for month i (in.)

$\Delta Fault_i$ = incremental change (monthly) in mean transverse joint faulting during month i (in.)

$FAULTMAX_i$ = maximum mean transverse joint faulting for month i (in.)

$FAULTMAX_0$ = initial maximum mean transverse joint faulting (in.)

EROD = base/subbase erodibility factor

DE_i = differential deformation energy accumulated during month i

$\delta_{curling}$ = maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping

P_S = overburden on subgrade (lb)

P_{200} = percent subgrade material passing #200 sieve (%)

WetDays = average annual number of wet days (greater than 0.1 in rainfall)

$C_{12} = C_1 + C_2 \times FR^{0.25}$

$C_{34} = C_3 + C_4 \times FR^{0.25}$

FR = base freezing index defined as percentage of time the top base temperature is below freezing (32 °F) temperature

C_1 through C_8 = calibration constants.

The functional form of the model reflects the hypothesis that the faulting potential depends on the amount of PCC slab curling, base erodibility, and on the presence of fines and free water in the subgrade. Faulting potential decreases with an increase of overburden pressure on the subgrade. The rate of faulting development depends on the faulting level and decreases as the faulting increases until it stabilizes to a certain level (DG 2002, Appendix JJ).

Khazanovich et al. (2004) presented a summary of the procedures used to model the effects of transverse joint faulting in the design of Jointed Plain Concrete Pavements (JPCP) in the 2002 Design Guide. The paper describes the main concepts, presents the model overview, and provides the results of the model calibration (Equations 2-7 to 2-10). Several examples illustrating the sensitivity of the 2002 Design Guide faulting prediction to key design parameters, such as the dowel diameter, slab width, edge support, and built-in temperature gradient, are also provided.

The Design Guide 2002 faulting model identifies the differential energy of subgrade deformation as the mechanistic parameter governing joint faulting development. The differential energy of the subgrade deformation reflects the total pavement flexibility and the level of load transfer efficiency. The mean transverse joint faulting is predicted using an incremental approach. A faulting increment is determined each month; the current faulting level affects the magnitude of increment. Each month's faulting level is determined as a sum of the faulting increments from all previous months in the pavement life since the traffic opening.

The authors of the article concluded regarding the DG 2002 model that:

- The developed performance prediction model is a substantial enhancement of the FHWA PAVESPAC 3.0 faulting prediction model. Like the FHWA faulting model, the new model relates the differential energy of subgrade deformation to faulting development. The DG 2002 model retains all of the positive features of the PAVESPAC 3.0 model. It is capable of accounting for the effects of traffic volume, dowel diameter, PCC slab and base properties, subgrade support, and climatic conditions for faulting prediction.
- It uses axle spectrum distributions for traffic characterization.
- It uses an incremental damage approach that accounts directly for changes in the LTE and for PCC stiffness over time.

It accounts directly for seasonal and environmental effects on faulting development by considering seasonal variation in subgrade k-value, PCC slab warping, and curling.

The DG 2002 is a pavement analysis tool that predicts the long-term performance of pavement under traffic loads and environmental parameters; however, a pavement engineer can, by comparing performances, attempt several different pavement thicknesses in order to determine the optimum pavement design.

The program interface is user friendly. The user first provides the software with the General Information of the project and then provides inputs in three main categories: traffic, climate, and structure. All inputs for the software program are colour-coded. Input screens that have not been visited are coded "red". Those that have default values are coded "yellow." Those that have complete inputs are coded "green" (DG 2002, User's Guide) (Figure 2-4).

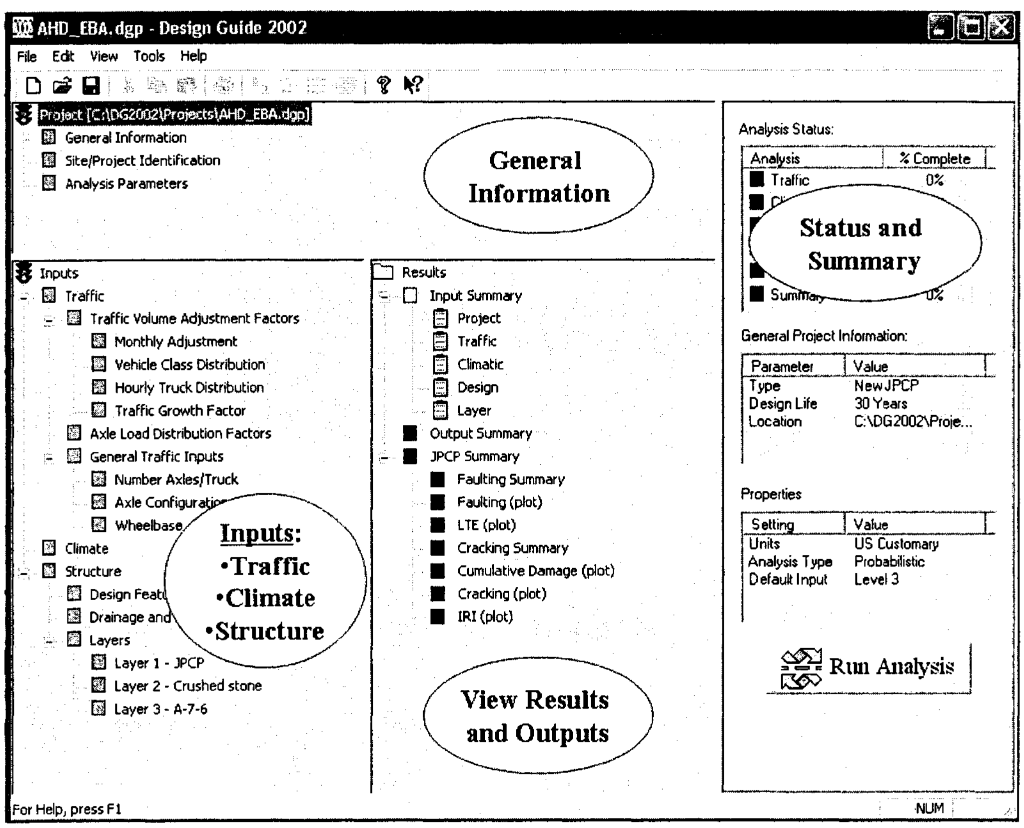

Figure 2-4: DG 2002 program inputs

Next, once all inputs are provided for the trial design, the user chooses to run the analysis. The software will execute the damage analysis and the performance prediction engines for the trial design input. The user can then view the input and output summaries created by the program. The program creates a summary of all inputs of the trial design as well as a summary of the distress and performance predictions, in both tabular and graphical formats. All charts are plotted in Microsoft Excel and can therefore be incorporated into electronic documents and reports (DG 2002, User's Guide).

2.2.3 DG 2002 for new jointed plain concrete pavement

In order to design a new Jointed Plain Concrete Pavement (JPCP) design project, in the main window of the program, the user has to access the General Information section, and select the option, "JPCP", from the New Pavement section (Design Guide 2002, Appendix D). The design life and the dates of construction for base, subbase, and pavement, and the level of opening traffic may also be selected in the same screen (Figure 2-5).

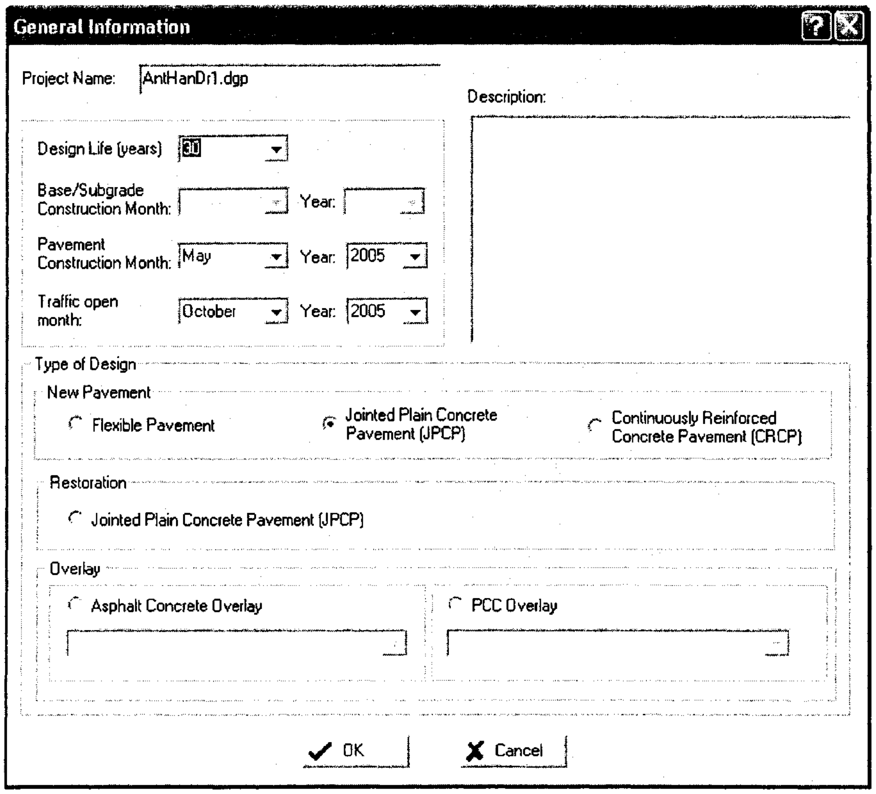

Figure 2-5: General information for DG 2002

The second option from the upper main menu is Site/Project Identification, into which the user may input data related to the location of the project. The third option from the same upper main menu is the Analysis Parameters input, into which the user inputs the initial

International Roughness Index (IRI), the analysis type, the distress limits, and their level of reliability (Figure 2-6).

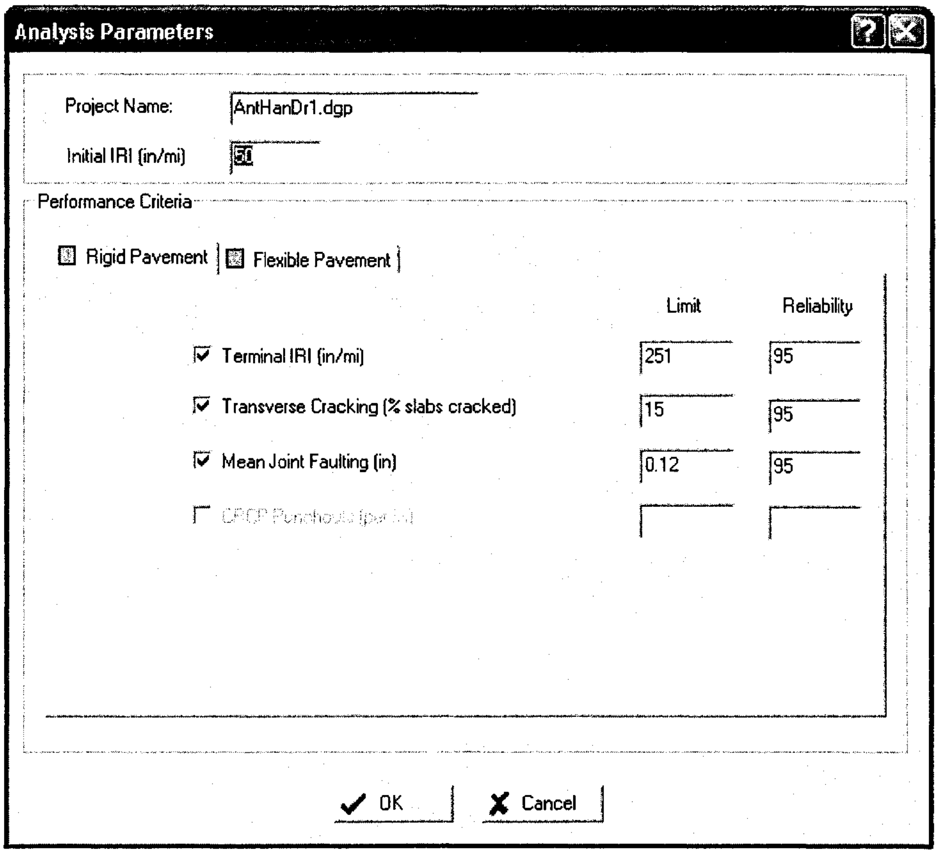

Figure 2-6: Analysis parameters for JPCP

After completing these general inputs, the user must input data related to traffic, climate, and structure. The user enters the traffic data in the main window, which will open a separate window (screen). In the traffic screen, data related to the two-way average annual daily truck traffic, the number of lanes in a design direction, the percentage of trucks in a design direction, the percentage of trucks in a design lane, operational speed, the traffic volume adjustment, the axle load distribution factors, general traffic inputs, and the traffic growth factor (Figure 2-7). With the traffic volume adjustment button, the user may input data related to Monthly Adjustment Factors (MAF), vehicle class distribution, hourly distribution, and traffic growth factor. For these input data, Level 1 or Level 3 accuracy may be selected if the user has site-specific data or uses the default values.

Furthermore, the user may input climate data in two ways: by selecting an available weather station or by giving geographical data of the site. The program automatically interpolates between the available weather stations and selects the most appropriate one (Figure 2-8).

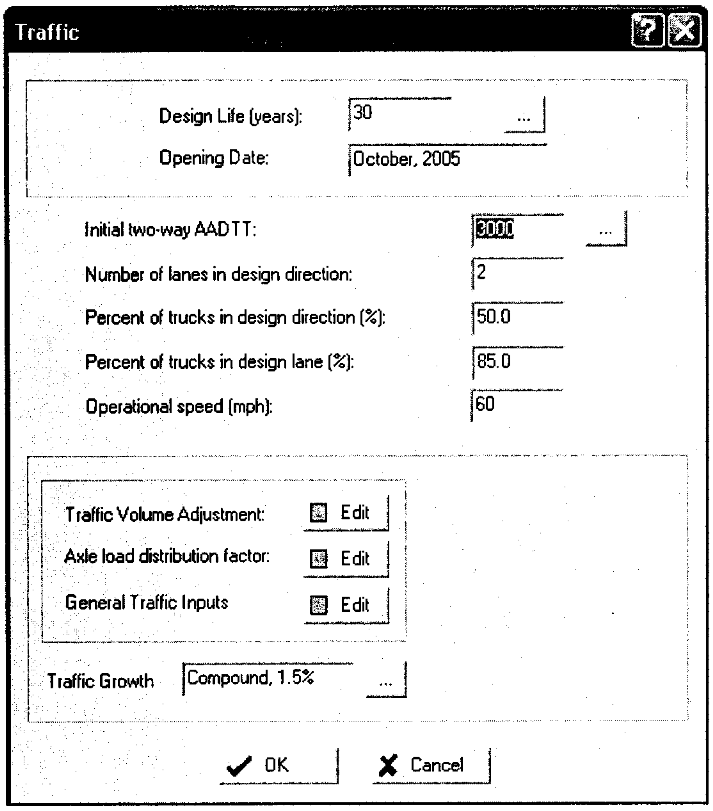

Figure 2-7: Traffic inputs

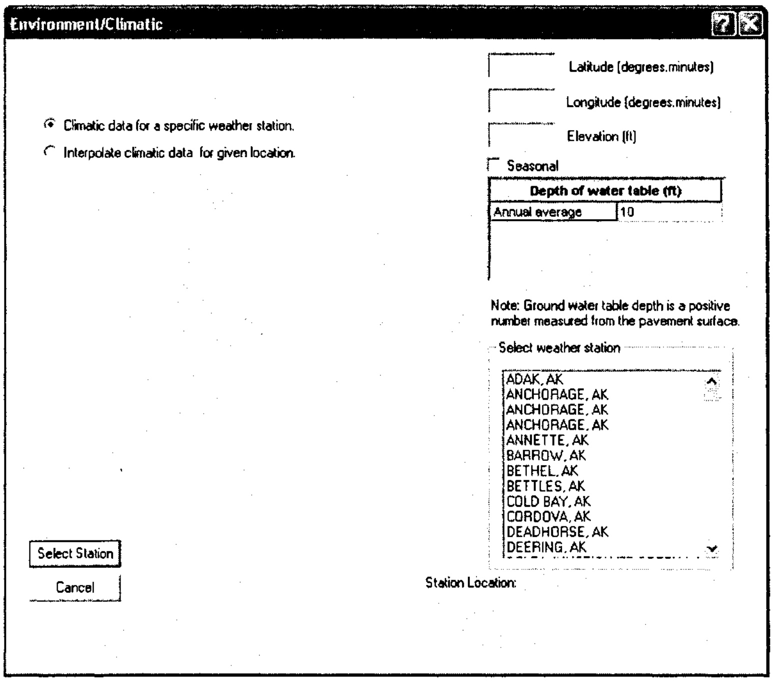

Figure 2-8: Generating climatic data file

In terms of Structural Data, on the JPCP Design Features screen (Figure 2-9), the user inputs the slab thickness, the permanent curl/warp effective temperature difference, the joint spacing, the sealant type, the dowel diameter, and the dowel bar spacing. In the same window, the user has access to other windows, which allow data related to pavement (layer type, material, and thicknesses) to be input (Figure 2-10).

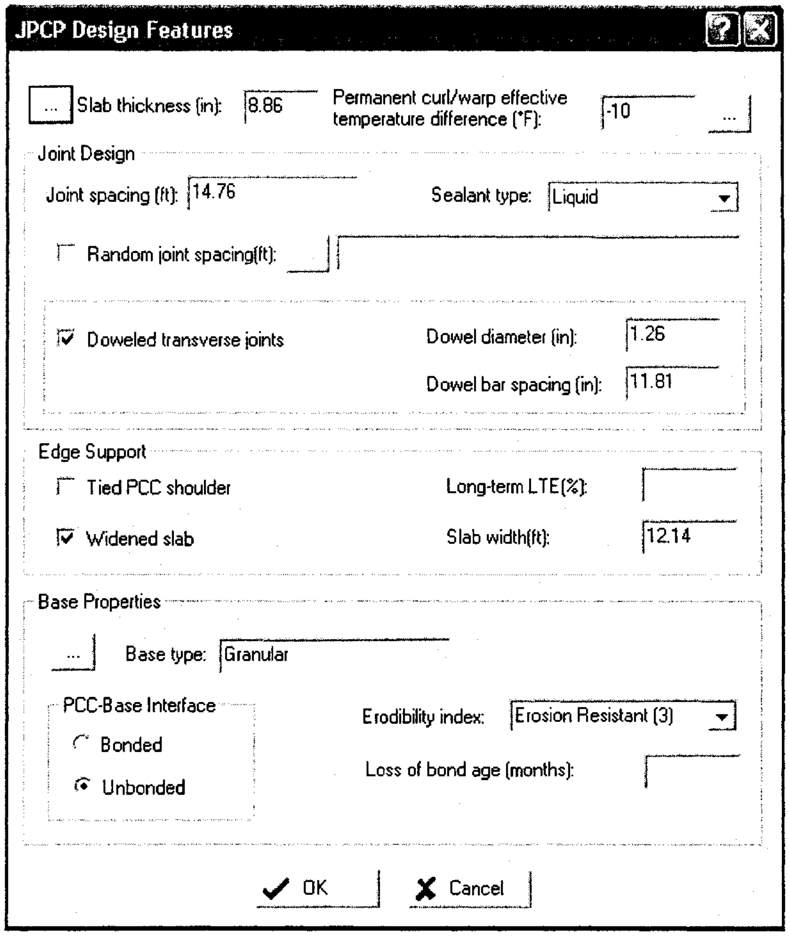

Figure 2-9: JPCP design features

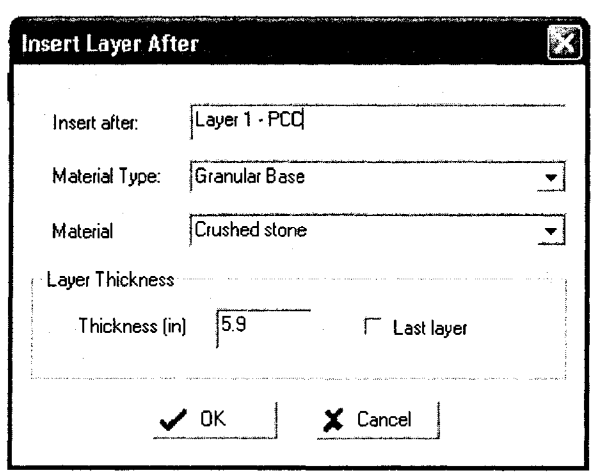

Figure 2-10: Layer data

For each pavement layer, different material properties must be used as inputs for the three levels of the DG 2002. In level 1, the DG 2002 requires a modulus of elasticity and a modulus of rupture at 7, 14, 28, and 90 days (Figure 2-11). In level 2 of the DG 2002, the compressive strength of concrete at 7, 14, 28, and 90 days must be measured. For level 1, the long-term ratio 20-year/28-day of modulus of elasticity and modulus of rupture; for level 2 the same ratio for the compressive strength must be estimated. In level 3, a 28-day PCC modulus of rupture or a 28-day compressive strength are needed as the PCC material properties. A 28-day modulus of elasticity can also be entered in the program as extra information for PCC material properties. Once all the input data were completed, the user should press the Run Analysis button in the main window. One run of the program for the new PCC design takes about five minutes using a Pentium 4 computer or equivalent.

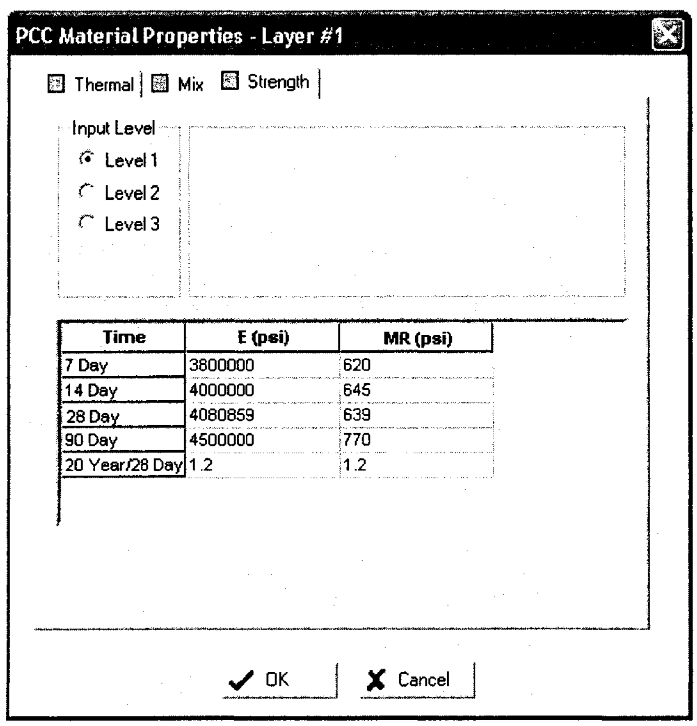

Figure 2-11: Material properties

The output of the analysis is an Excel file that uses tables and charts representing the input data and the cumulative distress evolution over the design life. The JPCP distresses considered by the program are the roughness (mm/m), the transverse cracking (% of slabs cracked) and the faulting (mm). Figures 2-12, 2-13, and 2-14 show a typical performance prediction from the DG 2002 in terms of IRI, TC, and faulting, respectively.

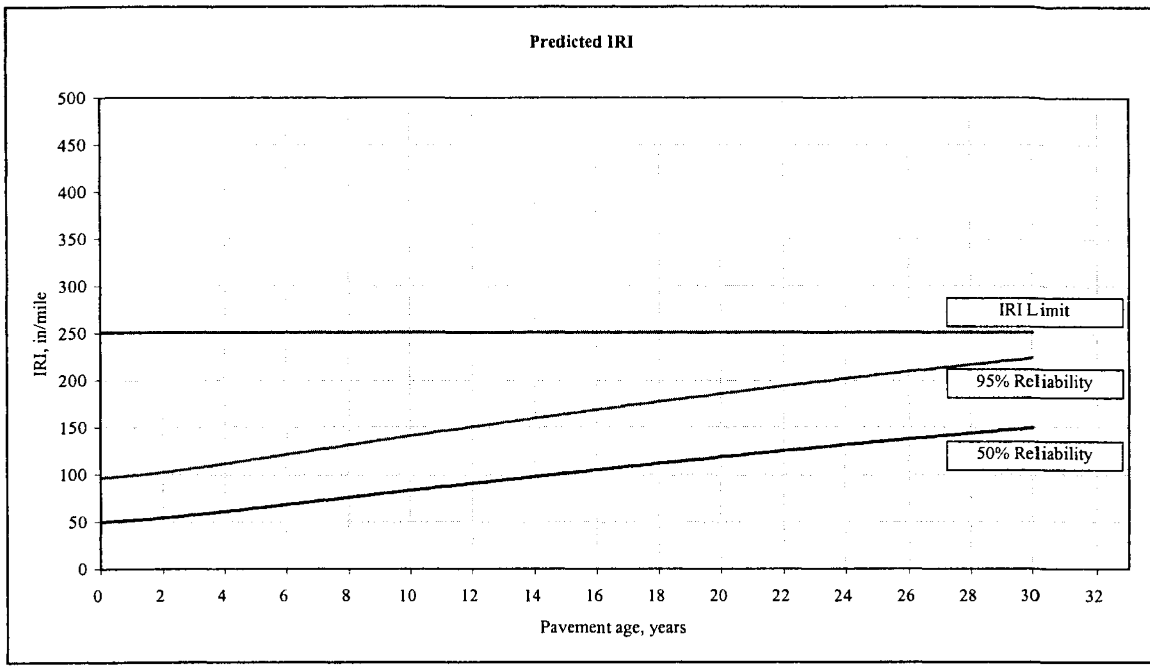

Figure 2-12: Example of IRI output chart for a PCCP by DG 2002

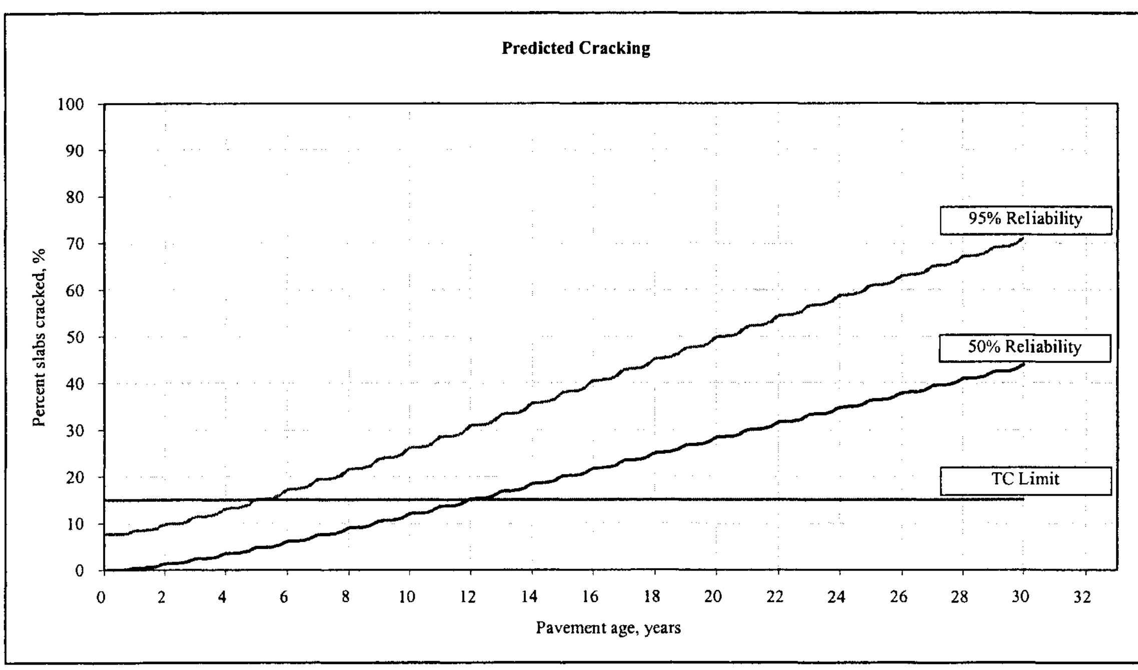

Figure 2-13: Example of transverse cracking output chart for a PCCP by DG 2002

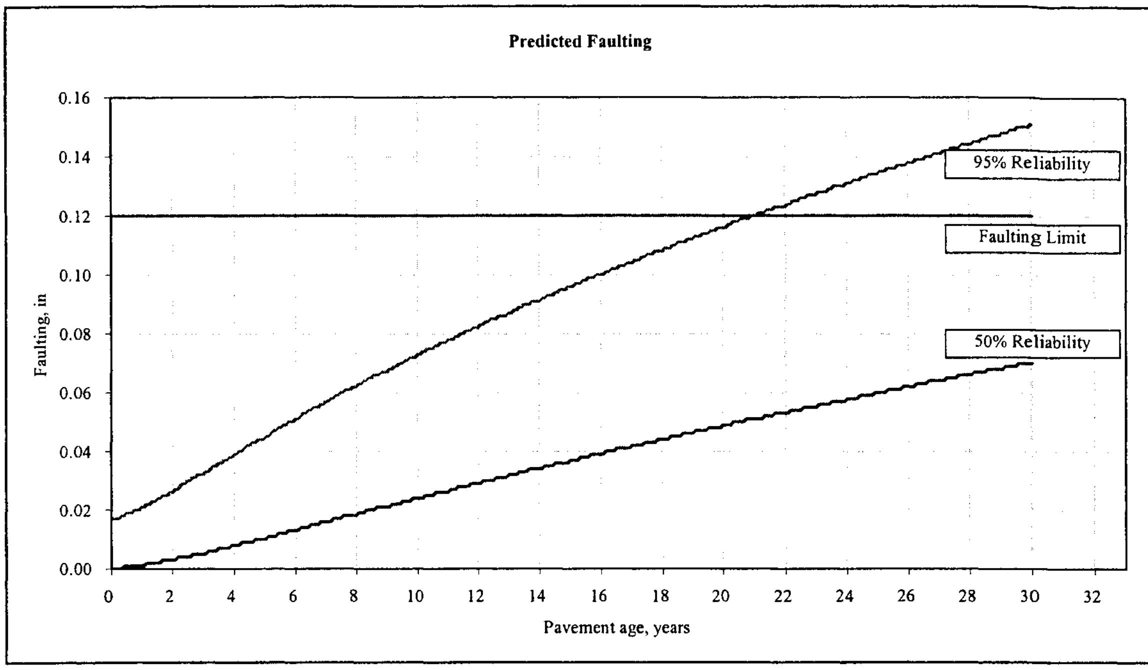

Figure 2-14: Example of faulting output chart for a PCCP by DG 2002

The output charts indicate the cumulative distress over the design life at a specified reliability level (e.g., 95%). A distress limit and a cumulative distress over the design life at a reliability level of 50% are predicted. In Figure 2-12, the IRI is predicted to be under the IRI limit for the entire design life of the project; however, in Figures 2-13 and 2-14, at the specified reliability level (different of the default 50%) the transverse cracking and faulting distresses pass the established limits after 17 and 11 years, respectively. In these cases, specific inputs need to be adjusted in order to have all three distresses predicted under the limits. The user has to run several different cases to select the optimum case from the feasible design options developed. For example, possible modifications to improve the faulting are:

- Increase the slab thickness (not best or economical alternative),
- Increase the diameter of the dowel bar across the transverse joint,
- Increase dowel bar size and decease thickness, and
- Increase thickness and decrease dowel diameter (uneconomical alternative).

The main difference between the DG 2002 and other existing design programs, in terms of output, is that the DG 2002 does not provide the required (designed) pavement (slab) thickness. By studying the output as a distress evolution, the user needs experience in order to know which inputs need to be updated and in what way, in order to reduce the magnitude of the distress which passes the distress limit before the end of the design life of the project. Therefore, it is important that engineer know which factors are most important for each pavement distress. This emphasizes the importance of sensitivity analysis for the DG 2002. This issue will be addressed further in Chapter 3. Another advantage of the DG 2002 is that it can predict when pavement fails. This function gives agencies a better decision-making tool for the maintenance and management of PCCP.

2.2.4 Challenges in implementation of the DG 2002 for Canadian agencies

Because the DG 2002 was developed in the USA and currently only a trial version is available, Canadian agencies face several problems in implementing the new design guide, including:

- The metric system option is not active,
- The climatic database is not complete and only specific location files for USA are available,
- The traffic and material data are not available at the detailed level requested by the program (for Level 1, specific field data), and
- The DG 2002 needs calibration for pavement materials, which are used in specific locations.

2.2.5 How this study attempts to address the implementation of DG 2002

In this study, material testing data from a typical Portland cement concrete used in a local project in Edmonton, Alberta were used as material input parameters in the new DG 2002 to predict the pavement performance. A sensitivity analysis was performed in order to establish the impact of the most important factors in the final results of the DG 2002. Because the methodology of the DG 2002 is different (the output is not the designed concrete slab thickness), the results of the DG 2002 can be compared only indirectly with the results of existing pavement design methods.

CHAPTER 3: SENSITIVITY ANALYSIS FOR PCCP DESIGN METHODS

3.1 Sensitivity analysis

Sensitivity analysis can help in identifying critical control points, prioritizing additional data collection or research, and verifying and validating a model. Sensitivity analysis can play an important role in model verification and validation throughout the course of model development and refinement (Kleijnen et al. 2000). Frey and Patil (2004) conducted a study on sensitivity analysis methods and grouped them as mathematical, statistical, and graphical. Each of the three groups includes a different sensitivity analysis method.

In this study, mathematical and graphical methods were applicable to each of the three rigid pavement design models. Mathematical methods assess the sensitivity of a model output to the range of an input's variation. These methods typically involve calculating the output for a few values of an input, in order to represent the possible range of the input (Salehi et al. 2000). These methods do not address the variance in the output as a result of the variance in the inputs, but they can assess the impact upon the output of the range of variation in the input values (Morgan and Henrion 1990). In some cases, mathematical methods can be helpful in screening the most important inputs (Brun et al. 2001).

The nominal range sensitivity method, such as the mathematical method, is also known as local sensitivity analysis or threshold analysis. This method is applicable to deterministic models. Nominal range sensitivity analysis evaluates the effect on model outputs exerted by individually varying only one of the model inputs across its entire range of plausible values while holding all other inputs at their nominal or base-case values. The results of this approach can be used to rank the order of the key inputs only if there are no significant interactions among the inputs, and as long as ranges are properly specified for each input. Nominal sensitivity analysis addresses only a small portion of the total possible space of input values, because interactions among inputs are difficult to capture (Cullen and Frey 1999).

Graphical methods represent sensitivity in the form of graphs, charts, or surfaces. Generally, graphical methods are used to indicate visually how an output is affected by input variation (Geldermann and Rentz 2001). Graphical methods can be used as a screening method before further analysis of a model or to represent complex dependencies between inputs and outputs (McCamley and Rudel 1995). Graphical methods can also be used to complement the results of mathematical and statistical methods, ensuring a better representation (Stiber et al. 1999).

Sensitivity analyses were conducted on the AASHTO, PCA, and DG 2002 to identify the most significant inputs in these rigid pavement design methods. For the AASHTO rigid pavement design method, there is a mathematical formula (Equation 2-3). This equation was rearranged and P_T (terminal serviceability) was considered as the dependant variable. Excel Solver function was used to calculate the range in P_T, as an indication of pavement performance, by changing all dependant variables. For the PCA design method, computer software was used to predict the change in pavement performance, in terms of erosion and fatigue cracking (PCAPAV 1990). For the DG 2002, the complete model is available as a computer program (DG 2002, User's Guide). As was explained in the previous section, this comprehensive pavement analysis model predicts the performance of rigid pavement in terms of roughness (IRI), transverse cracking, and faulting. For both PCA and DG 2002, input parameters were altered at different levels to determine the performance of the pavement.

In all sensitivity analyses, the average input values were taken either from a PCCP project in Edmonton or from typical values suggested by the various design methods.

3.2 Sensitivity analysis on AASHTO rigid pavement design method

The AASHTO rigid pavement design method was presented in Chapter 2. It is based on Equation 2-3 and is in imperial units. A sensitivity analysis was conducted to identify the most important factors in the AASHTO 1993 rigid pavement design equation. Table 3-1 presents the inputs of the AASHTO rigid pavement design method used in EBA design for the AHD pavement project in Edmonton.

Table 3-1: Input variables used in AASHTO rigid pavement design method by EBA

Design Inputs	Values/Type
Modulus of subgrade/subbase reaction, MPa/m	59.5
Modulus of rupture of PCC, MPa	4.7
Modulus of elasticity, MPa	25,000
Design traffic, 80 kN ESAL/direction	36.7×10^{6}
Load transfer coefficient	3.2
Slab thickness, mm	311
Reliability, %	95
Overall standard deviation	0.35
Drainage coefficient	1.0
Initial serviceability	4.5

Some considerations in the AASHTO 1993 sensitivity analysis are further explained. For this sensitivity analysis, the change in pavement serviceability ($\Delta PSI = P_I - P_T$) was considered to be the dependent variable. The performance is measured by a scale in which 0 reflects a smooth pavement and 5 reflects the worst pavement conditions.

By varying the input parameters, as presented in Table 3-2, the changes in pavement serviceability at the end of the service life (30 years), in terms of ΔPSI were predicted using the AASHTO rigid pavement design method and are presented in Figure 3-1.

Table 3-2: Input parameters and the different levels considered in the AASHTO rigid pavement design method sensitivity analysis

		Levels						
No	**Input parameters**	**Low1**	**Low2**	**Low3**	**EBA Design***	**High1**	**High2**	**High3**
1	D_R = the required depth of the concrete pavement, mm	-	220	254	**311**	356	406	-
2	$ESAL_D$ = accumulated 18-kip (80-kN) Equivalent Single Axle Loads over the life of the project	30,000,000	32,000,000	34,000,000	**36,700,000**	38,000,000	40,000,000	42,000,000
3	K_G = modulus of subgrade reaction, MPa/m;	13.6	27.1	40.7	**59.5**	67.8	81.4	94.9
4	P_I = initial serviceability	-	0.2	0.5	**0.8**	1.1	1.6	2.0
5	S'_C = concrete modulus of rupture, MPa	-	3.9	4.1	**4.7**	5.5	6.9	8.3
6	C_D = drainage coefficient	0.85	0.9	0.95	**1.0**	1.1	1.25	
7	J = joint transfer factor	2.9	3.0	3.1	**3.2**	3.3	3.4	3.5

* Bold values are from the EBA Design for Anthony Henday Drive project in Edmonton

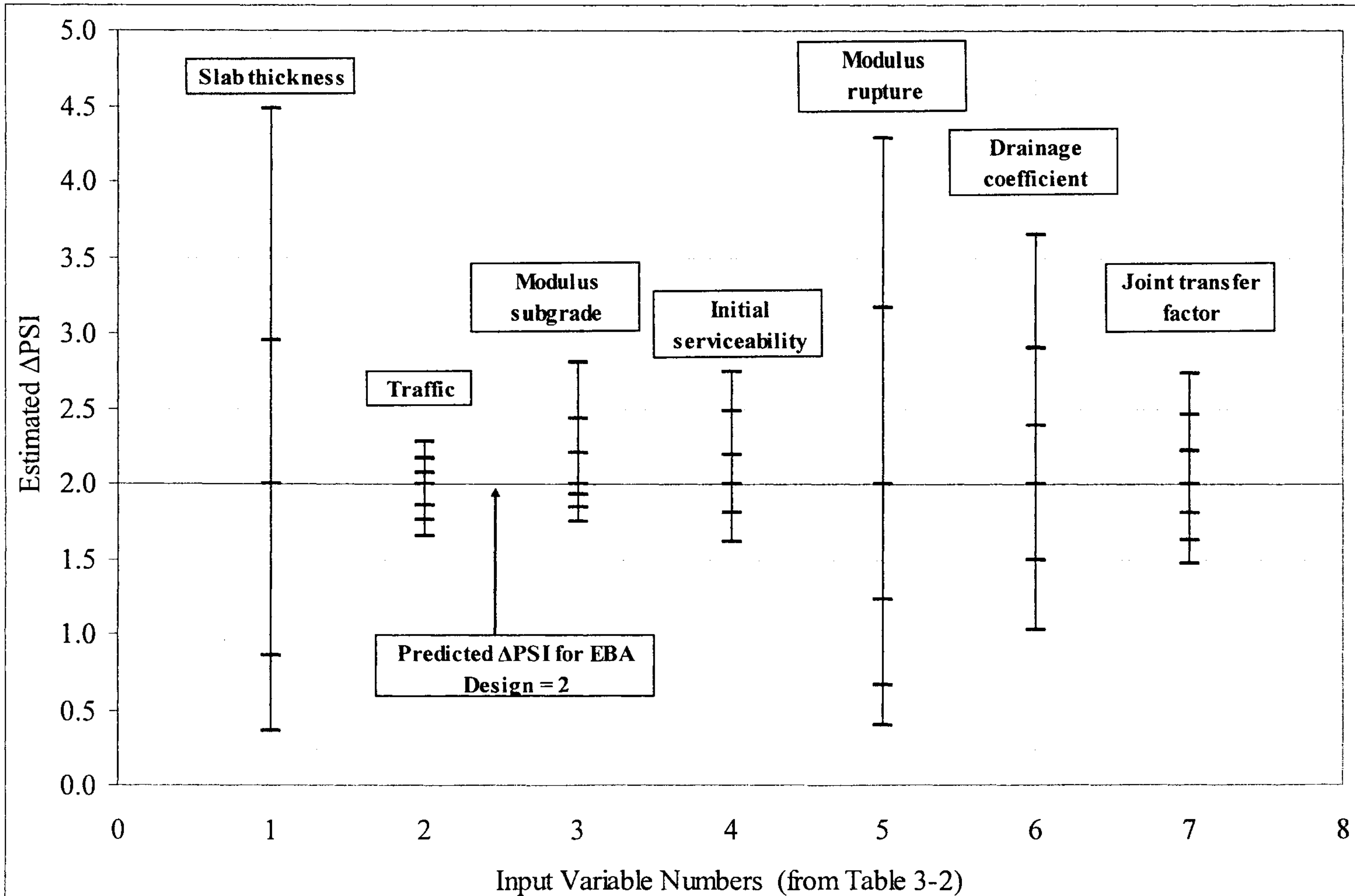

Figure 3-1: ΔPSI variability due to changes in various design inputs, using AASHTO rigid pavement design method

It can be concluded that the most sensitive parameters are slab thickness (#1), flexural strength of PCC (#5), and the drainage coefficient (#6). Other sensitive input parameters are the joint transfer factor (#7), initial serviceability (#4), and the modulus of subgrade reaction (#3). Traffic, in terms of Equivalent Single Axle Load (ESAL), is the least sensitive parameter in measuring the performance of rigid pavement based on the AASHTO design procedure.

3.3 Sensitivity analysis of PCA method

The Portland Cement Association PCAPAV computer program is based on the PCA design method, described in Chapter 2. It determines the slab thickness required to carry traffic loads on concrete highways (PCA 1990). The procedure applies to the following types of concrete pavements: plain, plain-doweled, reinforced, and continuously reinforced. The adequate pavement thickness is determined based on two criteria: fatigue (keeps pavement bending stresses due to repeated loads within safe limits) and erosion (limits the deflections of slab corners and edges). Some limitations of the PCAPAV program are related to the slab thickness, which must be between 102 and 356 mm, and k-value of subgrade/subbase, which must be between 14 and 190 MPa/m.

A sensitivity analysis was conducted on the PCA method to identify the most significant factors in performance of a typical concrete pavement in Edmonton. The average values for input variables used in the PCAPAV program by EBA Consultants Engineering in designing the PCC slab for the Anthony Henday Drive project are presented in Table 3-3.

Table 3-3: Input variables used in PCA design method by EBA

Design Inputs	Values/Type
Modulus of subgrade/subbase reaction, MPa/m	38
Modulus of rupture of PCC, MPa	4.2
Average Daily Truck Traffic (ADTT), (Load category 3)	3,200
Design life, years	30
Load transfer	Dowels and concrete shoulder
Load Safety Factor (LSF)	1.2
Slab thickness, mm	224
Axle load category	Input axles

Different levels for the input parameters used in this sensitivity analysis are presented in Table 3-4. The impact of these parameters on the performance of concrete pavement in terms of "fatigue" and "erosion" are presented in Figures 3-2 and 3-3, respectively.

Table 3-4: Input parameters and the different levels considered in the PCA design method sensitivity analysis

		Levels						
No	**Input parameters**	**Low1**	**Low2**	**Low3**	**EBA Design***	**High1**	**High2**	**High3**
1	D_R = the required depth of the concrete pavement, mm	-	-	216	**224**	241	254	267
2	ADTT = Average Daily Truck Traffic	1,500	2,000	2,500	**3,200**	4,000	4,500	5,000
3	K_G = Modulus of subgrade/subbase reaction, MPa/m	-	27	33	**38**	46	54	-
4	M_R = Modulus of Rupture, MPa	-	4.0	4.1	**4.2**	4.5	4.8	-
5	LSF = Load Safety Factor	-	1	1.1	**1.2**	1.3		-

* Bold values are from the EBA Design for Anthony Henday Drive project in Edmonton

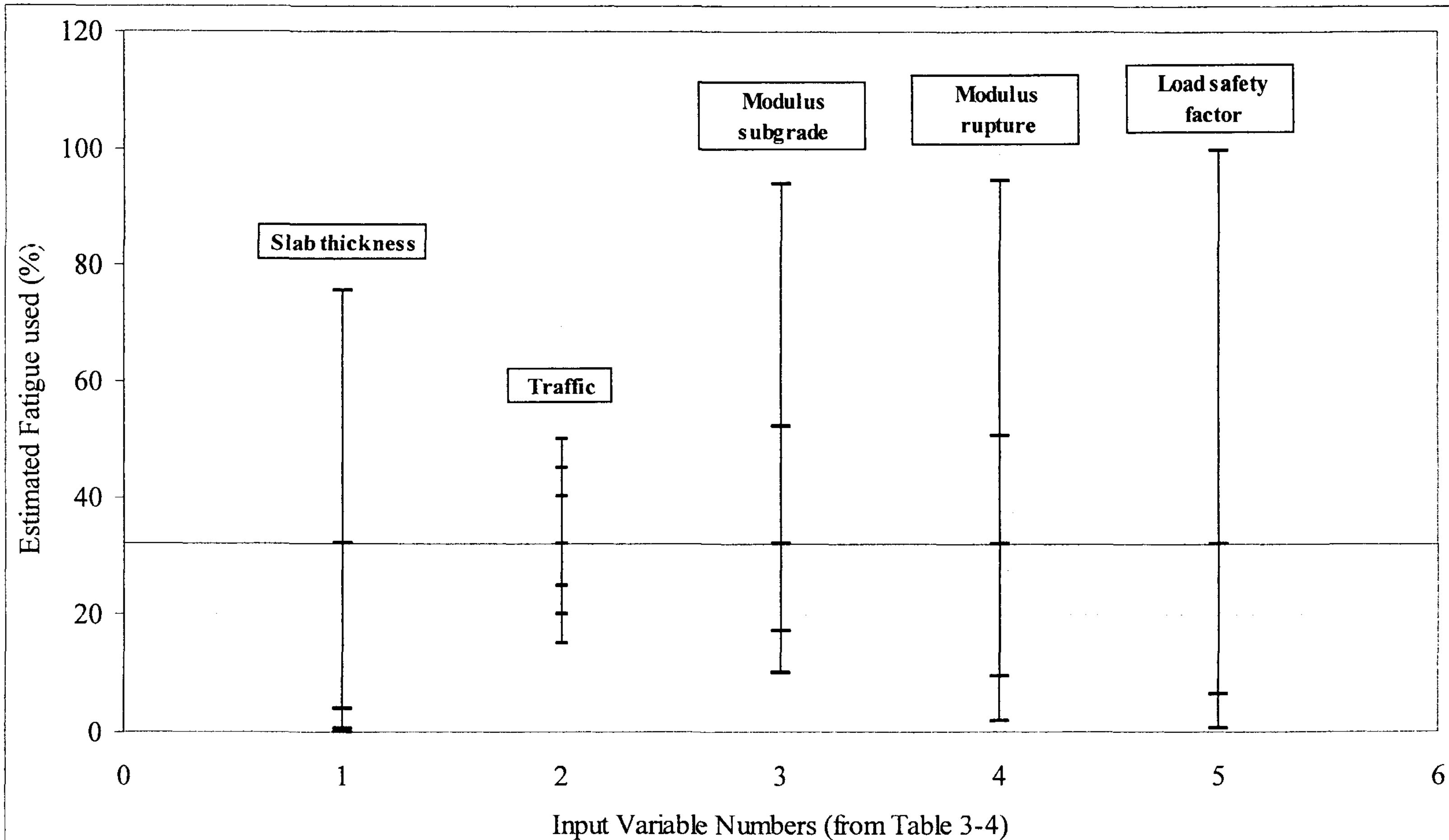

Figure 3-2: Total fatigue used variability due to changes in various design inputs, using PCA design method

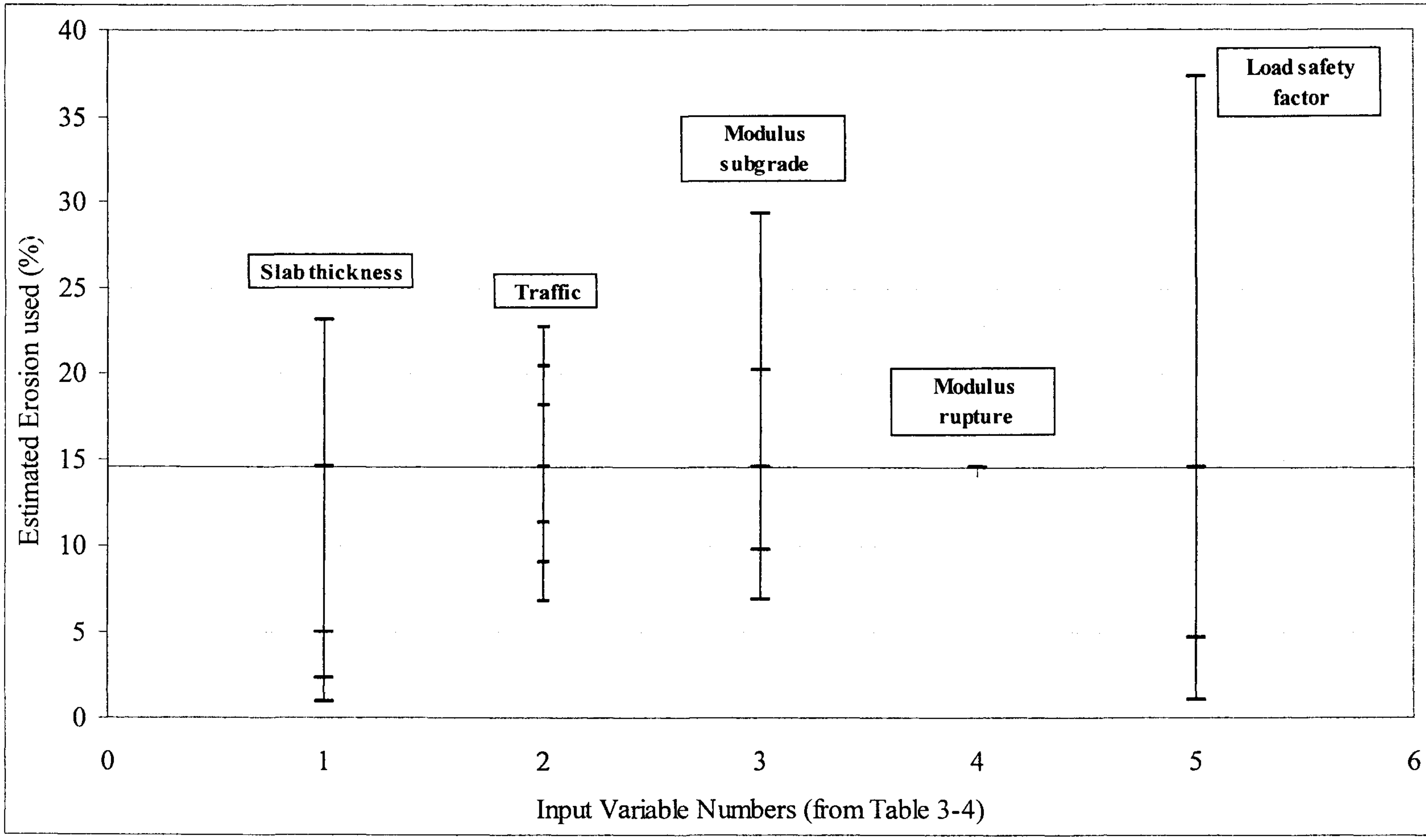

Figure 3-3: Total erosion used variability due to changes in various design inputs, using PCA design method

Considering the fatigue performance of rigid pavement, it can be concluded that the load safety factor (#5), concrete strength (#4), modulus of subgrade reaction (#3) and slab thickness (#1) are the most sensitive parameters. In terms of erosion performance, the most sensitive input parameter is the load safety factor (#5), followed by the modulus of the subgrade reaction (#3), slab thickness (#1), and traffic in terms of AADT (#2). The concrete strength, in terms of the modulus of rupture, does not affect the final erosion of the pavement.

3.4 Sensitivity analysis on DG 2002 for PCCP

There are four input categories in the Design Guide 2002 (DG 2002): general, traffic, climate, and structure inputs. Each category has subcategories and specific inputs. Various input parameters in DG 2002 also have different levels of significance in terms of predicted distress, including roughness expressed in International Roughness Index (IRI), transverse cracking, and faulting.

The purpose of the sensitivity analysis was to identify the most important inputs for each pavement distress. As there are many inputs in DG 2002, identifying more significant factors in pavement performance can help the user to achieve the best design in a shorter amount of time. In addition, it is important to impose higher quality control on the most sensitive parameters during data collection, testing, or construction.

3.4.1 PCCP Distresses in DG 2002

Among all the PCCP distresses, only roughness, transverse cracking, and faulting are considered in the DG 2002 analysis. For each distress, the user may select different reliability levels for performance prediction. The DG 2002 program predicts an increase in IRI and compares the final IRI (at the end of the design life) with the maximum admissible limit that can be established by the user.

The DG 2002 program predicts the percentage of slabs cracked versus time, and compares the ultimate value, at the end of the design life, with the maximum admissible limit established by the user.

The DG 2002 program predicts the faulting increase over time and compares the ultimate value, (at the end of design life) with the faulting limit established by the user.

3.4.2 Traffic inputs

To conduct a sensitivity analysis on DG 2002, several traffic factors were considered. These factors are:

- Initial two-way average annual daily traffic (AADT),
- Percent of heavy vehicles, class 4 or higher (%),
- Percent of trucks in design lane (%),
- Operational speed (km/h), and
- Traffic growth factor (%).

There are others (such as traffic volume adjustment factors, axle load distribution factors, and general traffic inputs) for which the field data were not available. As such, they were considered as constants in the analysis (using default values offered by the program).

3.4.3 Climate inputs

The trial version of the program allows the user to choose between weather stations in the United States. Given the fact that no Canadian climate data is available in the program, data from a Seattle weather station were used, as it was the closest available station to Alberta.

3.4.4 Pavement inputs

The pavement inputs include concrete material properties (MAT), mix design of concrete (MIX), and structural parameters of PCCP (STR). Based on the pavement inputs of the program, the following properties were considered in the sensitivity analysis:

Concrete, base, and subgrade materials and mix properties:

- Unit weight of PCC (kg/m^3) .... MAT
- Poisson's ratio of PCC .... MAT

- Coefficient of thermal expansion of PCC ($10^{-6}/^{o}C$) .... MAT
- Thermal conductivity of PCC (J/m-sec-°C) .... MAT
- Heat capacity of PCC (J/kg-°C) .... MAT
- Aggregate type .... MAT
- Cement type .... MAT
- Base material property, modulus (MPa) .... MAT
- Subgrade material property, modulus (MPa) .... MAT
- Cementitious material content (kg/m^3) .... MIX
- Water/Cement ratio .... MIX

PCC design and structural factors:

- Layer thickness (slab thickness, cm) .... STR
- Joint spacing (m) .... STR
- Dowel diameter (mm) .... STR
- Dowel bar spacing (mm) .... STR
- Slab width (m) .... STR
- Base thickness (mm) .... STR
- Reversible shrinkage of PCC (% of ultimate shrinkage) .... STR
- Time to develop 50% of ultimate shrinkage for PCC (days) .... STR
- Curing method .... STR
- Compression strength of PCC (MPa) .... STR
- Permanent curl/warp effective temperature difference (°C) .... STR

DG 2002 is the only PCCP design procedure that uses several thermal properties of concrete as inputs in the model. These thermal properties are:

- *Coefficient of thermal expansion* of PCC, which is a measurement of a material's expansion or contraction with temperature, is measured by AASHTO TP-60.
- *Thermal conductivity* of PCC, which is a measurement of the ability of the PCC to transfer heat, is measured by CRD-C 36-73.

- *Heat capacity* of PCC, which represents the amount of heat that must be added or removed from a unit mass of PCC to change its temperature by one degree, is measured by CRD-C 124-73.

To conduct a sensitivity analysis on DG 2002, several levels were used for each input parameter. The average values were selected from the design of the AHD project by EBA, which are shown in Table 3-5 in bold. As the design of this project was accomplished using the AASHTO and PCA methods and these methods do not consider all required inputs by DG 2002, typical values were selected for them. All average values (from the AHD design) and two or three levels higher or lower than the average values are shown in Table 3-5. The available trial version of the DG 2002 does not allow the work to be done using the metric system; therefore, the imperial system was used instead. The inputs and the outputs of the DG 2002 were later converted in the metric system.

Table 3-5: Input parameters and the different levels considered in the DG 2002 sensitivity analysis

		Levels						
No.	**Input Parameter**	**Low 1**	**Low 2**	**Low 3**	**EBA Design**	**High 1**	**High 2**	**High 3**
1	Initial IRI (mm/m); 4.5 PSI; IRI(mm/m)=1.5875*(5-PSI)		0.2	0.5	**0.8**	1.1	1.6	2.0
2	Two-way average annual daily traffic, AADT		20,000	25,000	**30,000**	35,000	40,000	50,000
3	Percentage of heavy vehicles (class 4 or higher)		2%	5%	**10%**	20%	30%	50%
4	Percentage of trucks in design lane, %		75%	80%	**85%**	90%	95%	
5	Operational speed, km/h	72	80	88	97	105	113	121
6	Traffic growth factors		0.5%	1.0%	**1.5%**	2.5%	3.5%	5.0%
7	Slab thickness, mm		171	178	**225**	305	381	
8	Unit weight of PCC, kg/m^3		2,243	2,323	2,403	2,483	2,563	
9	Poisson's ratio		0.15	0.17	0.20	0.25	0.30	
10	Coefficient of thermal expansion, $10^{-6}/°C$		2	4	5.5	7	10	
11	Thermal conductivity, J/m-sec-°C		0.9	1.7	2.2	2.6	3.5	
12	Heat capacity, J/kg-°C		670	838	1173	1676	2095	
13	Cement type			Type 2	Type 1	Type 3		
14	Cementitious material content, kg/m^3	237	267	297	**335**	356	386	415
15	Water/Cement ratio		0.30	0.35	**0.40**	0.45	0.50	
16	Aggregate type			Quartzite	Limestone	Dolomite	Granite	Rhyolite
17	Reversible shrinkage, % of ultimate shrinkage		30%	40%	50%	60%	70%	80%
18	Time to develop 50% of ultimate shrinkage, days		30	33	35	40	45	50
19	Curing method				**Compound**	Wet		
20	Compression strength f'c (Level 3), MPa	21	24	28	**30**	34	41	48
21	Permanent curl/wrap effective temperature difference, °C	-30	-29	-26	-23	-21	-18	
22	Joint spacing, m		3.0	3.7	**4.5**	5.2	6.1	
23	Dowel diameter, mm		25	28	**32**	36	43	
24	Dowel bar spacing, mm		254	279	**300**	330	356	
25	Slab width, m				**3.7**	4.0	4.3	
26	Base thickness, mm	76	102	127	150	178	203	229
27	Base material property (modulus), MPa		265	269	276	282	289	
28	Subgrade material property (modulus), MPa				34	48	62	83

* Bold values are from EBA Design for Anthony Henday Drive Project

3.4.5 DG 2002 sensitivity analysis results

Among the 28 input parameters, 14 of them (see Table 3-5, "EBA Design" column, the bolded values) were the values designed for the Anthony Henday Drive project in Edmonton. The remaining 14 input parameters considered typical values. Changes in pavement performance, in terms of IRI, TC, and faulting, were estimated by running the DG 2002 software. The program was run by changing only one input parameter at a time, considering all the other parameters as constants. The reliability level considered in the analysis was 95%. Design reliability is defined as the probability that each of the key distress types will be less than a selected critical level over the design period. The DG 2002 program was run more than a hundred times in order to calculate the changes in the performance of pavement due to changes in all input parameters. The results of each run consist of an analysis in terms of roughness (IRI), transverse cracking, and faulting.

Figure 3-4 presents the results of the program runs, for the final IRI, with a reliability level of 95%. The plot presents the relevance of each input parameter to the final IRI. In the following sensitivity analyses, the term "Final" will be used to define the distress level at the end of the 30-year service life of the pavement (i.e., the Final IRI).

There are seven very sensitive inputs on the final IRI. These design parameters are the curl/warp temperature difference of PCC, the coefficient of thermal expansion of PCC, initial IRI, layer thickness, dowel diameter, percent of heavy vehicles, and joint spacing. From Figure 3-4 it can also be observed that the permanent curl/warp effective temperature difference and coefficient of thermal expansion are the most significant input parameters and increase the final IRI beyond the IRI limit of 4.0 mm/m. Initial IRI, slab thickness, joint spacing, and dowel diameter are also significant input parameters and have a high impact on the final IRI, bringing it beyond the IRI limit of 3.0 mm/m.

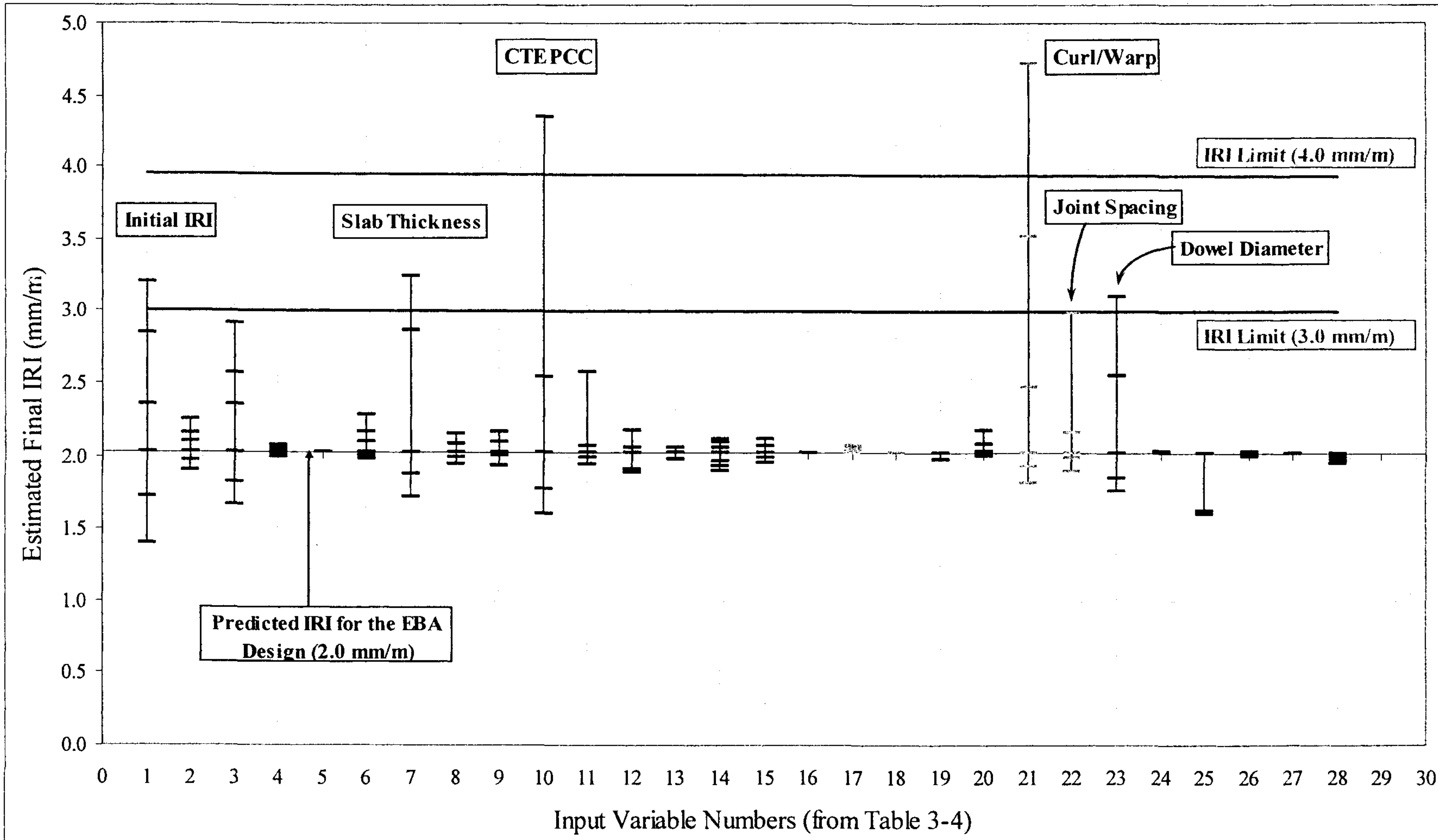

Figure 3-4: Changes in IRI due to changes of various design inputs predicted by DG 2002

DG 2002 software was used to predict changes in pavement transverse cracking (TC) by changing 28 input parameters. Figure 3-5 presents the impact of each input parameter on the transverse cracking at the end of the 30-year design life, with a 95% reliability level. It can be observed that there are five input parameters having significant impacts on the TC. The five high impact parameters are layer thickness, coefficient of thermal expansion, thermal conductivity, permanent curl/warp effective temperature difference, and joint spacing. The other three input parameters with high impact on the final TC are the percentage of heavy vehicles, the heat capacity, and the compressive strength of concrete. If a maximum of 15% TC is considered as maximum acceptable for performance of PCCP, eight parameters could bring the TC level beyond this maximum acceptable limit.

DG 2002 software was used to predict changes in pavement faulting distress by changing 28 input variables. Figure 3-6 presents faulting distress changes in the results, with 95% reliability. There are four parameters that have significant impacts on the faulting of PCCP. From Figure 3-6, it can be observed that the coefficient of thermal expansion and the permanent curl/warp effective temperature difference could cause a faulting above the acceptable limit of 7 mm. The dowel diameter and the percentage of heavy vehicles may bring the faulting to or above 5 mm.

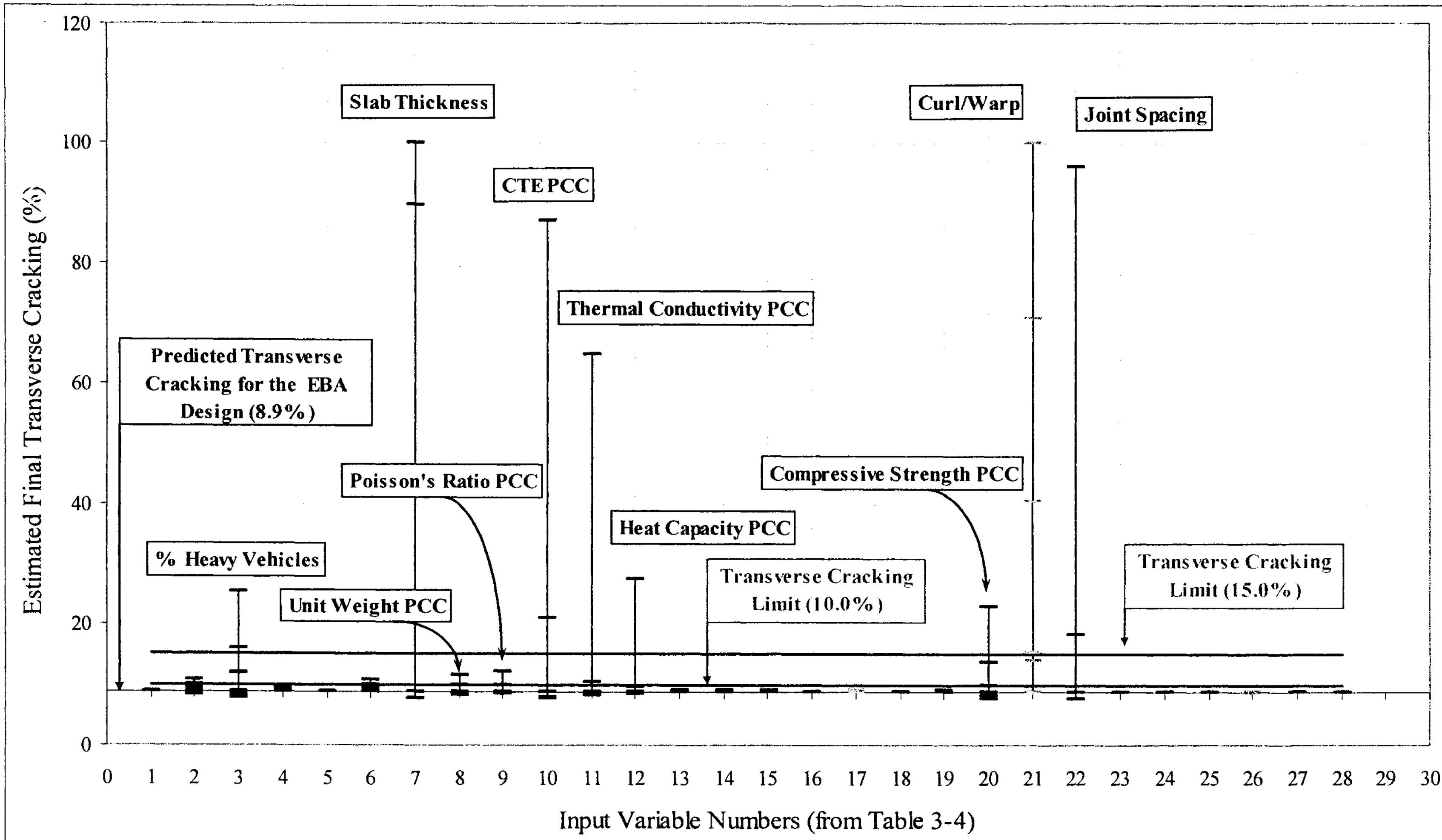

Figure 3-5: Changes in TC due to changes in various design inputs predicted by DG 2002

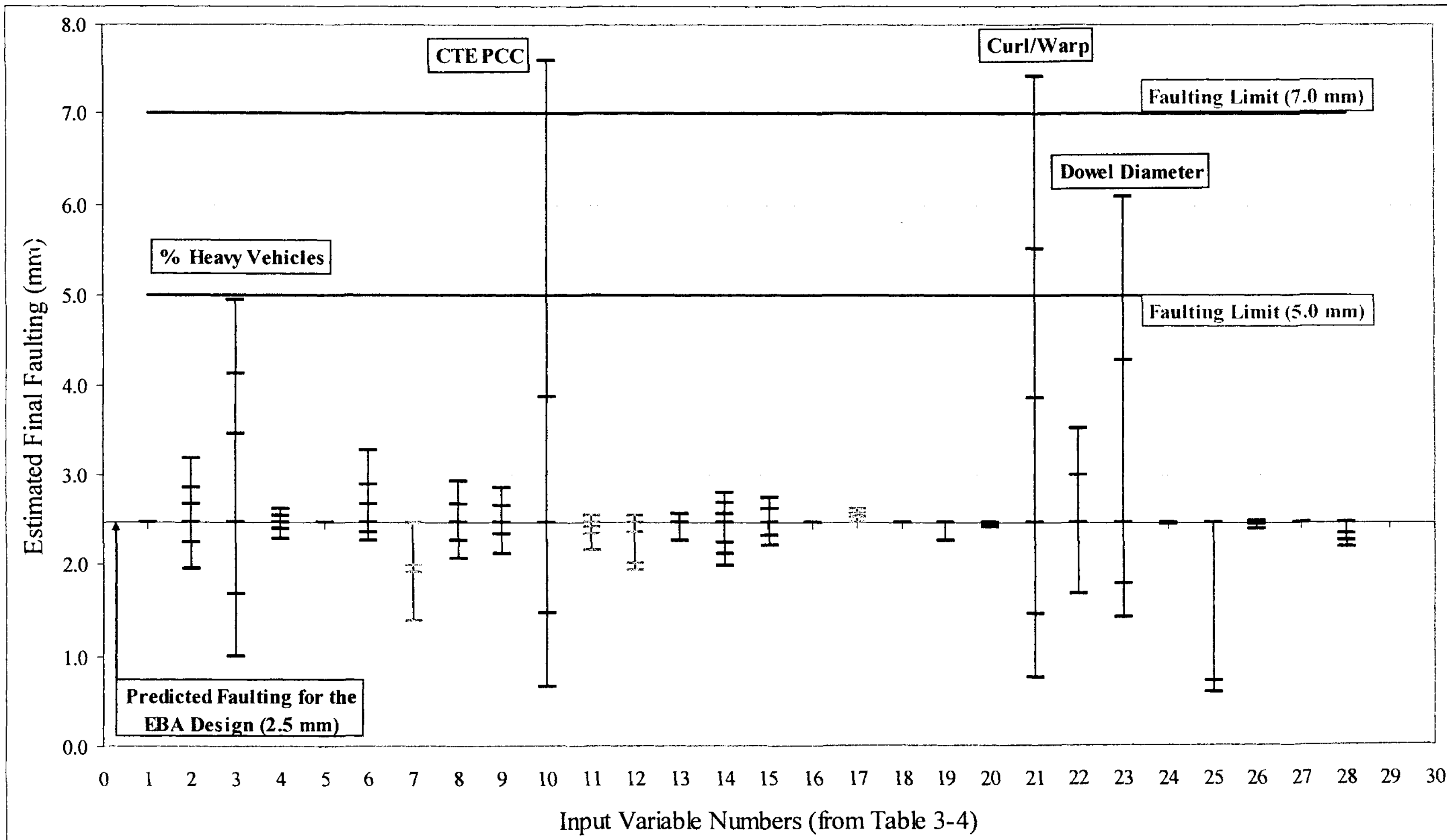

Figure 3-6: Faulting variability due to changes in various design inputs, using DG 2002

3.4.6 Summary of DG 2002 sensitivity analysis

Considering the performance of a PCCP in terms of IRI, TC, and faulting, the most significant input parameters are presented in Tables 3-6, 3-7, and 3-8, and graphically shown in Figures 3-7, 3-8, and 3-9. Bold numbers in Tables 3-6 to 3-8 show input values corresponding to the maximum performance predicted indicators.

Table 3-6: Most sensitive input parameters on pavement roughness (IRI)

Input Parameters	Unit	Low Value	High Value	Maximum predicted IRI after 30 years, mm/m
Permanent curl/warp temperature difference of PCC	°C	**-32**	-21	**4.7**
Coefficient of thermal expansion of PCC	10^{-6}/°C	2	**10**	**4.4**
Initial IRI	mm/m	0.2	**2.0**	**3.2**
Slab thickness	mm	**152**	381	**3.2**
Dowel diameter	mm	**25**	43	**3.1**
Joint spacing	m	3.7	**6.1**	**3.0**

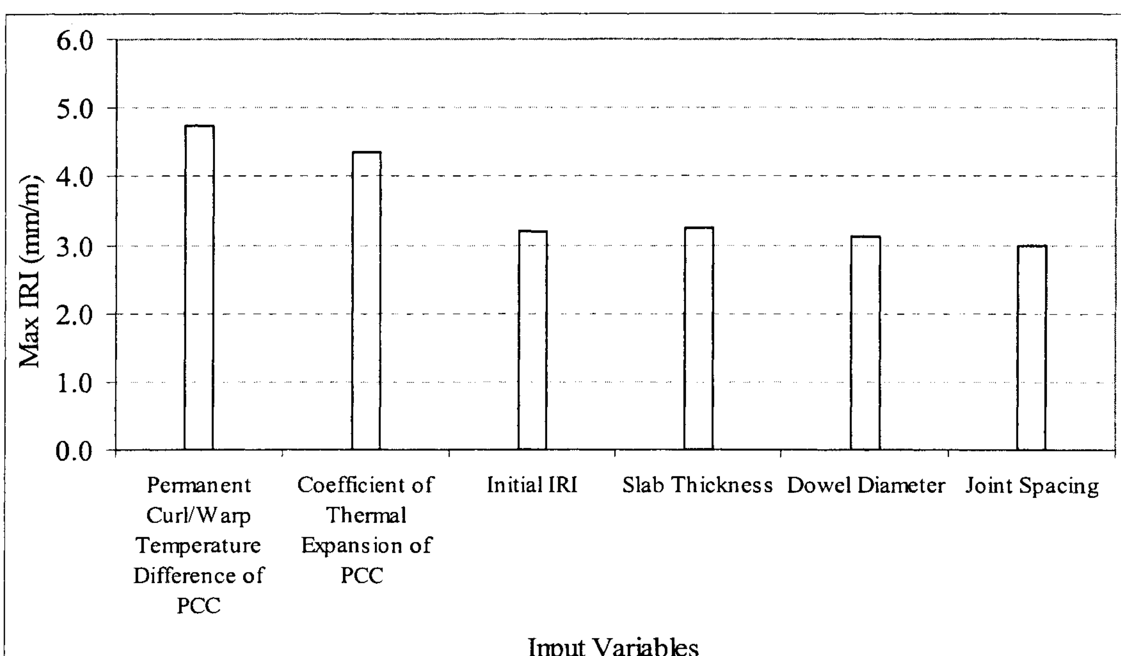

Figure 3-7: Most sensitive input parameters on pavement roughness (IRI)

Table 3-7: Most sensitive input parameters on pavement transverse cracking (%)

Input Parameters	Unit	Low Value	High Value	Maximum predicted TC after 30 years, %
Slab thickness	mm	**152**	381	**99.9**
Permanent curl/warp temperature difference of PCC	°C	**-32**	-21	**99.9**
Joint spacing	m	3.7	**6.1**	**96.2**
Coefficient of thermal expansion of PCC	10^{-6}/°C	2	**10**	**87.2**
Thermal conductivity of PCC	J/m-sec-°C	**0.9**	3.5	**65**
Heat capacity of PCC	J/kg-°C	**670**	2095	**27.6**
Percentage of Heavy Vehicles	%	2	**50**	**25.5**
Compressive strength of PCC	MPa	**21**	48	**22.9**
Poisson's ratio of PCC	-	0.15	**0.30**	**12.1**
Unit weight of PCC	kg/m^3	83	**95**	**11.6**
Two-way average annual daily traffic	-	20,000	**50,000**	**10.8**
Traffic growth factors	%	0.5	**5.0**	**10.8**

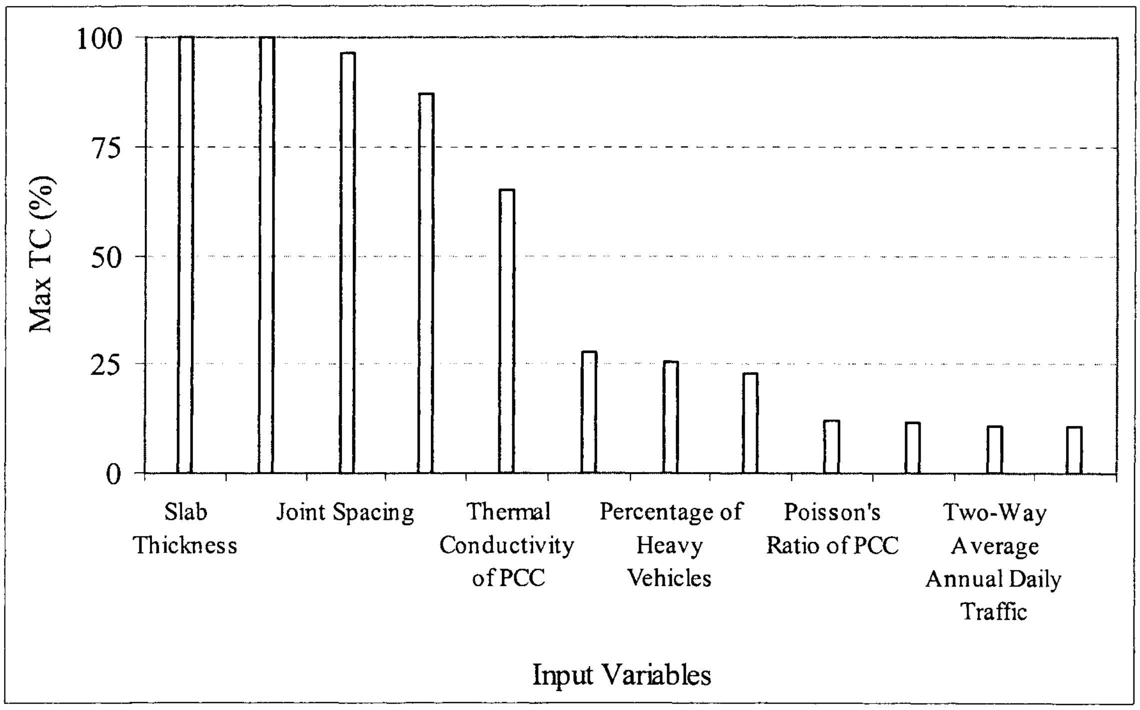

Figure 3-8: Highest TC variability and most important input parameters

Table 3-8: Most sensitive input parameters in pavement faulting (mm)

Input Parameters	Unit	Low Value	High Value	Maximum predicted faulting after 30 years, mm
Coefficient of thermal expansion of PCC	10^{-6}/°C	2	**10**	**7.6**
Permanent curl/warp temperature difference of PCC	°C	**-32**	-18	**7.4**
Dowel diameter	mm	**25**	43	**6.1**
Percentage of heavy vehicles	%	2	**50**	**5.0**

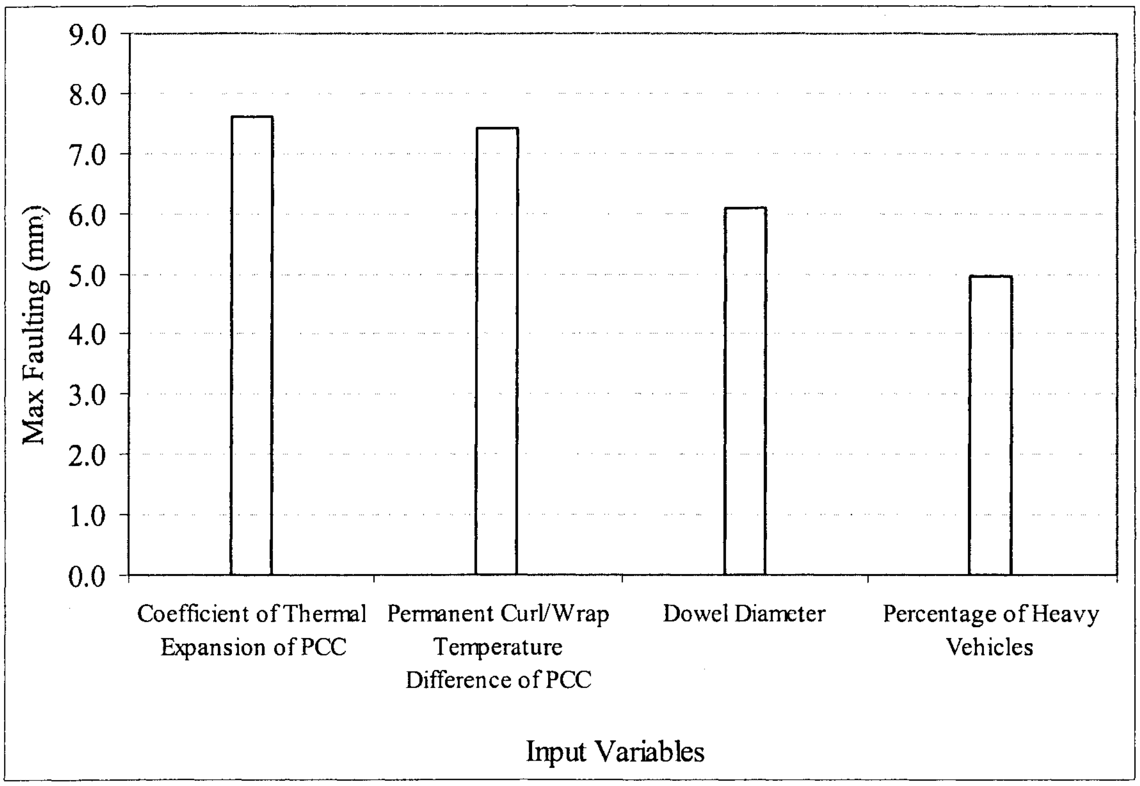

Figure 3-9: Most sensitive input parameters in pavement faulting (mm)

Table 3-9 summarizes the important input parameters for the three main concrete pavement distresses predicted by the DG 2002.

Table 3-9: Summary of the most important input parameters for PCCP performance

Input Parameter	Level of Significance	IRI	Transverse Cracking	Faulting	Ranking of Variables
Coefficient of thermal expansion of PCC	I	X	X	X	3X
Permanent curl/warp temperature difference of PCC	I	X	X	X	3X
Percentage of heavy vehicles	II		X	X	2X
Slab thickness	II	X	X		2X
Joint spacing	II	X	X		2X
Dowel diameter	II	X		X	2X
Initial IRI	III	X			1X
Thermal conductivity of PCC	III		X		1X
Heat capacity of PCC	III		X		1X
Compressive strength of PCC	III		X		1X
Poisson's ratio of PCC	III		X		1X
Unit weight of PCC	III		X		1X
Two-way average annual daily traffic	III		X		1X
Traffic growth factors	III		X		1X

The coefficient of thermal expansion and the permanent curl/warp effective temperature difference of concrete have the highest impacts on all three distresses considered by the DG 2002 program. There are other main input parameters such as the percentage of heavy vehicles, slab thickness, joint spacing, and dowel diameter, all of which have a high impact in two of the three distresses predicted. The third category of high impact factors in PCCP design using DG 2002 are the initial IRI, thermal conductivity, heat capacity, compression strength, Poisson ratio, unit weight of PCC, traffic AADT, and the traffic growth factor, all of which have a high impact in only one of the three distresses.

The Coefficient of Thermal Expansion (CTE) values for different concretes reflect the variation in CTE of concrete's component materials. Because aggregate comprises about 70% of the concrete, aggregate type has the greatest effect upon the CTE of concrete. The CTE of hardened cement paste, which is a function of factors such as w/c ratio, cement fineness, cement composition, and age, also affects the CTE of concrete. These factors make the CTE a controllable factor in the design of PCC.

Permanent curl/warp temperature difference is related to the temperature difference between the upper face and the bottom face of the concrete slab. This difference leads to curling (upper face temperature greater than the bottom face temperature) or warping (upper face temperature lower than the bottom face temperature). Since this temperature is mainly a function of the weather, it cannot be controlled; therefore, no control over the performance of PCC from this perspective can be achieved.

The percentage of heavy vehicles, as an input factor in DG 2002, represents a predicted value. This makes the percentage of heavy vehicles as an uncontrollable factor by designer in the performance of PCC using DG 2002.

The layer thickness, joint spacing, and dowel diameter are designed elements; they are controllable factors in PCC design. The initial IRI is a construction-related parameter and could therefore be controlled by improving construction methods and equipment. Thermal Conductivity of concrete depends on the aggregate's internal microstructure and its mineralogical composition making thermal conductivity a controllable factor in the selection of materials for PCC.

3.5 Comparison between findings of this study and other studies

A review of the available literature found two other studies focusing on the area of sensitivity analysis for DG 2002 rigid pavement design. As the methodology of sensitivity analysis changes, it is possible to find other results. A comparison of the sensitivity analysis results from this study with other studies from this area was therefore implemented. Hall and Beam (2005) conducted a sensitivity analysis on DG 2002 rigid pavement design. A total of 29 inputs were evaluated by analyzing a standard pavement section and changing the value of each input individually. The three pavement distress models (cracking, faulting, and roughness) were not sensitive to 17 of the 29 inputs. All three distress models were sensitive to 6 of 29 inputs.

The criteria used to judge the "significance" of differences in distress predictions (i.e., sensitivity) were as follows:

- For the faulting model, differences (across the range of input values) in total faulting after 20 years exceeding 2.54 mm were judged significant,
- For the cracking model, differences (across the range of input values) in percentage of slabs cracked after 20 years exceeding 25 percent were judged significant, and
- For the smoothness model, differences (across the range of input values) after 20 years exceeding 0.47 mm/m were judged significant.

The specific numerical criteria used for judging significance were chosen arbitrarily, primarily based on the authors' experiences. Certainly, the use of different criteria would greatly affect the judgment of sensitivity.

The input factors found as significant in the PCC performance by DG 2002, were curl/warp temperature differences, joint spacing, dowel diameter, edge support, surface shortwave absorptivity, slab thickness, unit weight of PCC, Poisson's ratio of PCC, coefficient of thermal expansion of PCC, thermal conductivity, and concrete strength.

The most important findings from this study were:

- Based on the data generated in this study, few design inputs affect all performance prediction models.

- Many variables introduced by the DG 2002 that were not explicitly considered in previous PCC pavement design procedures do not appear to affect significantly the prediction of pavement performance in the DG 2002. In such cases, the use of the default value included in the software provides adequate results.
- Some variables (such as the coefficient of thermal expansion, thermal conductivity, and heat capacity) introduced by the DG 2002, which were not explicitly considered in previous PCC pavement design procedures, appear to affect the prediction of pavement performance in the DG 2002 significantly. In such cases, the use of the default value included in the software may not provide adequate results; designers are encouraged to determine a reasonable value for such variables consistent with the local situation (Hall and Beam 2005).

Guclu and Ceylan (2005) conducted a sensitivity analysis and identified the sensitivity of input parameters in designing jointed plain concrete pavements used in the DG 2002. The paper identifies input parameters ranging from "most sensitive" to "insensitive" for three critical rigid pavement performance measurements: faulting, transverse cracking, and smoothness. After varying each input parameter in the expected range, the plotted outputs of the program (time development of each of the three distresses) were visually inspected. The evaluation was made according to the pavement performance values as well as the degree of change in the pavement performance value due to the changing input variables. The input factors which were found sensitive to extremely sensitive in the PCC performance predicted by DG 2002 were permanent curl/warp temperature difference, joint spacing, edge support, slab thickness, unit weight of PCC, Poisson's ratio of PCC, coefficient of thermal expansion of PCC, thermal conductivity of PCC, cement content, water/cement ratio, concrete strength, mean wheel location (traffic wander), climate, surface shortwave absorptivity, AADT traffic, doweled transfer joints, and unbound layer modulus. The curl/warp effective temperature difference (built-in curling and warping of the slabs) and PCC thermal properties were found to be the most sensitive input parameters.

Table 3-10 summarizes the sensitivity analysis results from this study and the two reviewed papers.

Table 3-10: Summary of sensitivity analysis results on DG 2002

Input Parameters	Performance Criteria								
	IRI			TC			Faulting		
	This study	Hall and Beam	Guclu and Ceylan	This study	Hall and Beam	Guclu and Ceylan	This study	Hall and Beam	Guclu and Ceylan
28-day PCC compressive strength		S	S	S	S	S			
28-day PCC modulus of rupture		S	S		S	S			
Cement content			S						S
Climate						S			
Coefficient of thermal expansion of PCC	S	S	S	S	S	S	S	S	S
Doweled transverse joints			S						S
Dowell diameter	S	S					S	S	
Edge support		S			S	S		S	
Heat capacity of PCC				S					
Initial IRI	S								
Joint spacing	S	S	S	S	S			S	
Mean wheel location (Traffic wander)			S			S			S
Percentage of heavy vehicles				S			S		
Permanent curl/warp temperature difference	S	S	S	S	S	S	S	S	S
Poisson's ratio of PCC			S	S	S	S			
Slab thickness	S	S	S	S	S	S		S	
Surface shortwave absorptivity			S		S	S			
Thermal Conductivity of PCC			S	S	S	S			S
Traffic Growth Factors				S					
Two-way average annual daily traffic			S	S		S			S
Unbound layer modulus			S						S
Unit weight of PCC		S		S	S	S		S	
Water/cement ratio			S						S

S = most sensitive inputs parameters

Table 3-10 indicates that concrete thermal properties were found to be the most significant input parameters in all three studies, having the highest impact on all three PCC performance criteria. Several other input parameters were found to be important parameters in two or three of the mentioned studies. The differences in results may come from different ranges for input values, different average values considered, different climate data used, and the subjectivity in methodology for the selection of the maximum acceptable limit.

3.6 Comparison of PCC design methods by sensitivity analyses

In terms of input parameters, the existing rigid pavement design methods use a limited number of inputs (see Tables 3-1 and 3-3) compared to the number of inputs from DG 2002 (see Table 3-5):

- Traffic as an input parameter in the AASHTO 1993 method is expressed in accumulated 18-kip (80-kN) Equivalent Single Axle Loads (ESAL) over the life of the project; that is, as a single traffic characteristic. In the PCA method, traffic is expressed in Average Daily Truck Traffic (ADTT) and is calculated based on axle distribution per axle load (ten axle loads) for single and tandem types of axles. In DG 2002, traffic is considered by all its characteristics, including all 28 inputs. The most important traffic inputs in DG 2002 are the traffic as two-way AADT, percentage of heavy vehicles, percentage of trucks, operational speed, and traffic growth factors.
- Climate input parameters are directly considered in DG 2002 as multiple local weather characteristics (rain, wind, air temperatures, etc.), while in AASHTO and PCA methods they are not considered as direct parameters.
- Regarding the concrete material from the pavement, in PCA and AASHTO rigid pavement design methods, only the modulus of rupture characterizes the concrete, while in DG 2002 many other concrete characteristics are used (see inputs 8 to 21 in Table 3-5). Also, more pavement characteristics (see inputs 22 to 25 from Table 3-5) are used in DG 2002 to describe the concrete slab dimensions and the connections between them.
- One important element in DG 2002 is the introduction of thermal properties for concrete (coefficient of thermal expansion, thermal conductivity, and the heat capacity of the PCC) in rigid pavement design. This introduction comes as a result of the sensitivity analysis indicating that they are significant factors, highly affecting the pavement performance.

AASHTO and PCA pavement design guides give indications of pavement performance at specific traffic levels. Therefore, to have a full performance prediction, it is necessary to run them several times at different traffic levels. DG 2002 provides a full performance prediction for each run. The sensitivity analysis results for the AASHTO 1993 method show that the pavement thickness and strength of the concrete are the most important input parameters. For the PCA method, sensitivity analysis indicated that all input parameters are important as erosion criteria, except for the concrete strength. In the PCA design method, the variation of the modulus of rupture (M_R) does not modify the erosion distress.

The sensitivity analysis for the DG 2002 method revealed that thermal properties of the concrete (coefficient of thermal expansion), permanent curl/warp temperature difference, and heavy traffic (percentage of heavy vehicles) are the most important factors. Other important input parameters in DG 2002 are pavement thickness, joint spacing, dowel diameter, initial IRI, thermal conductivity, heat capacity, and the compressive strength of PCC.

CHAPTER 4: CHARACTERIZATION OF A TYPICAL PCCP FOR VALIDATION AND CALIBRATION OF THE DESIGN GUIDE 2002 IN ALBERTA

Over the past 50 years, pavement design has relied mainly on empirical procedures that have improved over time but still have significant deficiencies. Due to these deficiencies, the National Cooperative Highway Research Program (NCHRP) 1-37A project team has developed a new mechanistic-empirical pavement design method, the Design Guide 2002 (DG 2002). This design method has been developed based on findings from the Long Term Pavement Performance (LTPP) study and other research studies in this area.

As the DG 2002 has been developed using pavement performance measurements obtained primarily from the LTPP study in the USA, it needs local and regional calibration to better represent local conditions. It is not likely that the relative sensitivity of input design factors in the DG 2002 will change with local calibration; however, the local calibration of DG 2002 is important (Hall 2005). Many highway agencies have started to calibrate/validate traffic, materials and environmental aspects of DG 2002 as a required step for the future implementation of DG 2002.

DG 2002 is structured in a hierarchical manner with three pavement design levels. It is predicted that Design Level 1 will not be implemented, except for a limited number of high traffic highways, due to the excessive cost of material characterization and traffic data collection. Design Level 2 would be used for projects on primary and selected secondary routes. Design Level 3 is the least accurate level of the DG 2002. The Design Level 1 requires real measurement data input while Design Levels 2 and 3 use certain models and typical values. This type of structure gives agencies more flexibility in designing their projects.

The Design Guide 2002 suggests certain pavement performance models and typical values for the properties of PCC to be used as inputs in the rigid pavement design. It is necessary to validate the models and typical values suggested by DG 2002. A PCCP project in Edmonton, Alberta was selected for material calibration of DG 2002. This is one of the first attempts in implementation of DG 2002 for a Canadian highway agency.

4.1 Anthony Henday Drive, first PCCP Project in Edmonton

Alberta Infrastructure & Transportation (AI&T) is working on an extension of Anthony Henday Drive from the Yellowhead Trail in northwest Edmonton to Calgary Trail in south Edmonton. Anthony Henday Drive South West is part of the North-South Trade Corridor and Edmonton's Ring Road (Figure 4-1).

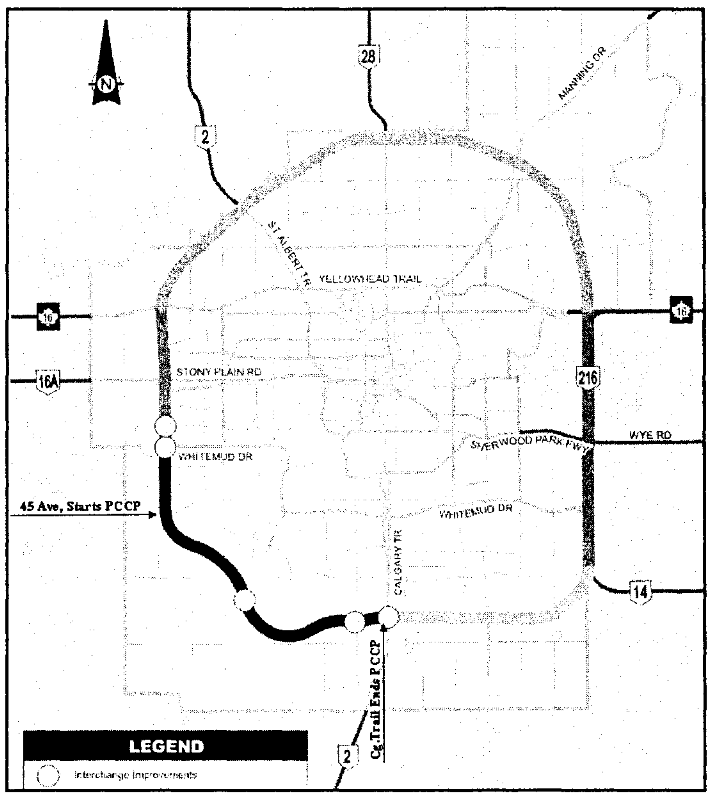

Figure 4-1: Anthony Henday Drive, part of the South-West Edmonton ring road

The project is a four-lane, 15-km divided highway, with five intersections, having a design pavement structure of 225 mm concrete slab over 150 mm granular base course. This 30-year design life project has an estimated construction cost of $245 million. It involves the moving of 10 million cubic meters of earth, and uses 230,000 metric tonnes of crushed gravel and uses over 100,000 cubic meters of concrete. The pavement section is the first PCCP in Alberta following the construction of a section of the Deerfoot Trail in Calgary, which was constructed 15 years ago.

EBA Engineering used two methods for PCCP design: PCA was the primary design tool and AASHTO was used to check the PCA results (EBA 2003). The results (designed slab thickness) were 225 mm and 312 mm thick concrete slab using the PCA and AASHTO pavement design methods, respectively, for a total 36,700,000 Equivalent Single Axle Loads (ESAL) during its service life. The final concrete slab thickness was 225 mm based on PCA design method.

4.2 Scope and objective

The main objective of this research was to facilitate the implementation of DG 2002 rigid pavement design by characterizing a typical Alberta PCC pavement. Two sets of concrete samples were collected from this project and tested for all required material inputs in the DG 2002.

4.3 Testing program

In the case of PCCP, compressive strength (f'_c), modulus of elasticity (E_c), modulus of rupture (M_R), indirect tensile strength (f_t), the coefficient of thermal expansion (α), Poisson's ratio (η), and shrinkage strain (ε_c) are the main material inputs in the DG 2002. Two sets of concrete samples were tested. The first concrete sample was taken on June 7, 2005 and the second set of concrete samples was collected on August 4, 2005 in order to determine the impact, if any, of variability in concrete properties. Testing results were compared with suggested models and the typical values from the DG 2002. Table 4-1 shows the experimental design of this study.

Table 4-1: Testing program for each set of concrete samples

Testing Procedures / Standard	Testing Ages	Number of Samples	Validation
Compressive strength (f'_c) ASTM C39	7, 14, 28, and 90 days	10 cylinders	Compressive strength gain curve
Modulus of elasticity (E_c) and Poisson's ratio (η) ASTM C469	7, 14, 28, and 90 days	10 cylinders	$E_c=33\rho^{3/2}(f'_c)^{1/2}$ and elastic modulus gain curve η=0.15-0.18
Modulus of rupture (M_R) ASTM C78	7, 14, 28, and 90 days	10 beams	$M_R=9.5(f'_c)^{1/2}$ and modulus of rupture gain curve
Indirect tensile strength (f_t) ASTM C496	7, 14, 28, and 90 days	10 cylinders	Tensile strength gain curve
Coefficient of thermal expansion (α) AASHTO TP60	28 days	3 cylinders	Typical values
Shrinkage (ε_c) AASHTO T160	Immediately after demolding and will continue to 35 days	3 beams	$\varepsilon_c=C_1C_2(26w^{2.1}(f'_c)^{0.28}+270)$
Unit weight (ρ) ASTM C642	On fresh and hardened concrete	2 samples	Typical ranges
Thermal conductivity CRD-C 36-73	28 days	2 samples	0.865 to 3,462 J/m-sec-°C
Heat capacity CRD-C 124-73	28 days	2 samples	670 to 2,095 J/kg-°C

4.4 Concrete samples collection and curing

From the construction site of the Anthony Henday Drive (AHD) project in Edmonton, two sets of concrete, each including 50 cylinders (100×200 mm) for testing compressive strength, modulus of elasticity, indirect tensile strength, unit weight, coefficient of thermal expansion, ten beams (153×153×563 mm) for testing modulus of rupture, and six beams (77×77×286 mm) for evaluating shrinkage were collected from the batch of concrete. The concrete samples were collected according to ASTM C31. They were cured in a moist room (temperature 23±2°C, humidity 95 to 100%) according to ASTM C511. The concrete specimens for testing compressive strength, modulus of elasticity, and Poisson's ratio were capped with sulphur before being tested according to ASTM C617. The concrete properties and the ingredients used for the AHD concrete project are presented in Table 4-2.

Table 4-2: Mix design of the concrete

Concrete Properties and Ingredients	**Value, Unit**
Compressive strength at 28 days	min. 30 MPa
Slump	40±20 mm
Air content	5 to 8%
Water/Cementitious material (w/cm)	max 0.45
Maximum aggregate size	25 mm
Cement content (Type I)	300 kg/m^3
Fly ash	35 kg/m^3
Water	120 kg/m^3

Three aggregate sources were combined, as is shown in Table 4-3, to achieve a combined aggregate gradation in Figure 4-2.

Table 4-3: Aggregates used for the AHD concrete project (% of the weight)

	Aggregate type	**20-25 mm**	**5-14 mm**	**Sand**	**Total**
	Proportions	**23.0%**	**42.2%**	**34.8%**	**100%**
Sieve Size (mm)	25	98.36	100.00	100.00	99.62
	20	46.39	100.00	100.00	87.68
	14	2.89	93.22	100.00	74.83
	12.5	1.53	73.34	100.00	66.14
	10	0.57	53.47	99.91	57.51
	5	0.27	2.18	93.36	33.53
	2.5	0.26	0.50	81.15	28.56
	1.25	0.24	0.00	73.23	25.59
	0.63	0.21	0.00	65.15	22.76
	0.315	0.18	0.00	25.97	9.10
	0.16	0.13	0.00	4.93	1.75
	0.08	0.06	0.00	0.75	0.28

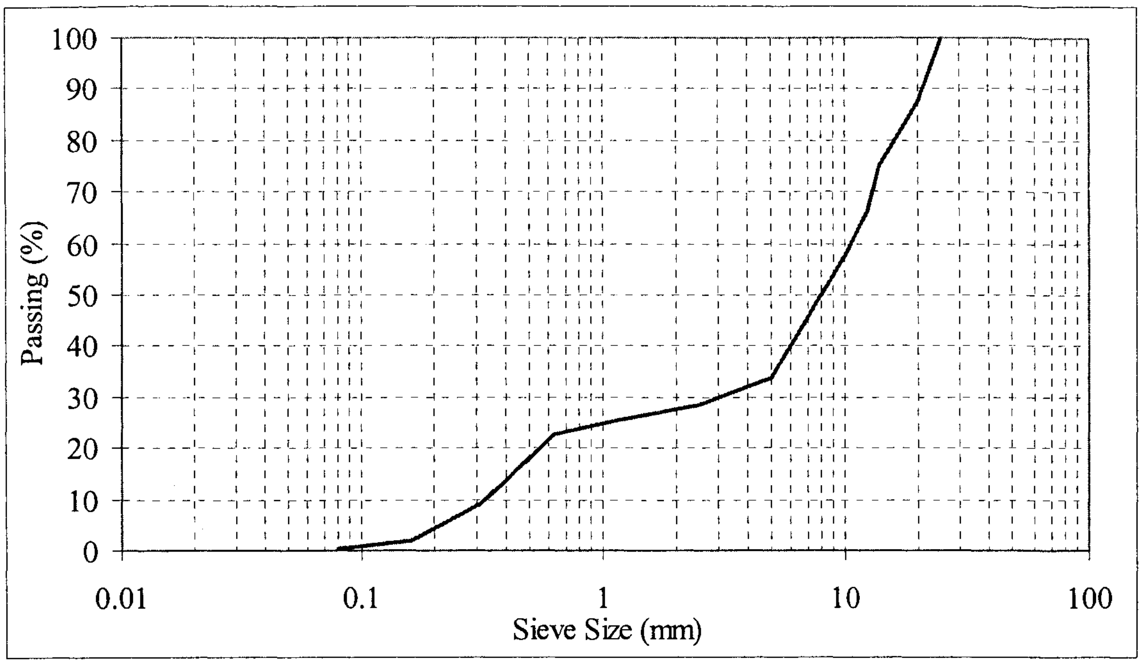

Figure 4-2: Total aggregate gradation for AHD concrete mix

4.5 Laboratory concrete characterization

All concrete material properties required in DG 2002 (Table 4-1) were measured for both sets of concrete samples. These measured properties were used for predicting the performance of the AHD project. In addition, these measured properties were compared with suggested models and values from the DG 2002.

4.6 Summary of concrete testing results

A summary of concrete testing results is presented in Table 4-4.

Table 4-4: Summary of concrete testing results from the AHD project

		7 days		**14 days**		**28 days**		**90 days**	
Tests [unit]	**Sample**	**Set 1**	**Set 2**	**Set 1**	**Set 2**	**Set 1**	**Set 2**	**Set 1**	**Set 2**
Compressive strength (f'_c) [MPa]	1	29.7	26.1	34.1	28.3	37.9	34.9	40.5	40.7
	2	30.7	24.1	35.1	28.3	37.2	35.4	39.7	41.3
Modulus of elasticity (E_c) [MPa]	1	-	-	23,431	22,851	23,995	24,821	27,538	31,773
	2	-	22,349	23,624	23,792	25,907	24,515	-	25,417
Poisson's ratio (η)	1	0.21	0.14	0.16	0.17	0.16	0.16	0.15	0.21
	2	0.17	0.16	0.15	0.17	0.17	0.17	0.19	0.19
Modulus of rupture (M_R) [MPa]	1	-	3.29	4.14	3.65	4.75	3.70	-	4.80
	2	3.67	2.96	-	3.33	4.53	3.75	-	4.72
Indirect tensile strength (f_t) [MPa]	1	3.11	2.61	3.61	-	3.51	3.69	-	4.12
	2	2.97	2.43	3.42	2.72	3.61	3.52	3.73	4.23
Coefficient of thermal expansion (α) [10^{-6}/°C]	1	-	-	-	-	12.81	12.36	-	-
	2	-	-	-	-	12.91	12.69	-	-
Unit weight (ρ) fresh and 28-day [kg/m^3]	1	2,262	-	-	-	2,286	2,263	-	-
	2	-	-	-	-	2,279	2,269	-	-
Thermal conductivity [J/m×s×°C]	1	-	-	-	-	1.21	0.94	-	-
	2	-	-	-	-	1.26	0.95	-	-
Heat capacity [J/kg×°C]	1	-	-	-	-	947	882	-	-
	2	-	-	-	-	953	886	-	-
Shrinkage [strain×10^{-6}]	1	-	-	-	-	745	544	-	-

An explanation for each of the concrete properties and testing results compared with typical values and models from DG 2002 are presented in the following sections.

4.6.1 Unit weight of PCC (ρ)

The fresh unit weight of a concrete sample for set 1 was 2,262 kg/m^3. The unit weights of concrete samples after 28 days were 2,283 and 2,266 kg/m^3, determined according to ASTM C642 (Appendix A-1). These parameters must be measured for levels 1 and 2 in DG 2002. Design Level 3 of DG 2002 recommends a typical range of 2,243 to 2,563 kg/m^3. Therefore, the unit weight of concrete measured in Alberta is below the average and close to the lower value suggested by the DG 2002. This is based also on another study undertaken for a High Performance Concrete (HPC) project in Edmonton, which found the unit weight of HPC lower than values suggested in textbooks (Soleymani 2006). This could be due to the lighter aggregates used in Northern Alberta for construction projects.

4.6.2 Compressive strength of PCC (f'c)

The compressive strength of concrete, measured according to ASTM C39, is used to estimate other properties such as the modulus of elasticity, modulus of rupture, and indirect tensile strength at Design Levels 2 and 3. Compressive testing results for two sets of samples from the AHD project are reported in Table 4-5.

Table 4-5: Measured compressive strength

Compressive strength (f'c) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	30.23	34.59	37.59	40.11
Concrete set 2	25.09	28.29	35.15	41.00
Average	27.66	31.44	36.37	40.56
[(set 1- set 2)/average]×100	9.3%	10.0%	3.3%	1.1%

DG 2002 suggests a ratio of 1.35 for the long-term compressive strength of concrete (20y/28d).

Table 4-5 shows that there is a maximum of 10% difference in the compressive strength of concrete, as compared with the average, between two concrete sets at an early age (14 days) and the difference decreased to 1% at 90 days. Since DG 2002 does not provide a strength gain equation for compressive strength, the Euro-International Concrete Committee (CEB-FIP - Comité Euro-Federation Internationale de la Precontrainte du Béton) Model Code 90 (MC-90 1993) was used to compare the strength gain of concrete used in this study.

CEB-FIP MC-90 uses Equations 4-1 and 4-2 to estimate the compressive strength of concrete at any age, based on the age value of 28 days.

$$f_{cm}(t) = \beta_{cc}(t) \times f_{cm} \qquad \text{Equation 4-1}$$

$$\beta_{cc}(t) = \exp\left\{ s \times \left[1 - \left(\frac{28}{t/t_1}\right)^{1/2} \right] \right\} \qquad \text{Equation 4-2}$$

Where:

$f_{cm}(t)$ = mean compressive strength at the age of "t" days (MPa)

f_{cm} = mean compressive strength after 28 days (f'c) (MPa)

$\beta_{cc}(t)$ = coefficient which depends on the age of the concrete, "t"

t = the age of the concrete (days)

t_1 = 1 day

s = coefficient, which depends on the type of cement; 0.25 for normal and rapid hardening cements; 0.38 for slowly hardening cements.

Figure 4-3 shows the CEB-FIP MC-90 compressive strength gain and measured values for the AHD project.

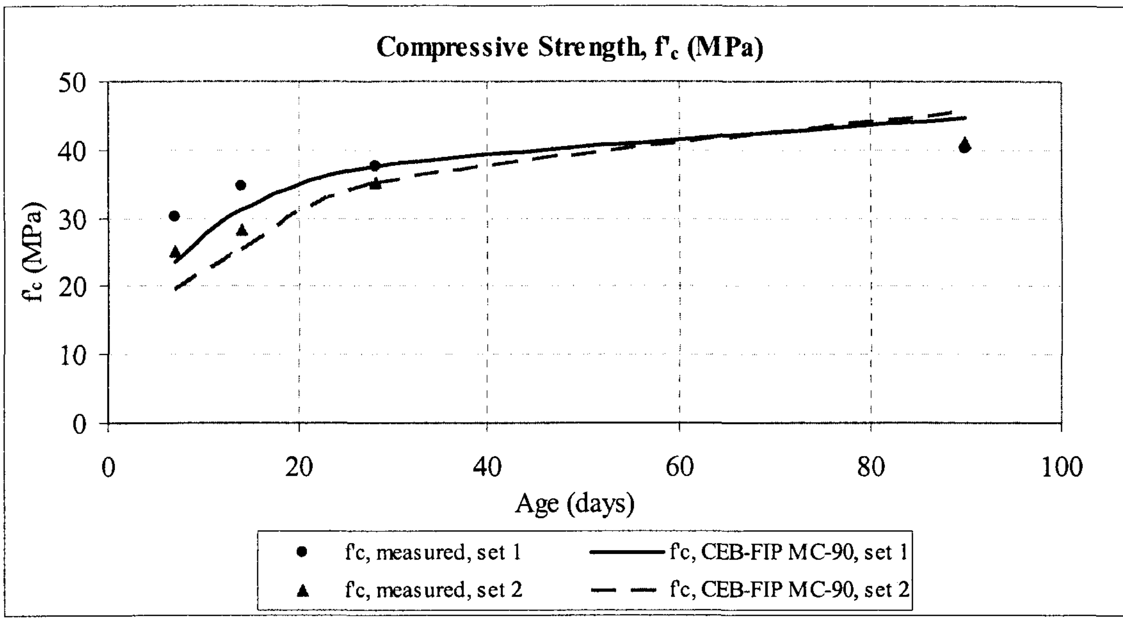

Figure 4-3: Measured compressive strength and estimated by CEB-FIP MC-90

As can be seen, the strength gain for the AHD concrete project was above that for the CEB-FIP MC-90 model at the early ages; nevertheless, it was below this model at a higher age of concrete (90 days).

4.6.3 Modulus of elasticity of PCC (E_c)

The modulus of elasticity represents the ratio of stress to strain in the elastic range of a stress-strain curve for a specific concrete. It is influenced by the ratio of water to cementitious materials, the relative proportions of paste and aggregate, and the type of the aggregate. The PCC elastic modulus has a strong effect on pavement deflection and the stress throughout the pavement structure.

Design Level 1

Design level 1 for the PCC modulus of elasticity requires the following:

- PCC modulus of elasticity must be determined directly by laboratory testing. The chord modulus is obtained based on ASTM C469 at ages of 7, 14, 28, and 90 days.

- The long-term elastic modulus ratio (20 year/28 day) should be estimated.
- The modulus gain curve should be estimated using the data and long-term modulus ratio, which allows the prediction of E_c at any time over the design life.

In order to determine the modulus of elasticity by laboratory testing, concrete cylinders (100×200 mm) were tested using a metallic frame and Linear Variable Displacement Transducers (LVDT). The LVDTs measured the longitudinal and radial displacements and the compression load was measured by a load cell, as can be seen in Figure 4-4.

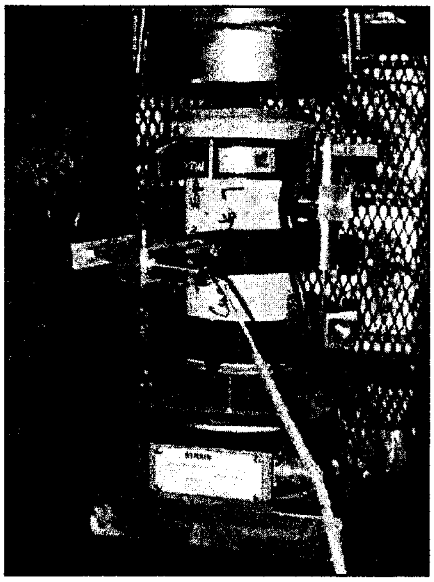

Figure 4-4: Modulus of elasticity loading system

Three calibration loads and three test loads were applied to each concrete sample. The calibration and test loads were equal to 20% and 40% of the maximum loads from compression strength, at different ages (7, 14, 28, and 90 days). One typical displacement testing result and one typical loading chart record are presented in Appendix A-2.

The modulus of elasticity was determined using Equation 4-3:

$$E_c = \frac{S_2 - S_1}{\varepsilon_2 - 0.00005} \qquad \text{Equation 4-3}$$

Where:

E_c = modulus of elasticity (MPa)

S_2 = compressive stress corresponding to 40% of the maximum load from the compression strength test (kN)

S_1 = compressive stress corresponding to 0.00005 mm longitudinal strain

ε_2 = longitudinal strain corresponding to S_2.

At each concrete age (7, 14, 28, and 90 days), three samples were tested and each one's modulus of elasticity was determined. The averages of these three testing results were used as a modulus of elasticity of concrete at that age (Appendix A-3). A summary of the measured modulus of elasticity for the concrete from AHD project in Edmonton is presented in Table 4-6.

Table 4-6: Measured modulus of elasticity

Modulus of elasticity (E_c) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	-	23,528	24,951	27,538
Concrete set 2	22,349	23,322	24,668	28,595
Average	22,349	23,425	24,809	28,067
[(set 1- set 2)/average]×100	-	0.4%	0.6%	1.9%

Modulus of elasticity measurements for both concrete sets, showed very consistent results, from which we can conclude that they are likely from the same concrete mix design. Level 1 of DG 2002 requires that the elastic modulus be measured. The ratio of 20 years to 28 days (20y/28d) could be considered equal to a ratio of 1.2.

In the DG 2002, it has been suggested that Equation 4-4 be calibrated for the prediction of modulus of elasticity at any ages for the 28-day modulus of elasticity of concrete.

$$MODRATIO = \alpha_1+\alpha_2\times\log(Age)+\alpha_3\times[\log(Age)]^2 \qquad \text{Equation 4-4}$$

Where:

MODRATIO = ratio of E_c at a given age to E_c at 28 days

Age = specimen age (years)

α_1, α_2, α_3 = regression coefficients.

Based on testing results from the AHD project, the regression coefficients from Equation 4-4 were determined and are presented in Table 4-7.

Table 4-7: Regression coefficients for modulus of elasticity

Regression coefficients	Concrete set 1	Concrete set 2	Average
α_1	1.2406	1.4379	1.3393
α_2	0.2365	0.5414	0.3890
α_3	0.0186	0.1353	0.0770

Design Level 2

In Design Level 2, the modulus of elasticity of the PCC is estimated based on compressive strength testing results and the unit weight of PCC, using the Equation 4-5.

$$E_c = 33 \times \rho^{3/2} \times (f'c)^{1/2} \quad \text{Equation 4-5}$$

Where:

E_c = PCC elastic modulus (psi)

ρ = unit weight of concrete (lb/ft^3)

$f'c$ = compressive strength of PCC (psi).

The estimated modulus of elasticity for Design Level 2 is presented in Table 4-8 and Figure 4-5.

Table 4-8: Estimation of modulus of elasticity at level 2

Estimated modulus of elasticity (E_c) MPa	Age (days)			
	7	14	28	90
Concrete set 1	25,632	27,417	28,579	29,523
Concrete set 2	23,093	24,521	27,333	29,520
Average, estimated	24,362	25,969	27,956	29,521
Average, measured	22,349	23,425	24,809	28,067
[(set 1- set 2)/average]×100	9.0%	10.9%	12.7%	5.2%

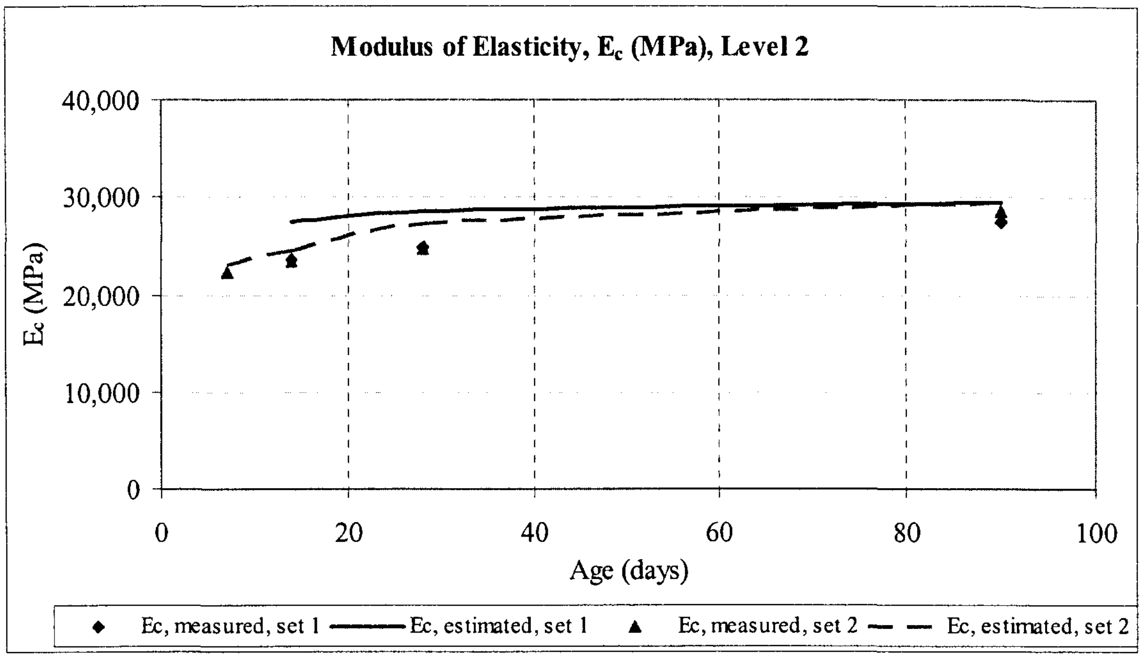

Figure 4-5: Estimation of modulus of elasticity at level 2

The modulus of elasticity predicted from Equation 4-5 gave higher values than the measured modulus of elasticity at all concrete ages. The difference is up to 13% at a 28-day concrete age.

Design Level 3

In Design Level 3, the elastic modulus is obtained from a single point, 28-day modulus of rupture (M_R) or compressive strength (f'_c). Equation 4-6 is used to determine the ratio of modulus of rupture at ages 7, 14, and 90, to a modulus of rupture of concrete at age 28 days.

$$F\_STRRATIO\_3 = 1.0+0.12\times\log(Age/0.0767)-0.01566\times[\log(Age/0.0767)]^2$$

Equation 4-6

Where:

F_STRRATIO = ratio of M_R at a given age to M_R at 28 days

Age = specimen age (years).

Furthermore, based on Equation 4-7, the compressive strength can be calculated from the modulus of rupture, with the result being that Equation 4-5 could be used to estimate the elastic modulus.

$$M_R = 9.5\times(f'_c)^{1/2} \qquad \text{Equation 4-7}$$

Where:

M_R = modulus of rupture or flexural strength (psi)

f'_c = compressive strength (psi).

The results of the estimated elastic modulus for Design Level 3, using as an initial value the 28-day modulus of rupture, are presented in Tables 4-9 and Figure 4-6.

Table 4-9: Estimation of the modulus of elasticity at Level 3 from the modulus of rupture

Estimated modulus of elasticity (E_c) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	25,285	26,392	27,421	28,979
Concrete set 2	20,101	20,981	21,800	23,039
Average, estimated	22,693	23,687	24,611	26,009
Average, measured	22,349	23,425	24,809	28,067
[(set 1- set 2)/average]×100	1.5%	1.1%	0.8%	7.3%

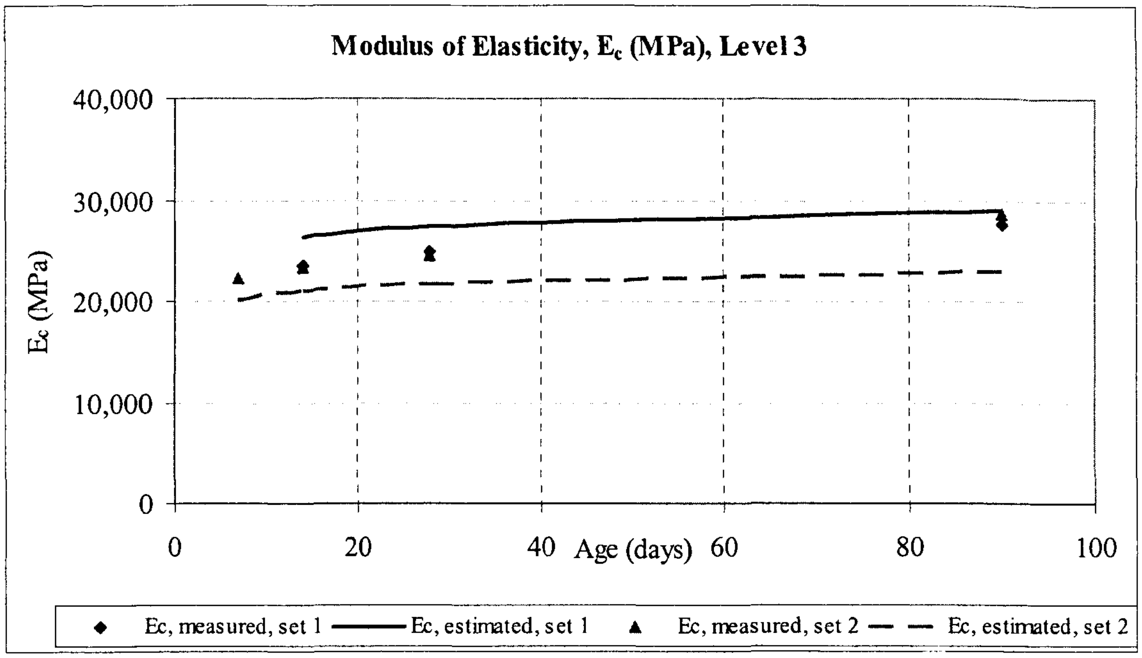

Figure 4-6: Estimation of modulus of elasticity at Level 3 from modulus of rupture

The estimated modulus of elasticity found using Equations 4-6, 4-7, and 4-5 gave very consistent results with the measured modulus of elasticity. The highest difference is around 7% at the 90-day concrete age. In another approach, Equation 4-7 can determine the estimated 28-day modulus of rupture of the concrete. The modulus of rupture ratios previously calculated are used to estimate the modulus of rupture at any age of the concrete. Equation 4-7 is implied again to transform the modulus of rupture into compressive strength at any age of the concrete. An estimated compressive strength is used in Equation 4-5 to predict the elastic modulus at any age of the concrete. The results of the estimated elastic modulus for Design Level 3, using as an initial value the 28-day f'_c, are presented in Tables 4-10 and Figure 4-7.

Table 4-10: Estimation of the modulus of elasticity at Level 3 from compressive strength

Estimated modulus of elasticity (E_c) MPa	**Age (Days)**			
	7	**14**	**28**	**90**
Concrete set 1	26,352	27,506	28,579	30,203
Concrete set 2	25,203	26,307	27,333	28,886
Average, estimated	25,778	26,907	27,956	29,545
Average, measured	22,349	23,425	24,809	28,067
[(set 1- set 2)/average]×100	15.3%	14.9%	12.7%	5.3%

This method for the prediction of the modulus of elasticity resulted in differences between 5% and 15% higher than the measured values. The difference has been decreased with increasing the age of the concrete.

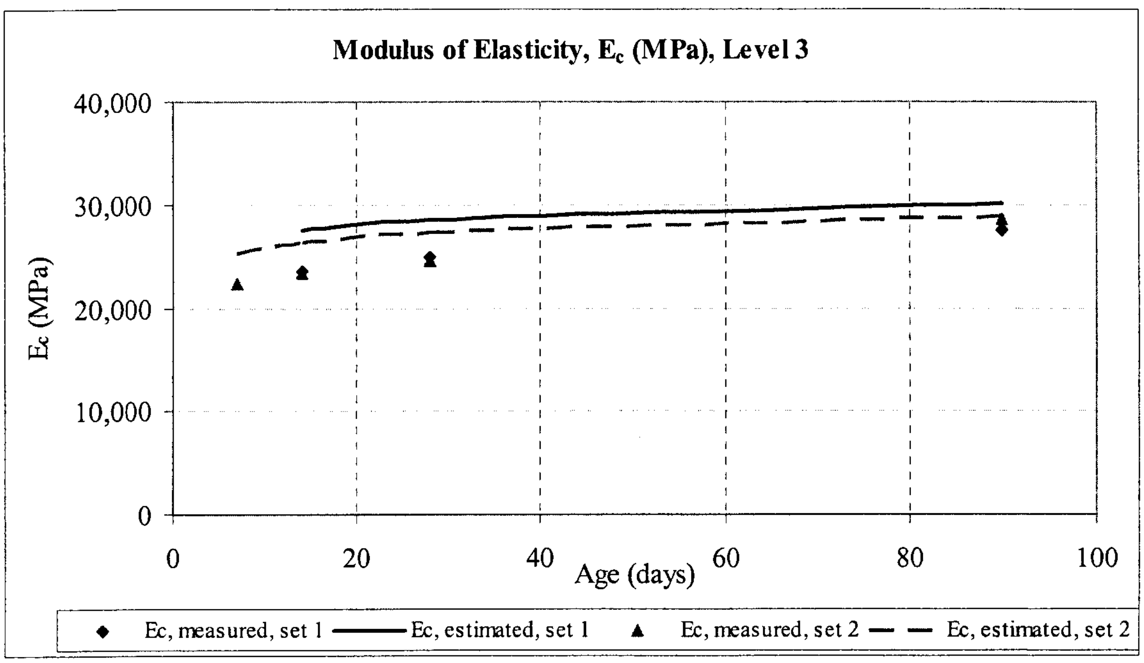

Figure 4-7: Estimation of the modulus of elasticity at Level 3 from compressive strength

It can be concluded that, at level 3, an estimation of the elastic modulus from the modulus of rupture was more accurate than using the compressive strength of concrete.

Prediction models for the elastic modulus in DG 2002 are, to some extent, new and many engineers are using other models for the prediction of the modulus of elasticity. It was decided to test the accuracy of CEB-FIP MC-90 model for the prediction of the modulus of elasticity. Using the CEB-FIP MC-90, Equations 4-8 and 4-9, and the 28-day elastic modulus, the elastic modulus can be predicted at any age of the concrete.

$E_{ci}(t) = \beta_E(t) \times E_{ci}$ Equation 4-8

$\beta_E(t) = [\beta_{cc}(t)]^{0.5}$ Equation 4-9

Where:

$E_{ci}(t)$ = modulus of elasticity at an age of "t" days (MPa)

E_{ci} = modulus of elasticity at an age of 28 days = $2.15 \times 10^4 \times (f_{cm}/10)^{1/3}$ (MPa)

$\beta_E(t)$ = coefficient which depends on the age of the concrete, "t"

$\beta_{cc}(t)$ = coefficient which depends on the age of the concrete, "t" (from Equation 4-2).

Figure 4-8 presents measured and estimated elastic modulus using CEB-FIP MC-90 modulus prediction model.

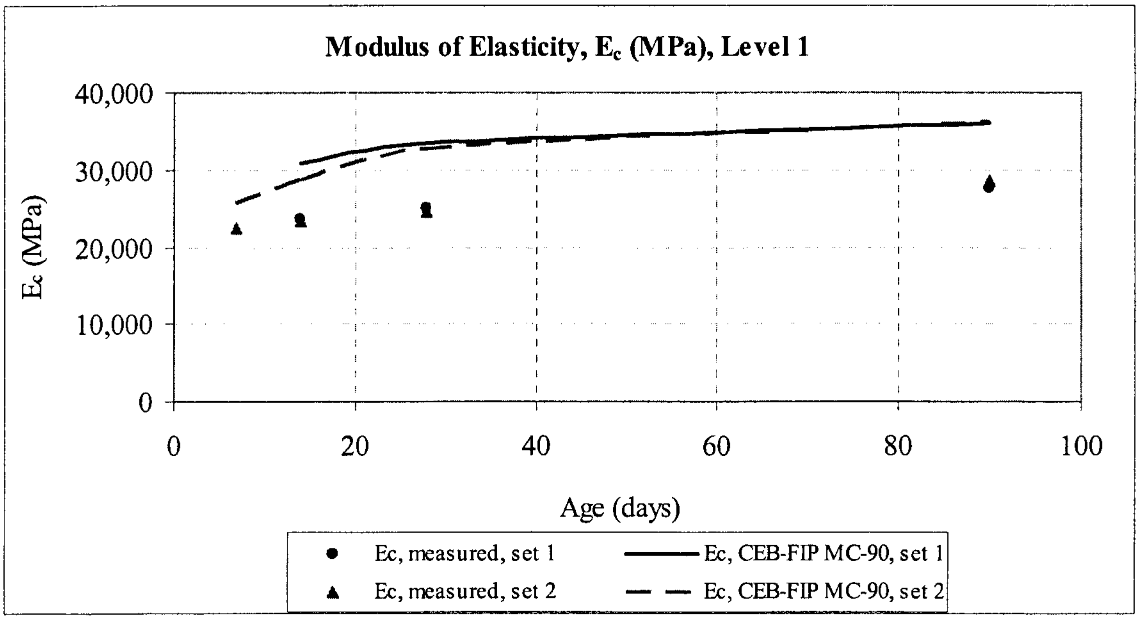

Figure 4-8: Estimation of modulus of elasticity by CEB-FIP MC-90

As can be seen, the measured modulus of elasticity from the AHD concrete project is 10-20% lower than the CEM-FIP MC-90 model prediction using the 28-day compressive strength of concrete. This is in agreement with the elastic modulus testing results from two HPCs in Alberta (Soleymani 2005 and William 2005).

4.6.4 Poisson's ratio of PCC (η)

Design Level 1

Poisson's ratio is determined simultaneously with the determination of the elastic modulus, according to ASTM C469. It was determined using Equation 4-10:

$$\eta = \frac{\varepsilon_{t2} - \varepsilon_{t1}}{\varepsilon_2 - 0.00005} \qquad \text{Equation 4-10}$$

Where:

η = Poisson's ratio

ε_{t2} = radial strain corresponding to S_2 stress (from Equation 4-3)

ε_{t1} = radial strain corresponding to 0.00005 mm longitudinal strain

ε_2 = longitudinal strain corresponding to S_2.

Details of Poisson's ratio calculations for AHD concrete are given in Appendix A-2. For each sample having three loading tests, three values of Poisson's ratio were determined and were then averaged to represent the result for the AHD concrete sample. The Poisson's ratio test results are presented in Table 4-11.

Table 4-11: Measured Poisson's ratio

Poisson's ratio	**Age (Days)**			
η	**7**	**14**	**28**	**90**
Concrete set 1	0.19	0.15	0.16	0.17
Concrete set 2	0.15	0.17	0.17	0.20
Average	0.17	0.16	0.17	0.19
[(set 1- set 2)/average]×100	11.8%	6.3%	3.0%	8.1%

Table 4-11 shows that there was a maximum difference of 12% between the two concrete sets, at a 7-day concrete age, in Poisson's ratio of concrete, compared to the average measurements. Design Level 2 is not applicable since there are no correlations or relationships that may be used to estimate Poisson's ratio from other material characteristics or other tests.

Design Level 3

Level 3 of DG 2002 suggests a typical range of 0.15 to 0.25 for Poisson's ratio. Poisson's ratio measured values from the AHD project are in the range of values suggested by DG 2002.

Measured Poisson's ratio using this concrete yields similar results to Poisson's ratio determined for a HPC project in Edmonton, which has been reported as 0.18 (Soleymani 2006).

4.6.5 Modulus of rupture or flexural strength of PCC (M_R)

The modulus of rupture (M_R) is defined as the maximum tensile stress at the rupture at the bottom of a simply supported concrete beam during a flexural test with third point loading (see Figure 4-9). M_R was determined for the AHD concrete project by testing according to ASTM C78.

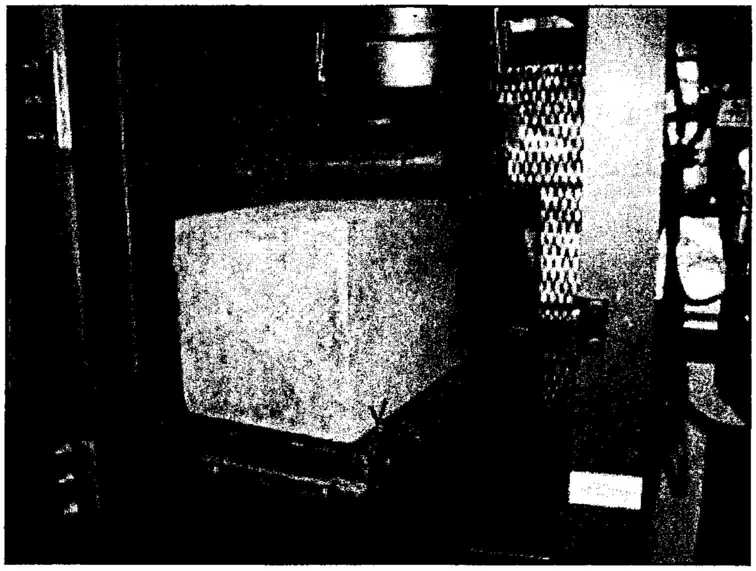

Figure 4-9: Modulus of rupture loading system

Design Level 1

Design level 1 requires measuring modulus of rupture for concrete based on ASTM C78. Table 4-12 and Figure 4-10 present testing results for the modulus of rupture for the AHD project.

Table 4-12: Measured modulus of rupture

Modulus of rupture (M_R) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	3.67	4.14	4.64	-
Concrete set 2	3.13	3.49	3.73	4.76
Average	3.40	3.81	4.18	4.76
[(set 1- set 2)/average]×100	8.0%	8.5%	10.8%	0.0%

Testing results for the modulus of rupture for the two sets of concrete samples showed a maximum 11% difference compared to the average results measured. Level 1 of the DG 2002 suggests a long-term ratio, 20y/28d, for a modulus of rupture, equal to 1.2.

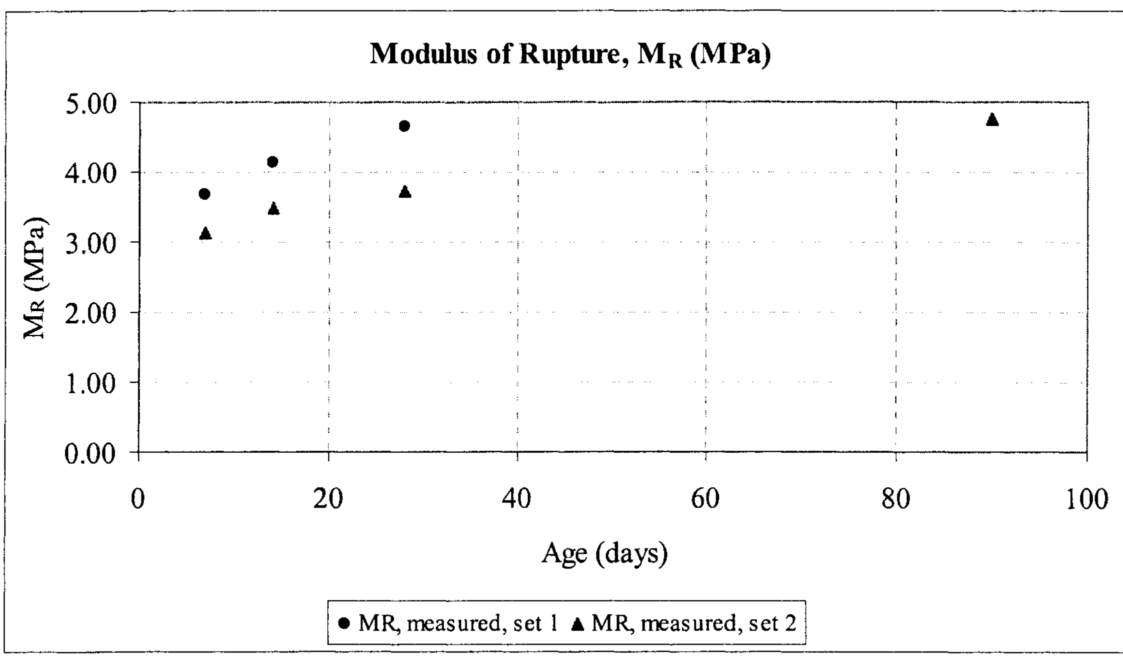

Figure 4-10: Measured modulus of rupture at level 1

DG 2002 suggests Equation 4-11 for time dependency of the modulus of rupture.

$$F\_STRRATIO = \alpha_1 + \alpha_2 \times \log(Age) + \alpha_3 \times [\log(Age)]^2 \qquad \text{Equation 4-11}$$

Where:

F_STRRATIO = ratio of M_R at a given age to M_R at 28 days

Age = specimen age (years)

α_1, α_2, α_3 = regression constants.

The regression coefficients were found and are presented in Table 4-13.

Table 4-13: Regression coefficients for modulus of rupture

Regression coefficients	**Concrete set 1**	**Concrete set 2**	**Average**
α_1	1.4705	1.7119	1.5912
α_2	0.4710	0.8421	0.6566
α_3	0.0440	0.1967	0.1204

Design Level 2

Design Level 2 for modulus of rupture is based on compressive strength testing results (Table 4-5) and Equation 4-7, which transforms the compressive strength to the modulus of rupture. The results of this prediction model are shown in Table 4-14 and Figure 4-11.

Table 4-14: Estimated modulus of rupture at Level 2

Estimated modulus of rupture (M_R) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	4.34	4.64	4.84	5.00
Concrete set 2	3.95	4.20	4.68	5.05
Average, estimated	4.14	4.42	4.76	5.02
Average, measured	3.40	3.81	4.18	4.76
[(set 1- set 2)/average]×100	22.0%	15.9%	13.7%	5.5%

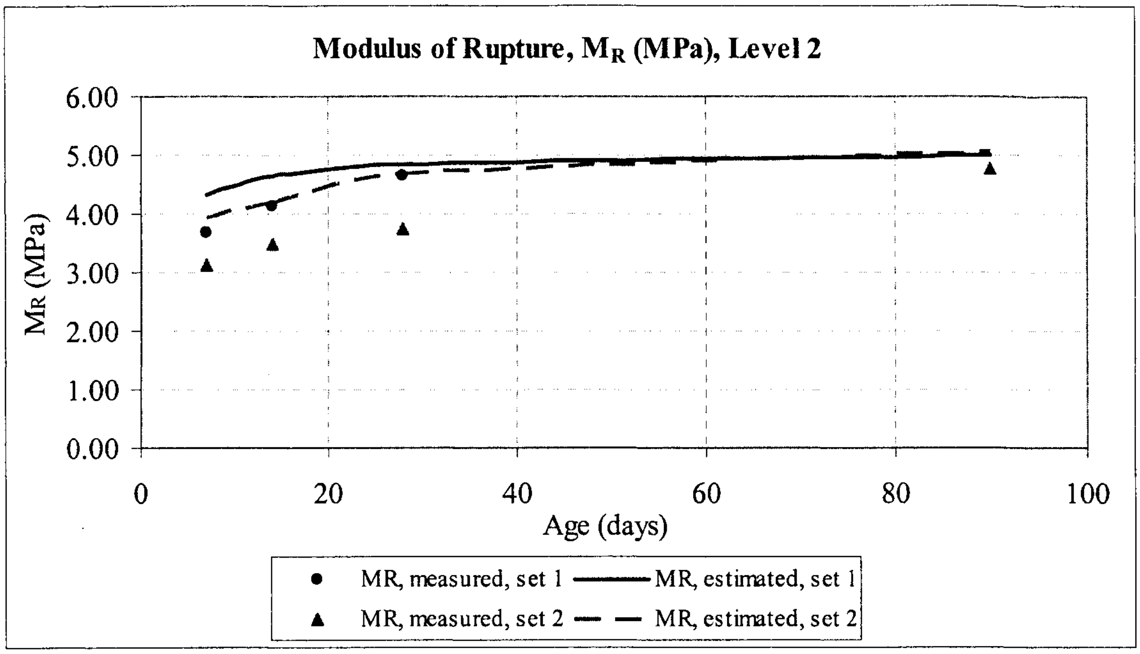

Figure 4-11: Estimated modulus of rupture at Level 2

In general, the suggested model in the DG 2002 Level 2 for modulus of rupture overestimated this parameter as compared to the values measured for the AHD concrete project. The difference is up to 22% at an early age of concrete (7-day); however, this difference drops to 6% for 90-day concrete.

Design Level 3

Design Level 3 for the modulus of rupture is based on a single point modulus of rupture or on compressive strength at the age of 28 days. Equation 4-6 is used to estimate the modulus of rupture at different ages. The results of the estimated modulus of rupture for Design Level 3, using as an initial value the 28-day M_R, are presented in Table 4-15 and Figure 4-12.

Table 4-15: Estimated modulus of rupture at Level 3 from the modulus of rupture

Estimated modulus of rupture (M_R) MPa	Age (days)			
	7	14	28	90
Concrete set 1	4.28	4.46	4.64	4.90
Concrete set 2	3.44	3.59	3.73	3.94
Average, estimated	3.86	4.03	4.18	4.42
Average, measured	3.40	3.81	4.18	4.76
[(set 1- set 2)/average]×100	13.5%	5.6%	-	7.1%

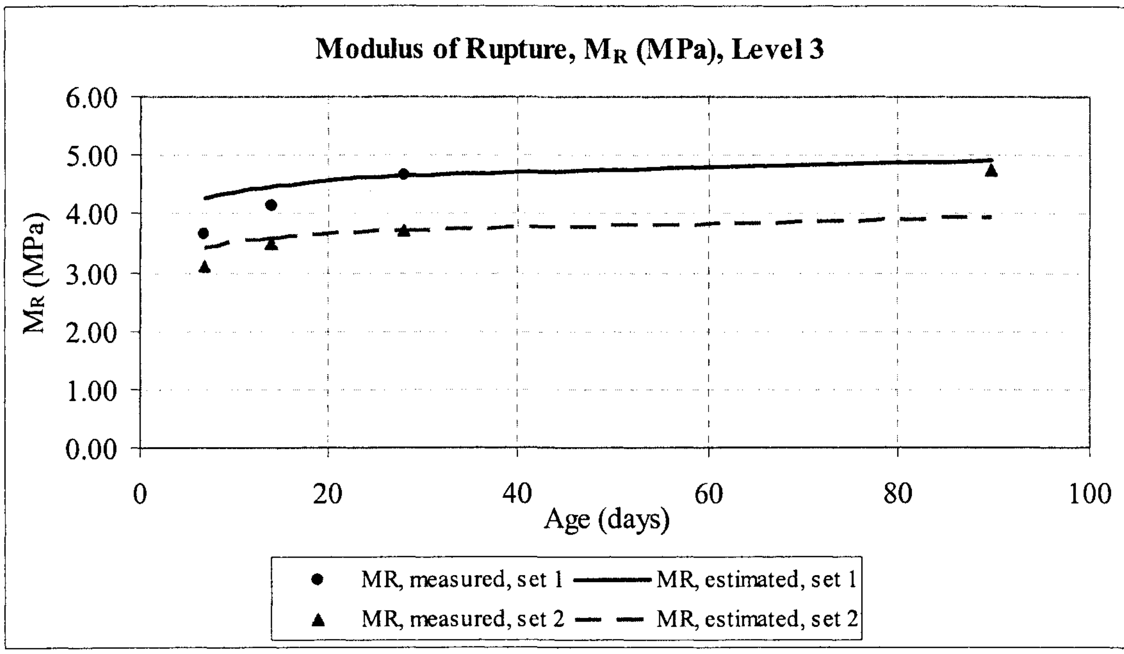

Figure 4-12: Estimated modulus of rupture at Level 3 from the modulus of rupture

The results showed a maximum 13% difference in estimating the modulus of rupture when compared to measured values. The estimated values of modulus of rupture are higher for concrete ages less than 28 days and lower later concrete ages (more than 28 days). The second option for estimating the modulus of rupture in Design Level 3 is to use a single point value of compressive strength at the age of 28-days and using Equations 4-7 and 4-6. The results of the estimated modulus of rupture for Design Level 3, using as an initial value the 28-day compressive strength, are presented in Table 4-16 and Figure 4-13.

Table 4-16: Estimated modulus of rupture at Level 3 from compressive strength

Estimated modulus of rupture, M_R, MPa	Age (days)			
	7	14	28	90
Concrete set 1	4.46	4.65	4.83	5.11
Concrete set 2	4.31	4.50	4.68	4.94
Average, estimated	4.39	4.58	4.76	5.03
Average, measured	3.40	3.81	4.18	4.76
[(set 1- set 2)/average]×100	29.1%	20.1%	13.7%	5.6%

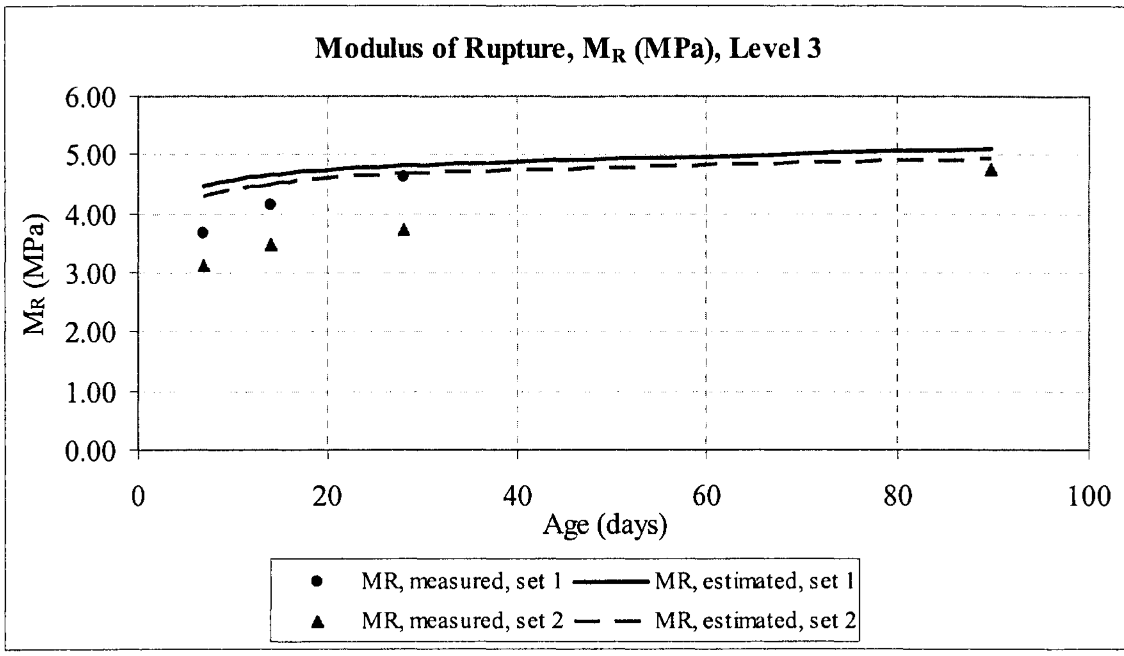

Figure 4-13: Estimated modulus of rupture at Level 3 from compressive strength

Using the 28-day compressive strength as well as Equations 4-7 and 4-6, the estimated modulus of rupture was higher than the measured values at all concrete ages. The maximum difference was up to 29% at the early age of concrete and dropped to 6% at a 90-day age. Comparing the methods in Level 3 for estimating the modulus of rupture, the method based on the 28-day modulus of rupture gives closer results to the measured values.

4.6.6 Indirect tensile strength of PCC (f_t)

The indirect tensile strength is determined by applying loads along the height of a concrete cylinder, as shown in Figure 4-14. The indirect tensile strength test must be performed according to ASTM C496.

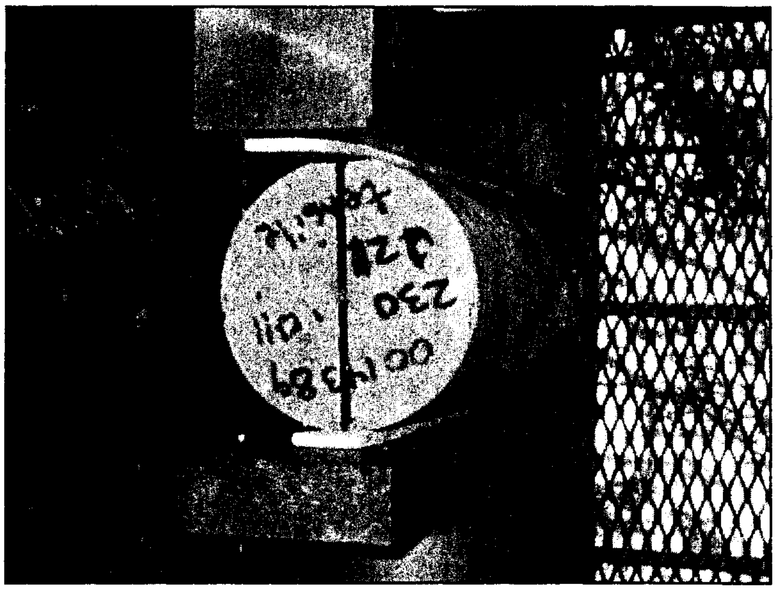

Figure 4-14: Indirect tensile strength loading system

Design Level 1

Design Level 1 for indirect tensile strength requires testing this property of concrete. Testing results for indirect tensile strength from the AHD concrete project are presented in Table 4-17 and Figure 4-15.

Table 4-17: Measured indirect tensile strength at Level 1

Indirect tensile strength (f_t) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	3.04	3.51	3.56	3.73
Concrete set 2	2.52	2.72	3.61	4.18
Average	2.78	3.12	3.58	3.96
[(set 1- set 2)/average]×100	9.4%	12.7%	0.7%	5.7%

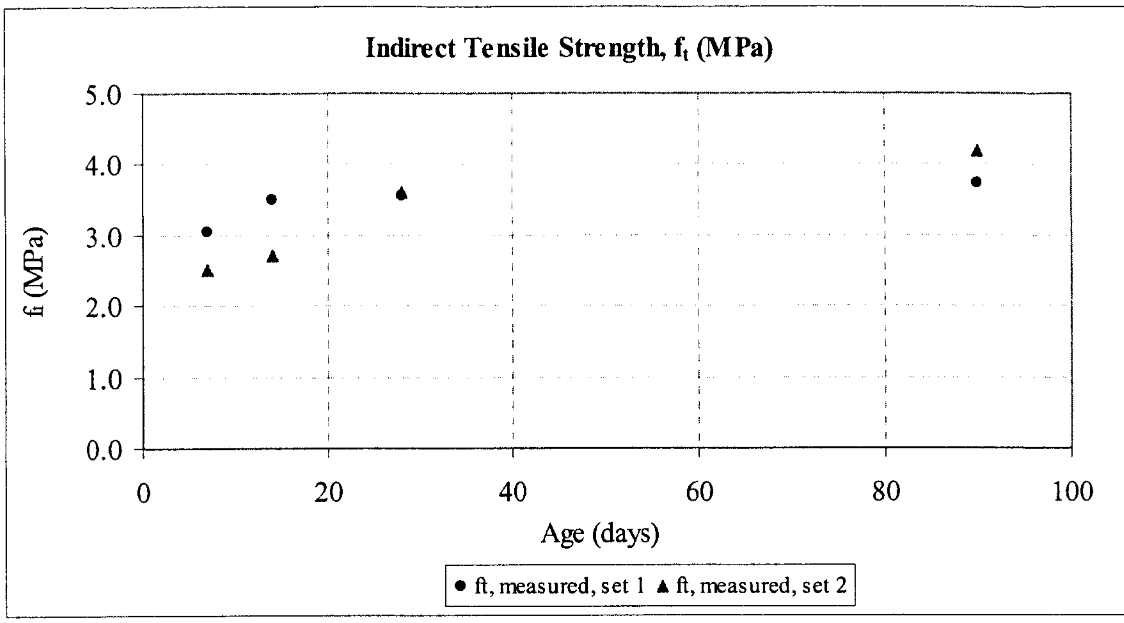

Figure 4-15: Measured indirect tensile strength at Level 1

Table 4-17 shows that there was a maximum of 13% difference in indirect tensile strength measurements, between the two concrete sets, compared to average. The DG 2002 recommends a long-term ratio, 20y/28d, equal to 1.20 for indirect tensile strength. The development of the indirect tensile strength gain curve for indirect tensile strength is based on the ratios of measured indirect tensile strength at different ages to the measured indirect tensile strength at the age of 28-days (Equation 4-12).

$$T\_STRRATIO = \alpha_1 + \alpha_2 \times \log(Age) + \alpha_3 \times [\log(Age)]^2 \qquad \text{Equation 4-12}$$

Where:

T_STRRATIO = ratio of f_t at a given age to f_t at 28 days

Age = specimen age (years)

α_1, α_2, α_3 = regression constants.

Equation 4-12 was used to find the regression constants. The results are presented in Table 4-18.

Table 4-18: Regression coefficients for indirect tensile strength

Regression coefficients	Concrete set 1	Concrete set 2	Average
α_1	0.9383	1.4104	1.1744
α_2	-0.2919	0.3829	0.0455
α_3	-0.1951	-0.0263	-0.1107

Design Level 2

In Design Level 2, the indirect tensile strength is determined from test data at different ages for compressive strength (f'_c), which are converted to moduli of rupture using Equation 4-7. The results of these estimates and measured values for indirect tensile strength are shown in Table 4-19 and Figure 4-16.

Table 4-19: Estimated indirect tensile strength at Level 2

Estimated indirect tensile strength (f_t) MPa	Age (days)			
	7	14	28	90
Concrete set 1	2.91	3.11	3.24	3.35
Concrete set 2	2.65	2.81	3.13	3.38
Average, estimated	2.78	2.96	3.19	3.37
Average, measured	2.78	3.12	3.58	3.96
[(set 1- set 2)/average]×100	0.1%	5.0%	11.1%	14.9%

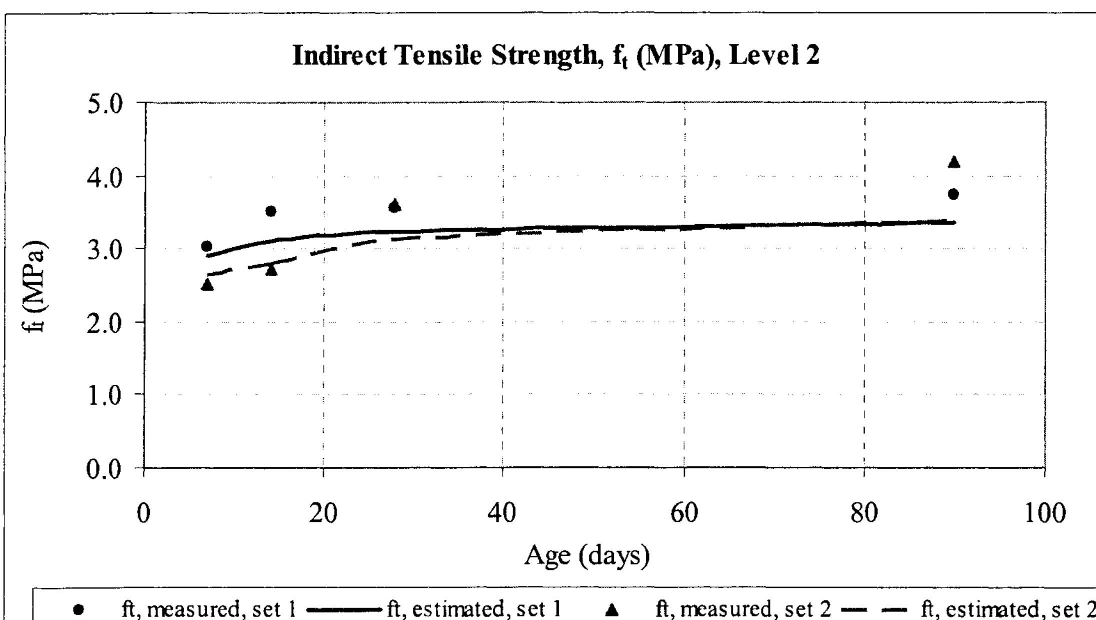

Figure 4-16: Estimated indirect tensile strength at Level 2

The estimated indirect tensile strength at Level 2 gave lower values than the measured values at all concrete ages. The difference was up to 15% at the 90-day concrete age.

Design Level 3

Design Level 3 requires a single-point modulus of rupture at the age of 28-days and uses Equation 4-13 for determining the indirect tensile strength.

$$T\_STRRATIO\_3 = 0.67 \times \{1.0 + 0.12 \times \log(Age/0.0767) - 0.01566 \times [\log(Age/0.0767)]^2 \}$$

Equation 4-13

Where:

T_STRRATIO_3 = ratio of f_t at a given age to f_t at 28 days

Age = specimen age (years).

The results of the estimated indirect tensile strength for Design Level 3, using as the initial value of M_R at the 28-day, and the measured values are presented in Table 4-20 and Figure 4-17, respectively.

Table 4-20: Estimated indirect tensile strength at Level 3 from modulus of rupture

Estimated indirect tensile strength (f_t) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	2.86	2.99	3.11	3.28
Concrete set 2	2.30	2.41	2.50	2.64
Average, estimated	2.58	2.70	2.80	2.96
Average, measured	2.78	3.12	3.58	3.96
[(set 1- set 2)/average]×100	7.0%	13.4%	21.8%	25.1%

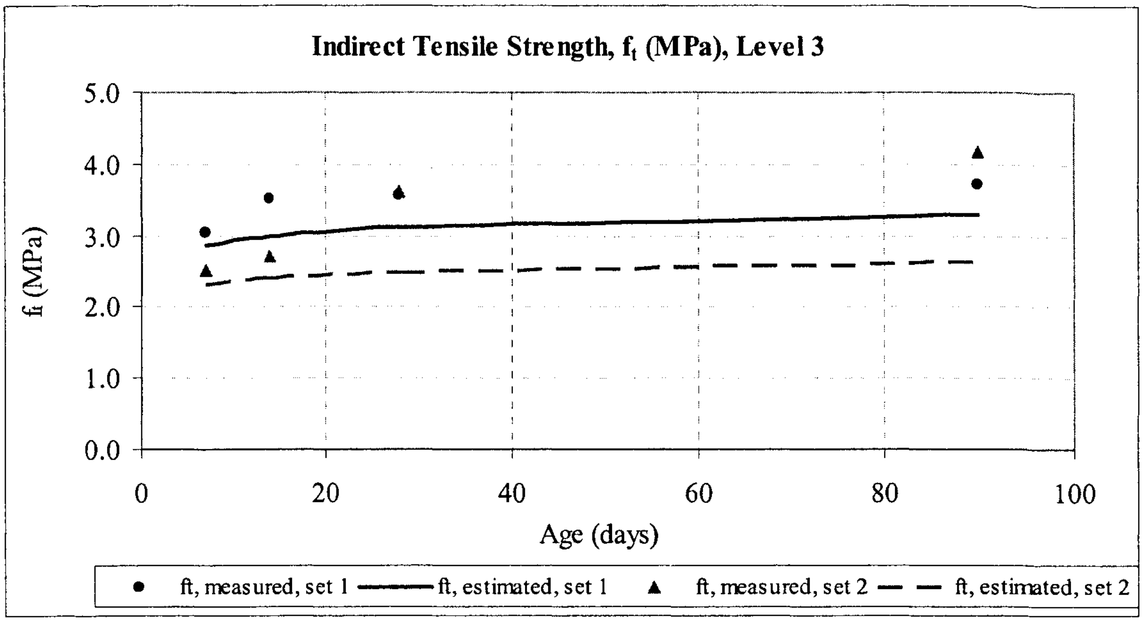

Figure 4-17: Estimated indirect tensile strength at Level 3 from the modulus of rupture

The estimated indirect tensile strength at Level 3, from the 28-day modulus of rupture, gave lower values than the measured values at all concrete ages. The difference was up to 25% at the 90-day concrete age. The second option for estimating indirect tensile strength at Level 3 uses a single point value of compression strength at the age of 28 days and Equations 4-7 and 4-13. The results of the estimated indirect tensile strength for Design Level 3, using 28-day f'_c as an initial value, are presented in Table 4-21 and Figure 4-18.

Table 4-21: Estimated indirect tensile strength at Level 3 from compressive strength

Estimated indirect tensile strength (f_t) MPa	**Age (days)**			
	7	**14**	**28**	**90**
Concrete set 1	2.99	3.12	3.24	3.42
Concrete set 2	2.89	3.02	3.13	3.31
Average, estimated	2.94	3.07	3.19	3.37
Average, measured	2.78	3.12	3.58	3.96
[(set 1- set 2)/average]×100	5.7%	1.6%	11.1%	14.8%

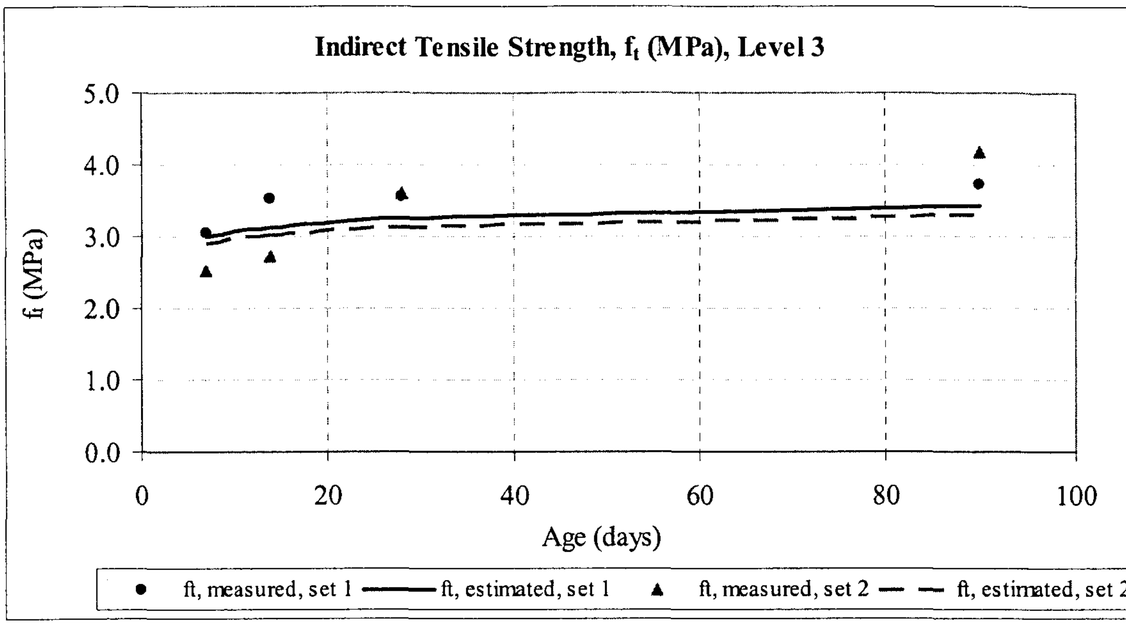

Figure 4-18: Estimated indirect tensile strength at Level 3 from compressive strength

The estimated indirect tensile strength at Level 3, from 28-day compressive strength, gave lower values than the measured values at all concrete ages except for the 7-day when the estimated value is higher than the measured one. For concrete ages greater than 7 days, the difference is up to 15% at 90-day concrete age. Comparing the two approaches for estimating the indirect tensile strength at Level 3, the proposed model using the 28-day compressive strength gives closer results to measured values.

4.6.7 Coefficient of thermal expansion of PCC (α)

The Coefficient of Thermal Expansion (CTE) is defined as the change in unit length per degree of temperature change. It is used to determine the unrestrained change in length produced by a given change in temperature, according to Equation 4-14. The DG 2002 is the first rigid pavement model that uses the CTE for pavement performance prediction.

$$\Delta L = \alpha \times \Delta T \times L \qquad \text{Equation 4-14}$$

Where:

ΔL = change in unit length of PCC due to a temperature change ΔT

α = coefficient of thermal expansion of PCC (10^{-6}/°C)

ΔT = temperature change (°C)

L = length of specimen.

The CTE of PCC was determined on concrete samples collected from the Anthony Henday Drive project in Edmonton, and cured in a moist room for 28 days. The test was performed according to AASHTO TP-60 standard. To measure the CTE of PCC, a stainless steel cylinder, similar to the size of a concrete cylinder was used as a control sample. A metallic frame was prepared and a stainless steel cylinder was used for its calibration (Figure 4-19 a). The metallic frame was used for testing the concrete samples (Figure 4-19 b). A constant temperature bath was used to keep the metallic frame and the sample at a constant temperature. In order to find the calibration factor of the metallic frame (the CTE of the metallic frame), the total CTE of the metallic frame and stainless steel cylinder was determined. Knowing the CTE of stainless steel cylinder, the CTE of the metallic frame was found. For the concrete CTE test, two concrete samples were tested for each of the two sets of concrete. The average of each two samples was used as the CTE.

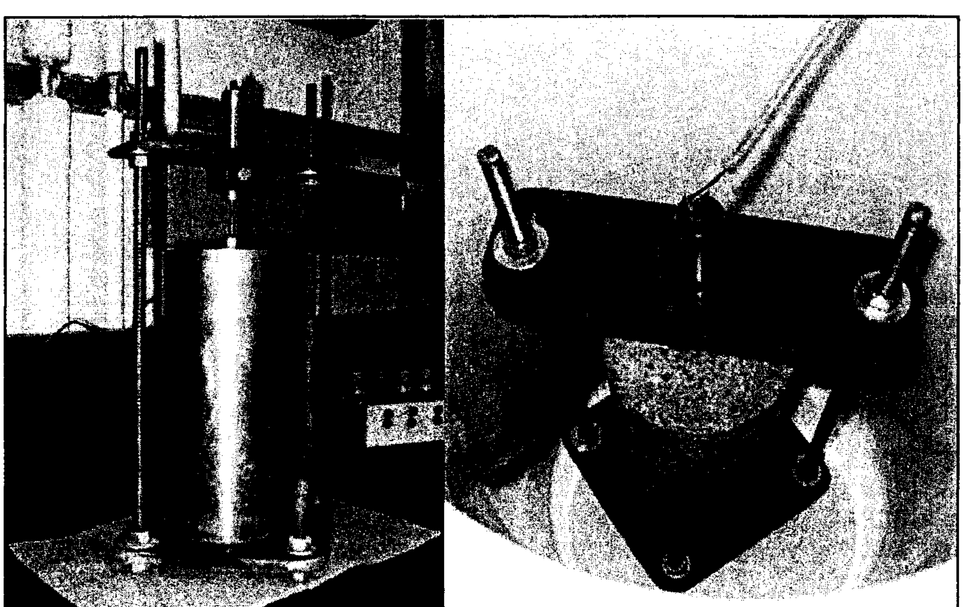

Figure 4-19: a) Stainless steel and b) Concrete sample in coefficient of thermal expansion test system

The test procedure consisted of setting the sample (stainless steel or concrete) in the metallic frame, installing the LVDT on top of the sample, placing the metallic frame and the sample in the thermal bath, and adding hot water to the tank. The LVDT length changes were measured continuously. When the length change stabilized, the water bath temperature was recorded. The hot water was then replaced with cold water and the length change and the corresponding water bath temperature were recorded. Later, the cold water was replaced by hot water and length change and the corresponding water bath temperature were recorded.

The three length changes of concrete specimens and temperatures were used to calculate the CTE of concrete sample. For each of the two temperature variations (hot-cold and cold-hot), one CTE was calculated and the two results were averaged to represent the CTE of one sample. The same procedure was repeated for the second concrete sample and the average results of the two samples were considered as the CTE of one set of concrete samples.

Design Level 1

Level 1 of DG 2002 requires that the CTE of concrete be measured according to AASHTO TP-60. The test results of the coefficient of thermal expansion of PCC for Design level 1 are showed in Table 4-22. Details of calculations are shown in Appendix A-4.

Table 4-22: Measured coefficient of thermal expansion

Coefficient of thermal expansion α, 10^{-6}/°C		**Age 28 days**	
Concrete set 1	sample 1	12.8	12.9
	sample 2	12.9	
Concrete set 2	sample 1	12.4	12.5
	sample 2	12.7	
Average		12.7	

Design Level 2

The Design Level 2 for the CTE of the concrete uses a linear, weighted average of the concrete constituent (aggregate and paste), CTE values, and relative volumes of the constituents (Equation 4-15):

$\alpha = \alpha_{agg} \times V_{agg} + \alpha_{paste} \times V_{paste}$ Equation 4-15

Where:

α = CTE of the concrete (10^{-6}/°C)

α_{agg} = CTE of aggregate (10^{-6}/°C)

V_{agg} = volumetric portion of the aggregate in the PCC mix (%)

α_{paste} = CTE of the cement paste (10^{-6}/°C)

V_{paste} = volumetric portion of the paste in the PCC mix (%).

Typical CTE values for concrete and its common components are presented in Table 4-23 (DG 2002, Part 2 - Chapter 2).

Table 4-23: Typical CTE ranges for common aggregates and concretes

Material Type	Coefficient of thermal expansion of aggregates, 10^{-6}/°C	Concrete coefficient of thermal expansion of concrete, 10^{-6}/°C
Aggregates		
Marbles	4.0-7.0	4.1
Limestones	3.6-6.5	6.1-9.2
Granites & gneisses	5.8-9.5	6.8-9.5
Syenites, diorites, andesite, basalt, gabbros, diabase	5.4-8.1	7.9-9.5
Dolomites	7.0-9.9	9.2-11.5
Blast furnace slag		9.2-10.6
Sandstones	10.1-12.1	10.1-11.7
Quartz sand and gravels	9.9-12.8	10.8-15.7
Quartzite, cherts	11.0-12.6	11.9-12.8
Cement Paste (saturated)		
w/c = 0.4 to 0.6	18.0-19.8	-
Concrete Cores		
Cores from LTPP pavement sections	N/A	7.2, 9.9, 13.0×10^{-6} (min-mean-max)

Based on typical volume proportions of aggregates (60-75%) and paste (25-40%) in a cement concrete mix and the typical values from Table 4-23 for a quartzite aggregate (used in the AHD concrete project), using Equation 4-15, the CTE of the concrete can be estimated in the range $12.2\text{-}14.1\times10^{-6}$/°C. The DG 2002, in Table 4-23, recommends that the CTE of the concrete made with quartzite aggregate to be considered between 11.9 and 12.8×10^{-6}/°C. The measured CTE of 12.7×10^{-6}/°C for the AHD concrete project is in the estimated and the recommended ranges mentioned.

Design Level 3

Design Level 3 suggests typical values or ranges of values for CTE of concrete when it cannot be measured. DG 2002 suggests a range between 5.4 and 14.4×10^{-6}/°C while the FHWA recommends a typical range between 7.4 and 13×10^{-6}/°C for CTE of the concrete (FHWA 2005). From DG 2002 sensitivity analysis using the AHD concrete project design inputs as average values, was found that a change in the concrete CTE from 3.6 to 18.0×10^{-6}/°C resulted in changes in final roughness (at the end of the 30-year design life) from 1.6 to 4.4 mm/m, final transverse cracking from 7.8 to 87.2%, and final faulting from 0.7 to 7.6 mm. The CTE test results for the concrete from the AHD project are within the suggested ranges for the CTE values but are closer to the upper level. Knowing that higher CTE values increase the roughness, transverse cracking, and faulting of the PCC slab, it can be concluded that it is a disadvantage for AHD concrete.

Mallela et al. (2005) conducted a study on the significance of CTE on rigid pavement design. They used cores taken from the Long Term Pavement Performance (LTPP) study to investigate the impact of CTE on slab roughness (IRI), cracking, and faulting. The following variables were chosen in this study:

- Slab thickness = 254 mm
- PCC modulus of rupture = 3.45 and 5.17 MPa
- Traffic = 1500 trucks per day (FHWA Class 4 through 13)
- Crushed stone base on AASHTO class A-4 subgrade
- Transverse joint spacing = 4.6 and 6.1 m
- Slab width = 3.7 m
- PCC CTE = 8.1, 9.9, and 12.6×10^{-6}/°C
- Lane width = 3.7 m
- Design locations (climate types) = Wet Freeze (Illinois) and Dry-No Freeze (Southern California) as per LTPP definitions.

Regarding the effect of the CTE on roughness, in general, increases in the CTE result in an increased IRI. This is due to increased cracking and joint faulting. Again, the combined effects of slab length and concrete strength show a relationship to the CTE in terms of its effect on smoothness. Longer slab lengths cause a higher IRI. For longer slabs, the effect of the concrete strength was insignificant at very high CTE values because excessive slab cracking occurs no matter what the CTE. However, at a lower CTE, concrete strength has a significant effect on the IRI.

Regarding the effect of the CTE on fatigue cracking, it was found that increases in the CTE resulted in increased slab cracking, top-down and bottom-up fatigue damage in both climates for both levels of joint spacing and concrete strength considered. Increased joint spacing also causes an increase in cracking. Shorter joint spacing in combination with increased strength makes transverse cracking practically insensitive to the CTE. For the longer slab length, cracking increased drastically, resulting in an increase in the CTE even at smaller CTE values. Cracking increases by 635% and 320% in the dry-no freeze and wet-freeze climates, respectively. Clearly, the combined effect of the longer slab length and reduced concrete strength make the transverse cracking performance of JPCP very sensitive to the CTE.

Regarding the effect of the CTE on faulting, it was found that, as expected, with an increase in the CTE and increased joint spacing, both joint opening and, consequently, faulting increased in the two climate types. The increase in faulting is higher for longer joint spacing and higher concrete strengths due to higher curling deflections for the larger joint spacing. Higher faulting values were observed in the wet-freeze climate than in the dry-no freeze climate. This increase in faulting is because of the greater amounts of moisture available for base erosion. Faulting increased by 36% and 24% in the dry-no freeze and wet-freeze climates, respectively. The combined effects of larger joint spacing and a lower modulus make the CTE more significant to mean joint faulting prediction.

The CTE of concrete was found to vary widely depending on the predominant aggregate type used in the concrete. These data were used to conduct a sensitivity analysis that showed that the CTE has a significant effect on slab cracking and, to a lesser degree, on joint faulting. Its overall effect on smoothness (IRI) was also significant. Given the fact that the CTE has not been used before in AASHTO and PCA pavement structural design methods, it was concluded that this design input is very important and must be fully considered in specifications and in the design process to reduce the risk of excessive cracking, joint faulting, and the loss of smoothness (IRI) for PCCP.

Won (2005) summarized the efforts to improve the accuracy and repeatability of testing procedures for the Coefficient of Thermal Expansion (CTE) of Continuously Reinforced Concrete Pavements (CRCP). The effects of a number of variables on the concrete CTE were investigated. The most important findings were:

- Concrete with a higher CTE is more prone to cracking, additional warping, and spalling.
- The effect of the rate of heating and cooling is negligible.
- Concrete age and specimen size also have a negligible effect.
- Coarse aggregate content in the concrete mix has an effect on the test results. Coarse aggregates from 32 sources in Texas have been evaluated using this test procedure. The results show that the coarse aggregate type has a significant effect on the CTE of concrete.
- A new testing procedure has been proposed for measuring the CTE of concrete. In this method, information on temperature and deformation is collected every minute throughout the testing. Regression analysis must be performed for temperatures and displacements falling in the range of 15°C to 45°C. This method provides more accurate and less variable test results than the AASHTO TP-60.
- The effect of the rate of heating and cooling in the range that can be achieved in most commercially available water baths is negligible.

Since the CTE of concrete is a very important parameter in rigid pavement performance, affecting all three PCCP outputs of DG 2002, it is recommended that this parameter be measured for rigid pavement design.

4.6.8 Heat capacity or specific heat of the PCC

The Heat Capacity (HC) is the amount of heat required to raise the temperature of a unit weight of a material by one degree, which is measured according to CRD-C 124-73. Testing equipment for the HC includes: constant temperature bath (calorimeter), stainless steel basket, crushed glass, and crushed concrete. The calibration process was performed using a sample of crushed glass with a known heat capacity. The calibration result was the water equivalent of the calorimeter. The concrete samples from the AHD project were tested to measure the HC of concrete.

The testing procedure was to bring the cold or hot concrete samples within the calorimeter, which contained one litre of water at room temperature. The water bath temperature was changed according to the concrete sample temperature. Time passed from the moment when the concrete sample was brought into the calorimeter until the stabilized water temperature was recorded. Initial and final temperatures of the water bath were recorded.

For each crushed concrete sample, four cold-hot and hot-cold temperature changes were performed. Also, the initial and final temperatures and the corresponding time intervals for the changing temperatures were recorded. For each temperature change, a heat capacity value was calculated. Two concrete samples were tested for each set. The results were averaged to represent the heat capacity of the PCC from each concrete set. The results from the two concrete sets were averaged to represent the heat capacity of the concrete from the AHD project.

Design Levels 1 and 2

Levels 1 and 2 of the DG 2002 require measuring the HC of the concrete. The results of the PCC heat capacity tests at Design Levels 1 and 2 are presented in Table 4-24. Detailed calculations are showed in Appendix A-5.

Table 4-24: Measured heat capacity

Heat capacity J/kg-°C		Age 28 days	
Concrete set 1	sample 1	947	968
	sample 2	953	
Concrete set 2	sample 1	882	884
	sample 2	886	
Average		926	

Design Level 3

Level 3 of the DG 2002 suggests typical values between 670 and 2,095 J/kg-°C for the heat capacity of concrete. The test results from the AHD concrete project is in the range of recommended values in the DG 2002 for the heat capacity of the PCC; however, it is close to the lower level of the suggested range, which is a disadvantage for concrete pavement, since a lower HC increases the transverse cracking of the PCC slab.

From the DG 2002 sensitivity analysis, it was found that a change in the HC from 2,095 to 670 J/kg-°C resulted in a change in the final transverse cracking (at the end of a 30-year design life) from 8.6% to 27.6%.

4.6.9 Thermal conductivity of PCC

Thermal conductivity is a measure of the ability of a solid or liquid to transfer heat. The thermal conductivity of PCC is calculated using the heat capacity (HC), thermal diffusivity (TD), and unit weight of hardened PCC (ρ), as shown in Equation 4-16.

$$\text{Thermal Conductivity (J/m-sec-°C)} = HC \times TD \times \rho \qquad \text{Equation 4-16}$$

Where:

HC = heat capacity of PCC (J/kg-°C)

TD = thermal diffusivity of PCC (m^2/sec)

ρ = unit weight of hardened PCC (kg/m^3).

The Thermal Diffusivity (TD) of the PCC was measured using thermocouples inserted in concrete samples (Figure 4-20). The concrete samples were cured in a moist room for 28 days before testing.

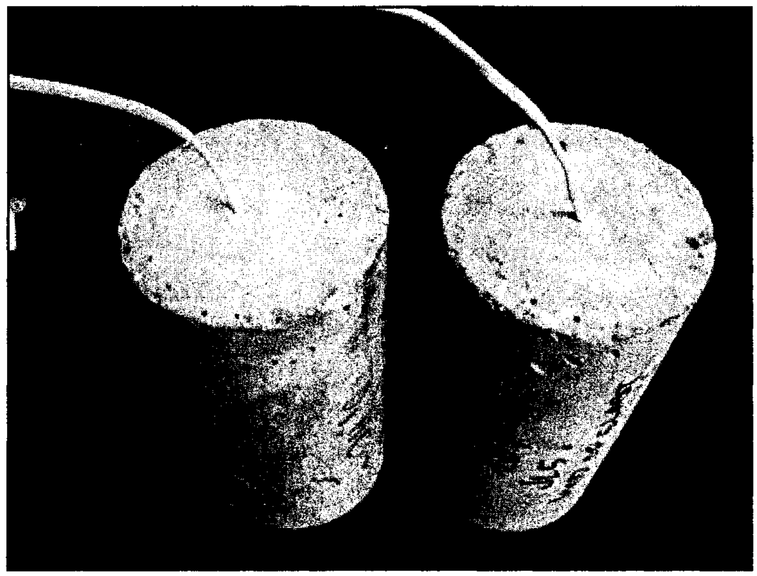

Figure 4-20: Thermocouples for measuring thermal diffusivity of PCC

To measure the TD according to CRD-C 36-73, a hot concrete sample was placed in a water bath with running cold water. The concrete and water bath temperatures were measured continuously. The time between the two specific times during the experiment (when the difference between the two temperatures was 44°C and 11°C) must be recorded. Two tests were performed for each concrete set sample. Based on the recorded time and the temperature differences, the thermal diffusivity of the concrete was calculated using Equation 4-17.

$$TD = \frac{60 \times \log\left(\frac{T_2}{T_1}\right)}{\left(\frac{5.783}{r^2} + \frac{\pi^2}{l^2}\right) \times (T_2 - T_1)} \qquad \text{Equation 4-17}$$

Where:

TD = thermal diffusivity (ft^2/hr)

T_2 = time corresponding to temperature of 11°C (hr)

T_1 = time corresponding to temperature of 44°C (hr)

r = concrete cylinder radius (ft)

l = concrete cylinder height (ft).

The results from the two tests were averaged to represent the thermal diffusivity of that concrete sample.

Design Levels 1 and 2

According to the DG 2002, the TD must be measured for the PCC at Levels 1 and 2. The thermal diffusivity test results and thermal conductivity calculation results are reported in Table 4-25. Detailed calculations of the thermal diffusivity tests are shown in Appendix A-6.

Table 4-25: Measured thermal conductivity and thermal diffusivity

		Thermal diffusivity m^2/sec		**Thermal conductivity J/m-sec-°C**	
Concrete set 1	sample 1	5.58E-07	5.69E-07	1.21	1.23
	sample 2	5.80E-07		1.26	
Concrete set 2	sample 1	4.73E-07	4.74E-07	0.94	0.95
	sample 2	4.74E-07		0.95	
Average		5.21E-07		1.09	

Design Level 3

DG 2002 suggests typical values of between 3.462 and 0.865 J/m-sec-°C for the thermal conductivity of PCC (DG 2002, Part 2, Chapter 2). The thermal conductivity test results for the PCC from the AHD project are close to the lower limit of the suggested range, which is a disadvantage for concrete pavement, since lower thermal conductivity values increase the transverse cracking of the PCC slab.

Based on DG 2002 sensitivity analysis, it was found that a change in the thermal conductivity from 3.462 to 0.865 J/m-sec-°C resulted in a change in the final transverse cracking (at the end of design life 30 years) from 8.2% to 65.0%.

4.6.10 Shrinkage of PCC (ε_c)

Drying shrinkage develops over time when PCC is subject to drying. If concrete rewets, the PCC expands to reverse a portion of the drying shrinkage; however, some of the shrinkage that occurs in the first drying is not reversible. The main factor that affects the reversible portion of the drying shrinkage is the ambient relative humidity. The ultimate shrinkage strain is the shrinkage strain that the PCC will develop due to prolonged exposure to drying conditions, which by definition is at 40% relative humidity (DG 2002, Part 2, Chapter 2). In DG 2002, the concrete shrinkage estimate is presented at three levels.

Design Level 1

At Level 1, the ultimate value of concrete mixture shrinkage should be determined in the laboratory. However, this is not a practical approach, since it could take several years to realize the ultimate shrinkage strain of concrete (a stable value). Field studies have shown that it could take at least 5 years to reach a stable maximum drying shrinkage value (DG 2002, Part 2, Chapter 2). Currently, there are no methods available to extrapolate short-term shrinkage measurements into ultimate shrinkage values. Agencies are encouraged to use AASHTO T160 standard to measure short-term (up to 180 days) shrinkage at 40% relative humidity in the laboratory. This approach is adopted in order to develop confidence in the ultimate shrinkage strains estimated using Level 2 and 3 approaches (DG 2002, Part 2, Chapter 2).

One of the inputs in the DG 2002 is the time required to develop 50% of ultimate shrinkage. At all design levels, a value of 35 days is recommended to be used for the time required to develop 50% of ultimate shrinkage. This value has been used for the calibrations of the pavement performance models in the DG 2002.

The results of the drying shrinkage test for the PCC from the Anthony Henday Drive project in Edmonton are presented in Figures 4-21 and 4-22. Shrinkage trends from Figures 4-21 and 4-22 are not as smooth as expected, due to testing at an uncontrollable temperature and humidity conditions. For estimating 50% of ultimate shrinkage from Figures 4-21 and 4-22, the shrinkages from trend lines at 35 days were used.

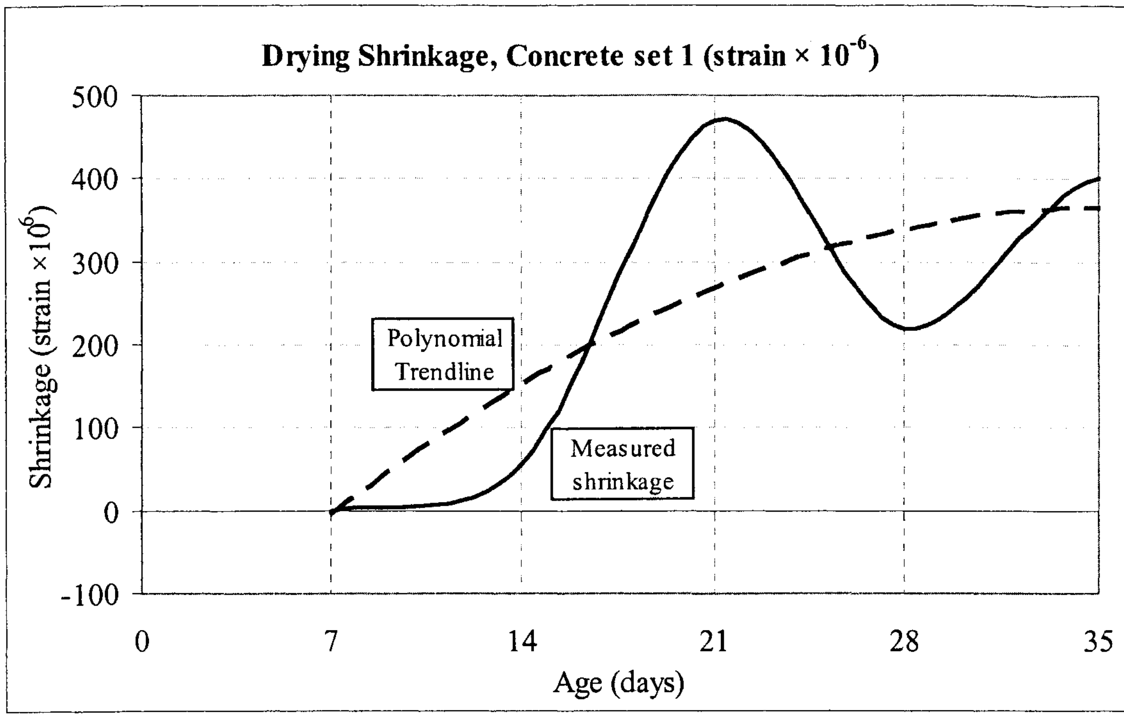

Figure 4-21: Drying shrinkage test results, concrete set 1 of samples

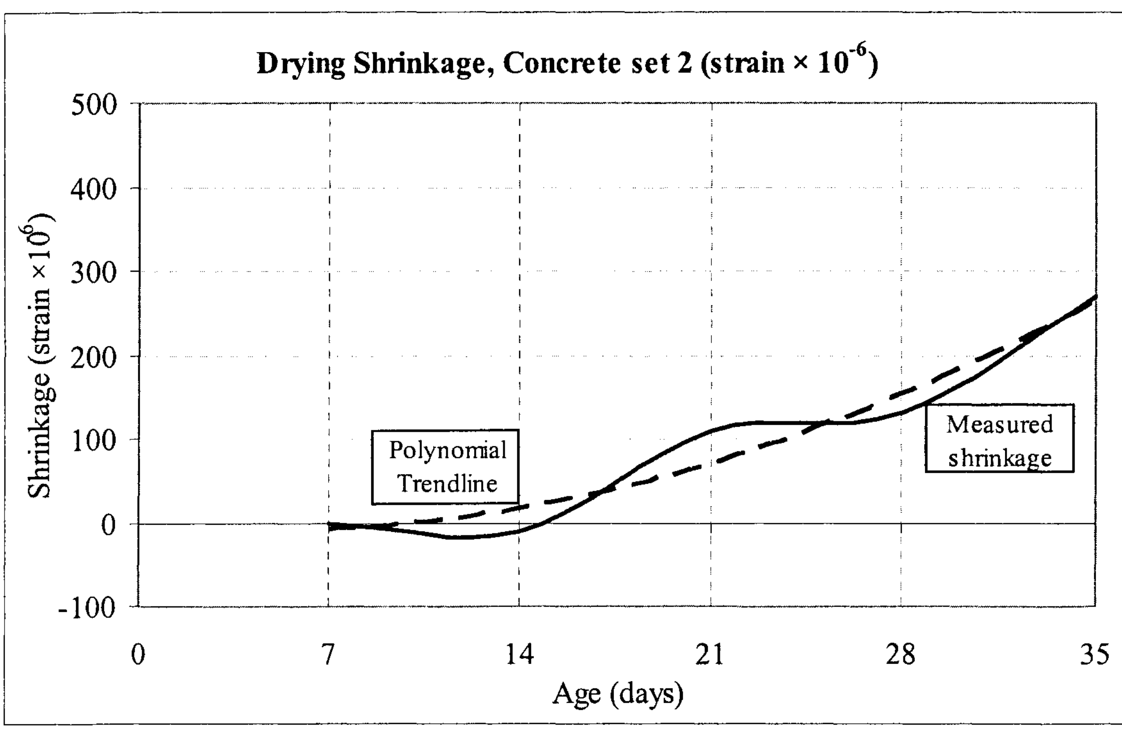

Figure 4-22: Drying shrinkage test results, concrete set 2 of samples

Design Level2

In Design Level 2, the ultimate shrinkage can be estimated through a standard correlation based on PCC mix parameters (cement type, cement content, and water-cement ratio), 28-day PCC compressive strength, and curing conditions. The model used to estimate the ultimate shrinkage is Equation 4-18.

$$\varepsilon_{su} = C_1 \times C_2 \times [26 \times w^{2.1} \times (f'_c)^{-0.28} + 270] \qquad \text{Equation 4-18}$$

Where:

ε_{su} = estimated ultimate drying shrinkage (strain$\times 10^{-6}$)

C_1 = cement type factor

C_2 = type of curing factor

w = water content (lb/ft^3)

f'_c = 28-day PCC compressive strength (psi).

For the AHD project, the concrete mix used cement Type I (C_1 = 1.0), samples were cured in 100% relative humidity (C_2 = 1.0), and the water content was 120 kg/m^3. The estimated ultimate drying shrinkage results are presented in Table 4-26.

Table 4-26: Estimated ultimate drying shrinkage at Level 2

Ultimate shrinkage ε_{su}, $\times 10^{-6}$	
Concrete set 1	430
Concrete set 2	433
Average	432

The test results from the two sets of concrete samples were close to each other.

Design Level 3

At Design Level 3, Equation 4-18 can be utilized with typical values or historical data for w and f'_c.

Another commonly used concrete shrinkage model is the CEB-FIP MC-90 model. This model was compared with a measured value and the suggested model DG 2002 (Equation 4-18). The CEB-FIP MC-90 prediction model for ultimate shrinkage is Equation 4-19.

$$\varepsilon_{cs}(t,t_s) = 1.55 \times \left[1-\left(\frac{RH}{RH_0}\right)^3\right] \times \left[160+10\times\beta_{sc}\times\left(9-\frac{f_{cm}}{f_{cm0}}\right)\right] \times \left[\frac{(t-t_s)/t_1}{350\times(h/h_0)^2+(t-t_s)/t_1}\right]^{0.5} \times 10^{-6}$$

Equation 4-19

Where:

ε_{cs} = total shrinkage

t = age of concrete (days); t = 35 days

t_s = age of concrete (days) at the beginning of shrinkage; t_s = 7 days

t_l = 1 day

RH = relative humidity of the ambient atmosphere (%); RH = 40%

RH_0 = 100%

β_{sc} = coefficient depending on the type of cement; β_{sc} = 5 for normal hardening cements

f_{cm} = mean compressive strength of concrete at age of 28 days (MPa)

f_{cm0} = 10 MPa

$h = 2A_c/u$

A_c = the cross-sectionl area of the concrete sample; Ac = 75×75 mm^2

u = perimeter of the cross-sectional in contact with the atmosphere; u = 75×4 mm

h_0 = 100 mm.

The predicted shrinkages at the age of 35 days for the concrete from the AHD project are 345 and 355×10^{-6} for set 1 and set 2 of the concrete samples, respectively. The 35-days drying shrinkage testing results, in Figures 4-21 and 4-22, (50% of ultimate shrinkage) are 372 and 272×10^{-6}, predicting a ultimate shrinkage of 745 and 544×10^{-6}, higher than the values determined by prediction model, Equation 4-18, from Level 2 of the DG 2002 (430 and 433×10^{-6}) and the CEB-FIP MC-90, Equation 4-19 (345 and 355×10^{-6}) (Figure 4-23).

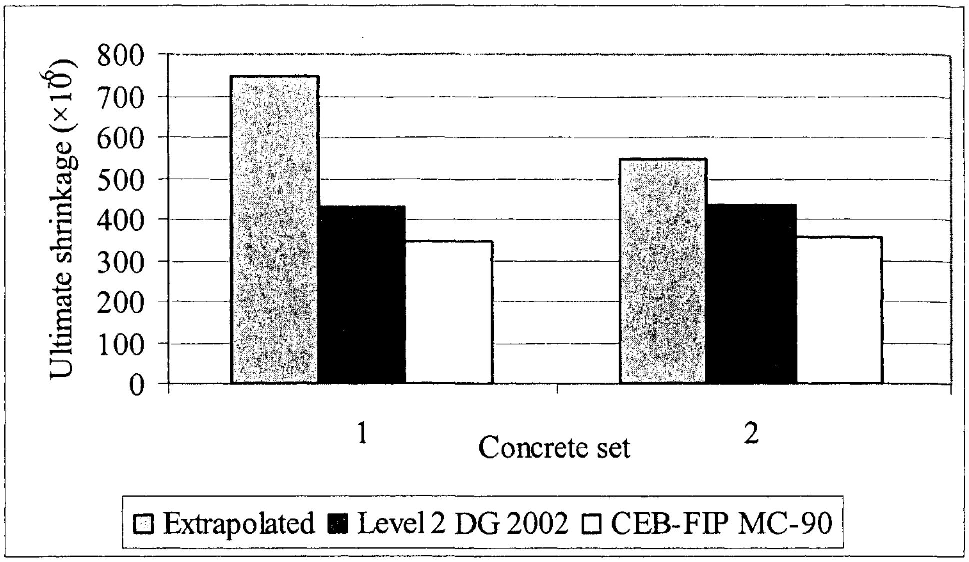

Figure 4-23: Estimated ultimate shrinkage comparison

4.7 Implementation of the DG 2002 for concrete pavements in Alberta

When a highway agency uses the DG 2002 for pavement design, a decision should be made related to which level of the program should be considered. In terms of material characterization, Table 4-27 presents a summary of PCC material inputs in DG 2002 for each level. The table also shows the required number of concrete samples and days needed to perform the concrete testing for Level 1 of the DG 2002.

Concrete strength tests including compressive strength, the modulus of rupture, and indirect tensile strength are routine tests that can be performed in any concrete lab. The modulus of elasticity and Poisson's ratio can be measured using conventional concrete testing equipment and only needs devices to measure longitudinal and the radial deformation of the concrete specimen during testing. The thermal properties of concrete, which were shown to significantly impact the performance of pavement, require more attention in the implementation of the DG 2002. There is a standard AASHTO procedure for the coefficient of thermal expansion; however, few concrete technologists are practicing this test. Therefore, for the implementation of Level 1 of the DG 2002, it is necessary to have specific training for concrete technologists in the laboratory. The good

Table 4-27: PCC concrete material inputs in the DG 2002

	DG 2002 - JPCP - Material characterization		
	Level 1	**Level 2**	**Level 3**
Number of concrete samples required	50	12	3
Time required for testing results (days)	90	90	28
Testing complexity	High	High-Medium	Low
Strength properties:			
Modulus of elasticity	Measured at 7, 14, 28, 90 days, estimated 20y/28d	-	Measured at 28-day (optional)
Modulus of rupture	Measured at 7, 14, 28, 90 days, estimated 20y/28d	-	Measured modulus of rupture or compressive strength at 28-day
Compressive strength	-	Measured at 7, 14, 28, 90 days, estimated 20y/28d	
Poisson's ratio	Measured	Use typical values	Use typical values
Thermal properties:			
Coefficient of thermal expansion	Measured	Use Equation 4-15 and/or Table 4-23	Use typical values (upper level of the range)
Thermal conductivity	Measured after 28 days moist-cured	Measured	Use typical values (lower level of the range)
Heat capacity	Measured	Measured	Use typical values (lower level of the range)
Shrinkage:			
Ultimate shrinkage at 40% RH	Measured/Predicted	Use Equation 4-18	Use Equation 4-18 with typical values for inputs

news is that the equipment required for these tests is not expensive and can be found easily. At the moment there is not any ASTM or AASHTO test methods for heat capacity and thermal conductivity of concrete and in this study, testing procedures from U.S. Army were used. These procedures, CRD-C 124-73 and CRD-C 36-73, can be downloaded from:

http://www.wes.army.mil/SL/MTC/handbook/crd_c124.pdf and

http://www.wes.army.mil/SL/MTC/handbook/crd_c36.pdf, respectively.

As a general finding in this study, it was concluded that it is possible to measure the required concrete material properties in Level 1. Therefore, in terms of concrete characterization, implementing the DG 2002 will not be difficult. It seems that the main challenge in implementing the DG 2002 Level 1 is locating the required traffic data, which is a task beyond the scope of this study.

At Level 3 of DG 2002, at least three concrete specimens are required for determining the compressive strength of concrete at 28 days. Other concrete strengths are estimated based on models suggested in the DG 2002. For other PCC material inputs, typical values can be used. During this study, it was determined that, by testing concrete samples for all required properties, certain prediction models and values work better for typical concrete from Edmonton. By using the models suggested in this study, it is possible to use Level 3 with greater accuracy. Without a similar guideline to the one developed in this study the use of certain prediction models and typical values could cause gross errors.

For PCC material inputs, Level 2 of the DG 2002 requires a relatively low number of concrete samples compared to Level 1. At this design level, two thermal concrete tests (thermal conductivity and heat capacity) and one common strength concrete test series (compressive strength) are required. The rest of PCC material inputs in Level 2 of the DG 2002 are based on the prediction models and typical values. Because the sensitivity analysis indicated that thermal properties of concrete are significant factors controlling the performance of concrete pavement, it is recommended that further studies be conducted into the characterization of thermal properties for aggregate in concrete pavements in Alberta.

4.8 Performance prediction of the PCCP project

Performance prediction of pavements is important in selecting the best design alternative during the design process, as well as in establishing maintenance and rehabilitation details and life cycle cost analysis. Details of performance prediction for the AASHTO 1993 and PCA rigid pavement design methods were explained in Chapter 2. As it was explained in Chapter 2, the AASHTO 1993 and PCA methods each use different material, traffic, and environmental design inputs.

This section of the study attempts to use measured material characteristics from the AHD project and to compare the predicted performance of the PCCP using AASHTO 1993, PCA, and the DG 2002 for this project.

4.8.1 Performance prediction by the AASHTO method

The EBA design values for the modulus of rupture, the elastic modulus of PCC, and the slab thickness were changed according to the values measured. The results are presented in Table 4-28.

Table 4-28: Concrete test results used in the AASHTO 1993 method

Input Parameter	EBA Design	AHD Project
Modulus of rupture of PCC 28-day, MPa	4.70	4.18
Modulus of elasticity 28-day, MPa	25,000	24,810
Slab thickness, mm	311	225
Modulus of subgrade/subbase reaction, MPa/m	59.5	59.5
Traffic, 80 kN ESAL/direction	36.7×10^6	36.7×10^6
Load transfer coefficient	3.2	3.2
Reliability, %	95	95
Overall standard deviation	0.35	0.35
Drainage coefficient	1.0	1.0
Initial serviceability	4.5	4.5

Using the EBA design inputs for traffic and measured testing results from this study, the change in pavement serviceability ($\Delta PSI=P_I-P_T$) was determined, with 95% reliability, as an indication of the AHD project performance. This parameter can be converted to P_T in order to estimate the service life of the project (see Figure 4-24).

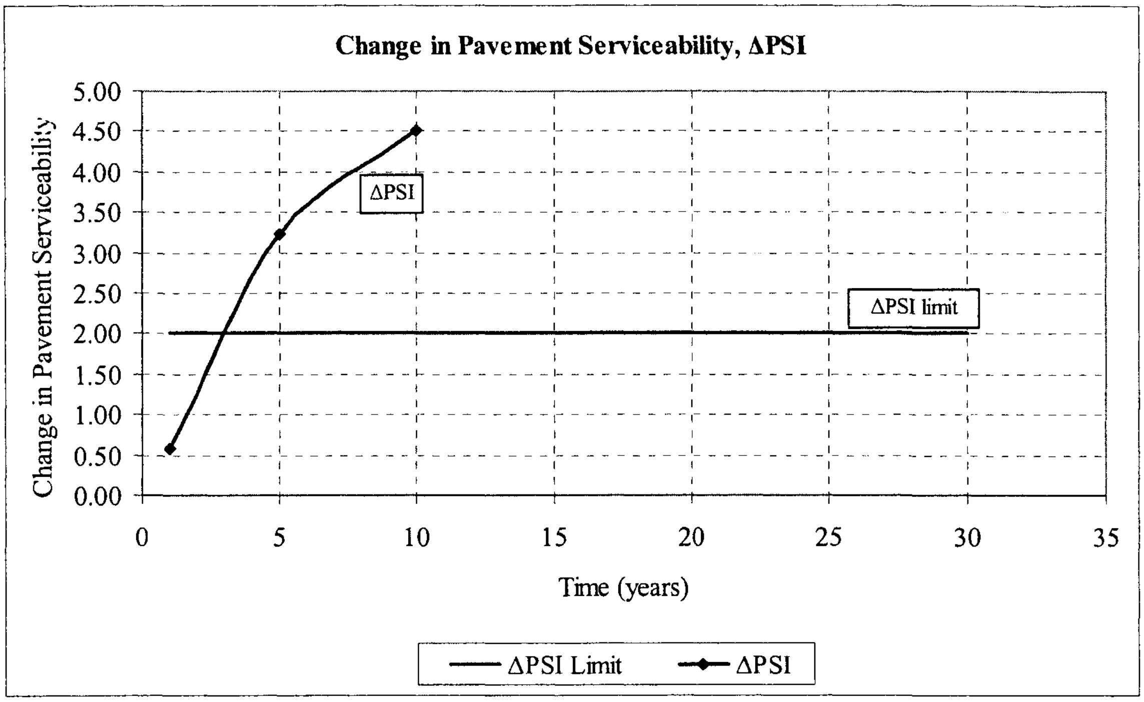

Figure 4-24: Change in pavement serviceability using specific test data

From Figure 4-24 it can be observed that by reducing three of the most important input parameters (the modulus of rupture, the modulus of elasticity of PCC, and the slab thickness) in AASHTO 1993 rigid pavement design, the predicted service life of the project is reduced from 30 to 3.2 years. This conclusion is based on the increase of ΔPSI resulting from an increase in traffic. Initial and terminal serviceability indices, according to EBA design, were 4.5 and 2.5, respectively, which leads to the ΔPSI limit of 2 (EBA 2003).

This performance prediction by AASHTO design method is unrealistic and indicates that the AASHTO rigid pavement design method overdesigns rigid pavement significantly.

4.8.2 Performance prediction by PCA method

The flexural strength design value of 4.20 MPa changed to 4.18 MPa using the 28 days concrete test (Table 4-29).

Table 4-29: Concrete test results used in PCA method

Input Parameter	EBA Design	AHD Project
Modulus of rupture of PCC 28-day, MPa	4.20	4.18
Modulus of subgrade/subbase reaction, MPa/m	38	38
Average Daily Truck Traffic (ADTT), (Load category 3)	3,200	3,200
Design life, years	30	30
Load transfer	Dowels and concrete shoulder	Dowels and concrete shoulder
Load Safety Factor (LSF)	1.2	1.2
Slab thickness, mm	224	224
Axle load category	Input axles	Input axles

Although the modulus of rupture of PCC is an important parameter in the PCA design method, the slight decrease in concrete strength still provides acceptable erosion and fatigue outputs at the end of the 30-year design life, as is presented in Table 4-30.

Table 4-30: Performance prediction by PCA method using AHD project data

Output parameter	Predicted by PCA (end of design life 30 years)
Erosion (%)	35.21
Fatigue (%)	14.55

These results show that the AHD concrete flexural strength results used in the PCA method, together with other design values, suggests an acceptable performance up to 30 years for AHD project.

4.8.3 Performance prediction by the DG 2002 method

In the DG 2002 method, material design input parameters measured in this study were used to predict the performance of the AHD project. These testing results are presented in Table 4-31.

Table 4-31: Concrete test results used in the DG 2002 method

Input parameter	Age (days)	AHD Test Data
Unit weight, kg/m^3	-	2,275
Poisson's ratio	28	0.17
Coefficient of thermal expansion, 10^{-6}/°C	-	12.69
Thermal conductivity of PCC, J/m×s×°C	28	2.13
Heat capacity of PCC, J/kg×°C	-	926
Shrinkage of PCC, 10^{-6} microns	35	542
Modulus of elasticity, MPa	7	23,621
	14	23,425
	28	24,809
	90	28,067
	20y/28d	1.2*
Modulus of rupture, MPa	7	3.40
	14	3.81
	28	4.18
	90	4.44
	20y/28d	1.2*

*Typical values suggested by DG 2002

For Design Level 1, 30,000 AADT and the values from Column "AHD Test Data" were used as input parameters for the PCC material characterization (Table 4-30). In Design Level 1, a long-term ratio of 20y/20d of elastic modulus and flexural strength values is required. After performing the AHD concrete tests, specific project data for Design Level 1 of the DG 2002 were used. At 95% reliability, according to the DG 2002 Level 1, the pavement performance of the AHD project was predicted and the results are shown in Table 4-32 and Figures 4-25, 4-26, and 4-27 for roughness, transverse cracking, and faulting, respectively. These performance predictions are based on real concrete property measurements; however, it is not possible to use environmental factors for Edmonton at this stage. Therefore, a weather station in Seattle, USA was used in this analysis.

As Figures 4-25, 4-26, and 4-27 indicate, for all three performance prediction indicators in DG 2002, the AHD project performs beyond its design life (30 years). IRI and faulting could be considered as the parameters which control the performance of this project.

Table 4-32: Performance prediction by the DG 2002 method using the AHD project data

Output Parameter	Predicted by DG 2002 (end of design life 30 years)	Suggested limits (source)
Roughness (IRI), mm/m	2.6	3 (AI&T)
Transverse Cracking (TC), %	13.3	15 (Khanum et al. 2005)
Faulting, mm	4.4	5 (ODOT 2005)

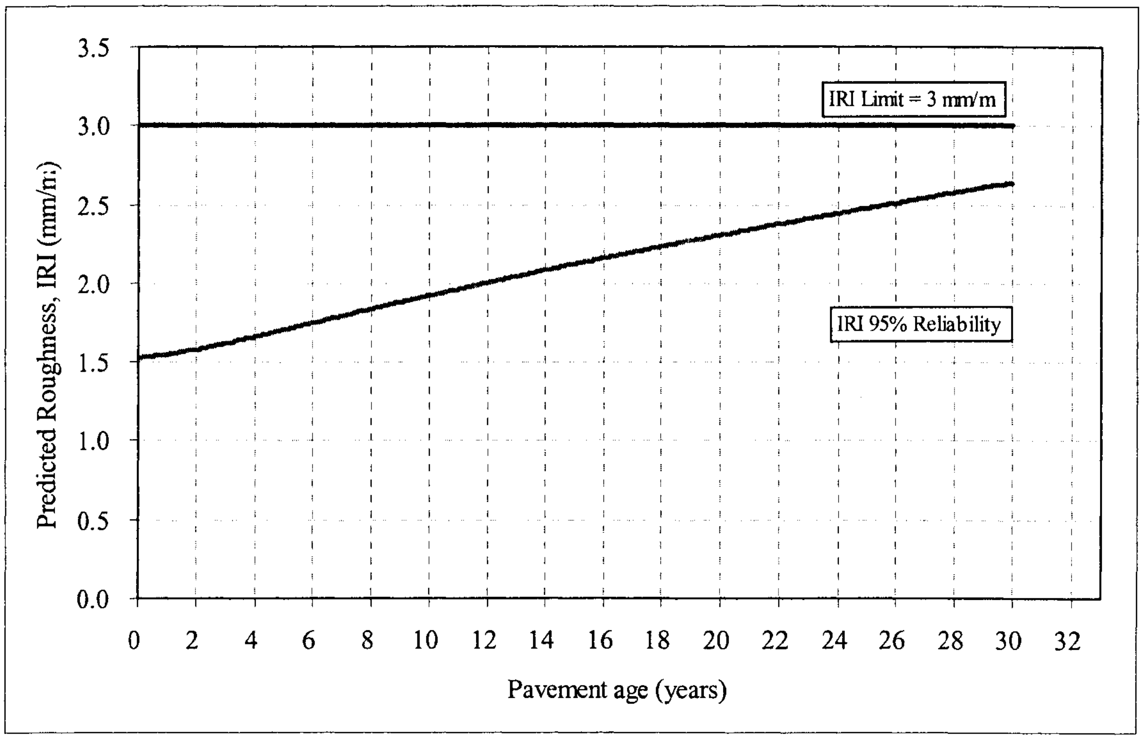

Figure 4-25: Predicted roughness by the DG 2002 using the AHD project data

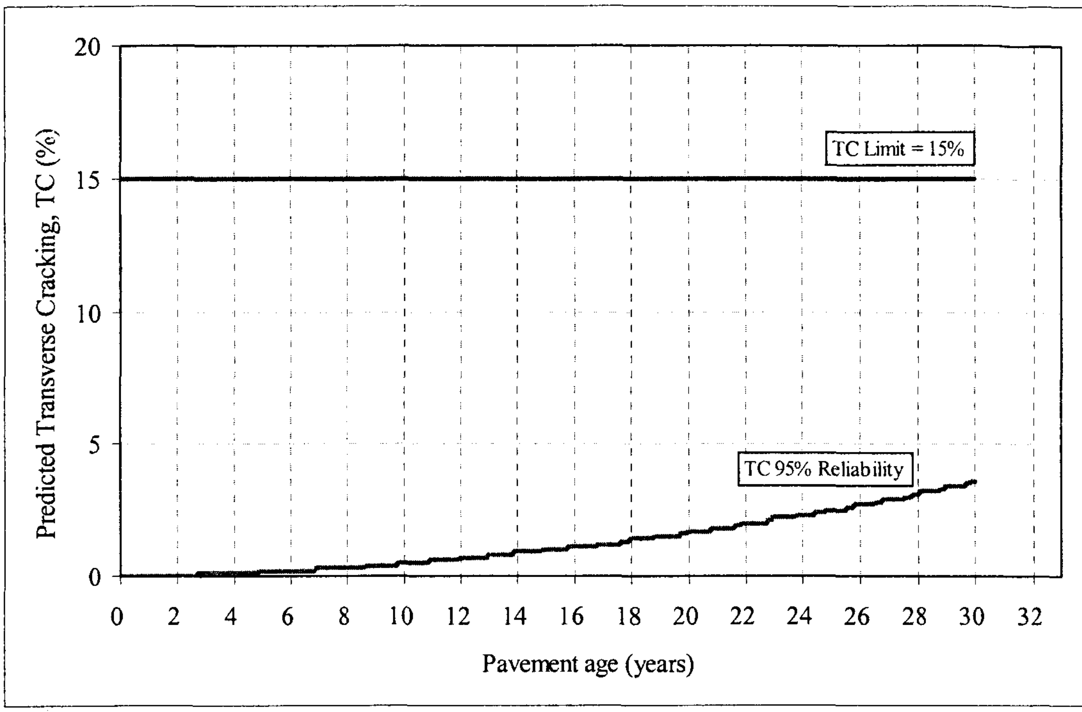

Figure 4-26: Predicted transverse cracking by the DG 2002 using the AHD project data

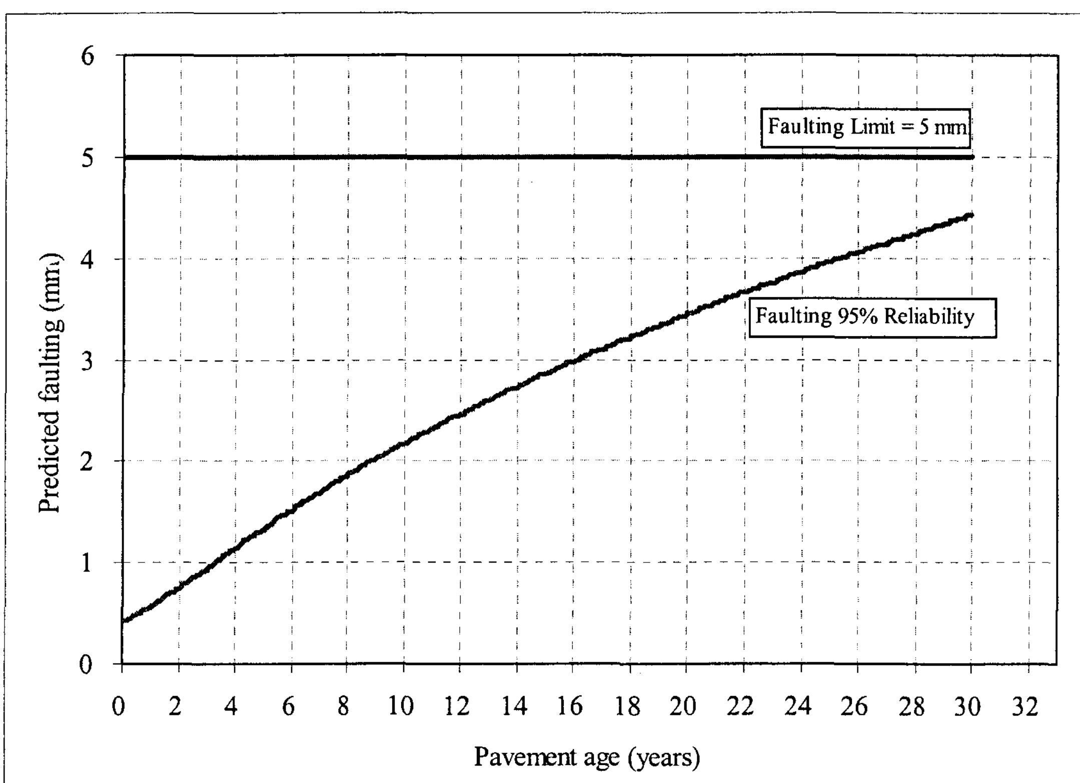

Figure 4-27: Predicted faulting by the DG 2002 using the AHD project data

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

The existing rigid pavement design methods (AASHTO 1993 and PCA) were compared with the new mechanistic-empirical Design Guide 2002 (DG 2002) method for designing new Jointed Plain Concrete Pavements (JPCP). Sensitivity analyses were performed on these three rigid pavement design methods to find the most important factors in each method:

- The sensitivity analysis on the AASHTO 1993 method showed that slab thickness and the strength of the concrete are the most important input parameters.
- Sensitivity analysis on the PCA method revealed that slab thickness, the modulus of subgrade reaction, the concrete strength, and the traffic load safety factors are most important in terms of fatigue performance. From the perspective of erosion performance, the traffic load safety factor is the most important input parameter followed by the slab thickness, traffic, and the modulus of subgrade reaction. Concrete strength has no effect on erosion based on this design method.
- The sensitivity analysis for the DG 2002 method revealed that the coefficient of thermal expansion, the permanent curl/warp temperature difference, and heavy traffic (the percentage of heavy vehicles) are the most important input parameters. Other important input parameters in the DG 2002 are slab thickness, joint spacing, dowel diameter, initial IRI, thermal conductivity, heat capacity, and the compressive strength of the PCC.

In comparing the results of the AASHTO 1993 and PCA design methods, for the AHD project the AASHTO 1993 over-designed the slab thickness significantly relative to the PCA method.

Typical concrete from a PCCP project in Edmonton (Anthony Henday Drive) was characterized based on the DG 2002. The CEB-FIP MC-90 models for the prediction of compressive strength, the elastic modulus, and the shrinkage of PCC were also employed to compare their predictions with values measured for these properties. Several conclusions were drawn based on the characterizations of concrete materials:

- If highway agencies plan to implement the DG 2002, it is feasible to characterize concrete pavement for all the required material inputs using the DG 2002. Determining the thermal properties of concrete requires training and some inexpensive laboratory equipment.
- Compressive strength test results from two concrete sets were found to be very consistent (low variability in results) at all concrete ages. It was observed that the CEB-FIP MC-90 model underestimates early age (7-day) and overestimates the compressive strength of concrete on the Anthony Henday Drive (AHD) concrete project for late ages (90-day).
- The modulus of elasticity testing results from the AHD project on both concrete sets were compared and found to be very consistent at all concrete ages. The modulus of elasticity predicted at Level 2 gave higher values than the values measured for the modulus of elasticity at all other concrete ages. At Level 3, the estimated elastic modulus based on the modulus of rupture gave results closer to those values measured than those estimated based on the compressive strength. It was observed that the local values of the elastic modulus were lower than those predicted by the CEB-FIP MC-90. The measured Poisson's ratios were in the range suggested by the DG 2002.
- The model suggested at Level 2 of the DG 2002, which estimated the modulus of rupture based on the compressive strength results, overestimated this parameter. On the other hand, the use of the 28-day modulus of rupture gives closer results to those measured.
- The indirect tensile strength, estimated based on the compressive strength results at Level 2 of the DG 2002, gave lower values than those values measured at all other ages of concrete. In a comparison of the two approaches through an estimate of the indirect tensile strength at Level 3, the second approach, which uses the 28-day compressive strength value, gives closer results to the measured values.
- The unit weight of concrete measured in the AHD project is close to the lower value in the range suggested by the DG 2002. This could be due to the lighter aggregates used in northern Alberta on construction projects.

- The Coefficient of Thermal Expansion (CTE) test results for concrete from the AHD project fall within the suggested ranges of the DG 2002, but are closer to the upper level. Since a higher CTE increases the roughness, transverse cracking, and faulting of the PCC slab, this is a disadvantage for Alberta concrete.
- The heat capacity (HC) test results for concrete from the AHD project are close to the lower level of the suggested range, which is a disadvantage since a lower HC increases the transverse cracking of the PCC slab. If Level 3 of the DG 2002 is used by highway agencies in Alberta, values closer to the lower level of the range should be used as typical values.
- The thermal conductivity test results on the PCC from the AHD project are close to the lower level of the suggested range by DG 2002, which is a disadvantage since lower thermal conductivity values increase the transverse cracking of the PCC slab. If Level 3 of the DG 2002 is used by highway agencies in Alberta, lower values of the range should be used as typical values.
- The measured shrinkage values, which are estimated to represent 50% of the ultimate shrinkage, were higher than the ultimate shrinkage values determined by the prediction model of the DG 2002 Level 2 for the AHD concrete project. The ultimate shrinkage values determined by the prediction model of CEB-FIP MC-90 for shrinkage were lower than the measured values. As a recommendation for highway agencies in Alberta, the use of the DG 2002 prediction model in estimating the ultimate shrinkage of concrete is considered to be a reliable solution.
- A comparison of the pavement performance predictions by AASHTO, PCA, and the DG 2002 for the AHD project demonstrates that the AASHTO method is the only one that predicts a lower service life than the designed service life of 30 years. If one considers the concrete slab thickness as the only variable among the inputs of the three design methods, a greater slab thickness is required to satisfy the design life requirements of the AASHTO method. This result reinforces the concern that the AASHTO rigid pavement design method overdesigns the slab thickness.

- In terms of the material characterization for the PCCP using the DG 2002, it is recommended that Levels 1 and 2 be implemented for the future PCCP projects.
- Since the CTE of concrete is a very important parameter in rigid pavement performance, affecting all three PCCP outputs of the DG 2002, more research in the area of the thermal properties of concrete in Alberta must still be conducted.

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CHAPTER 7: APPENDIX

Appendix A-1: Concrete unit weight

Unit Weight, $\omega = \rho_w * M_{dry}/(M_{dry} - M_{immers})$

ω = unit weight of hardened concrete, kg/m^3

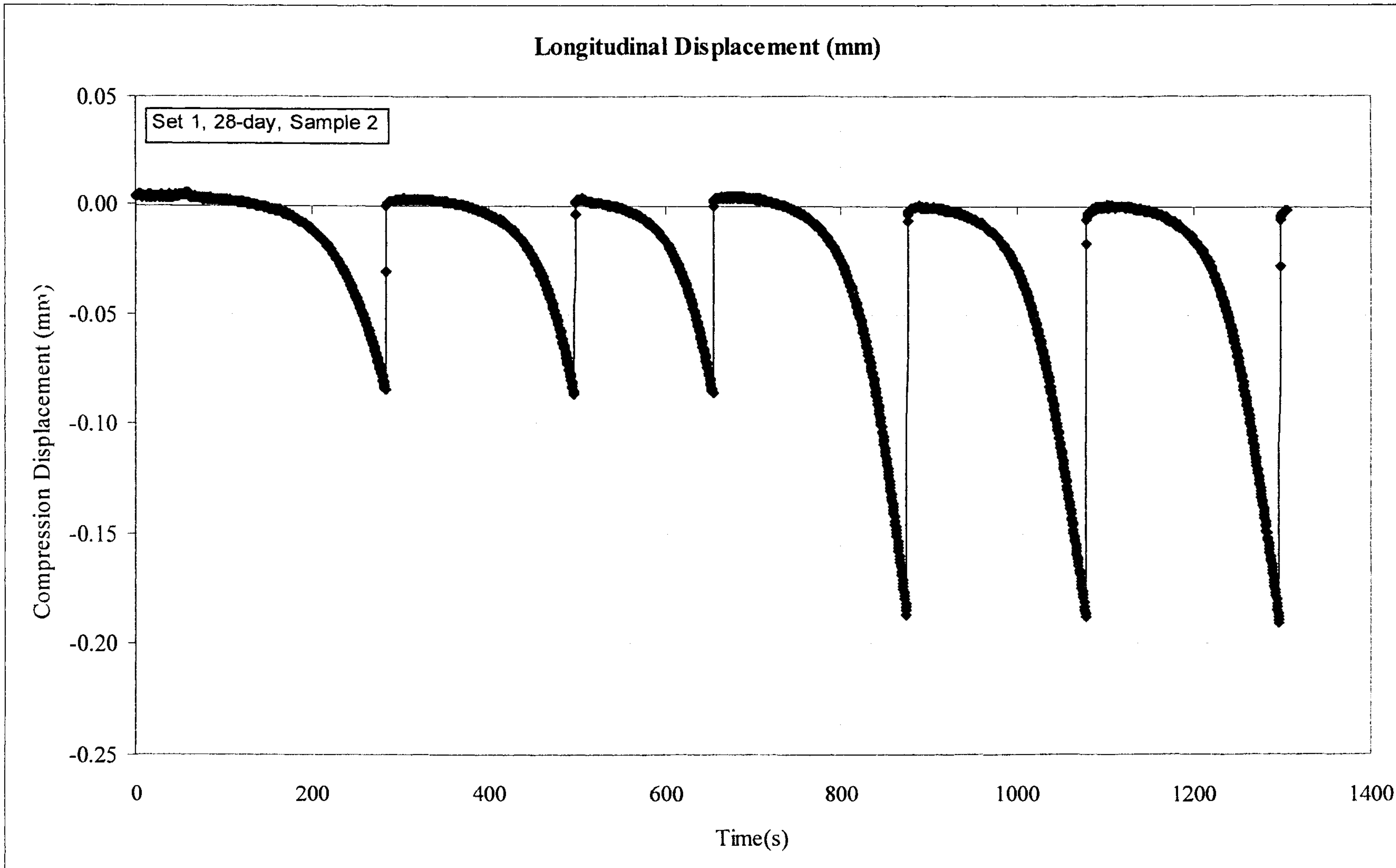
ρ_w = unit weight of the water, 1000 kg/m^3

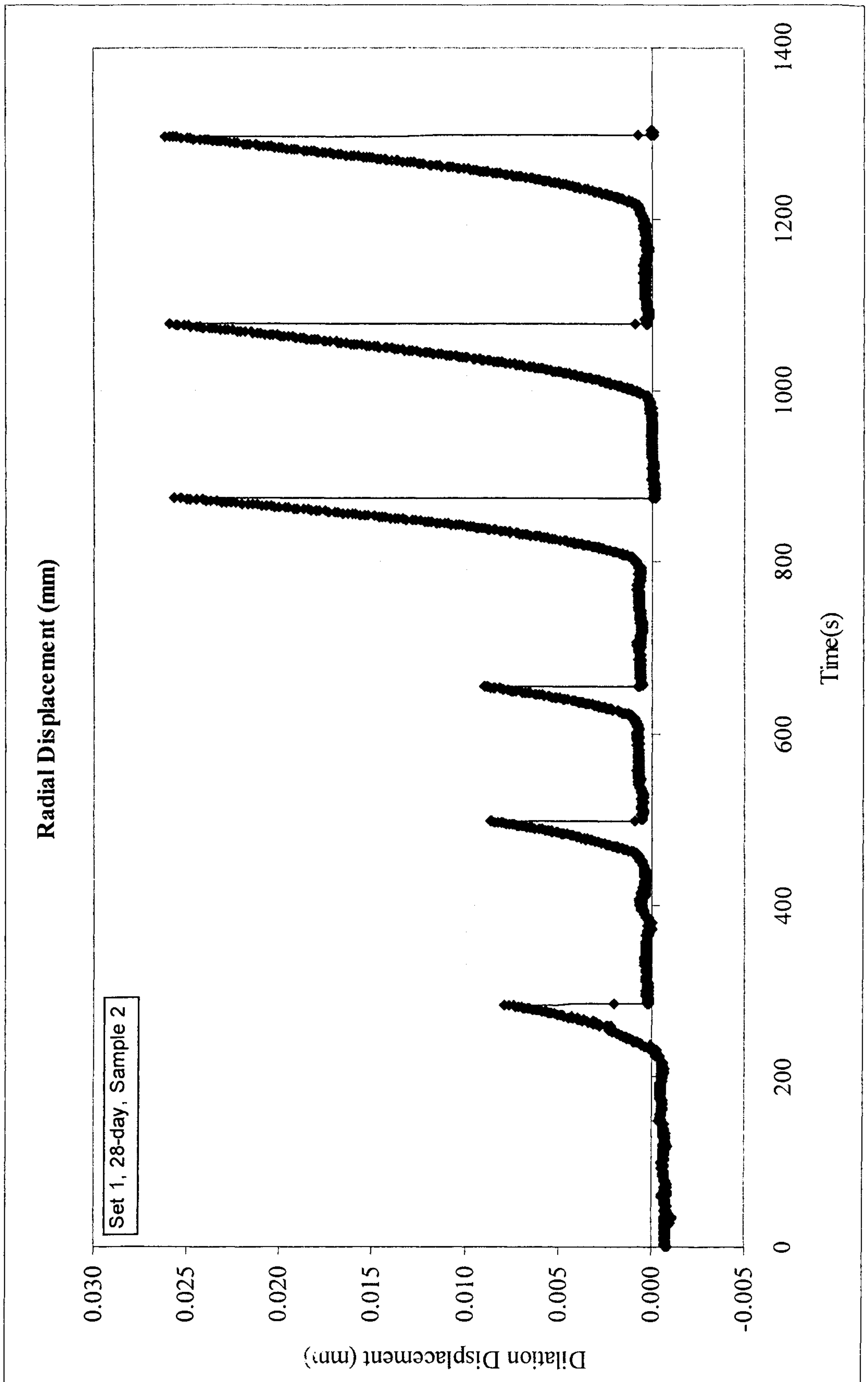
M_{dry} = mass of dry sample, kg

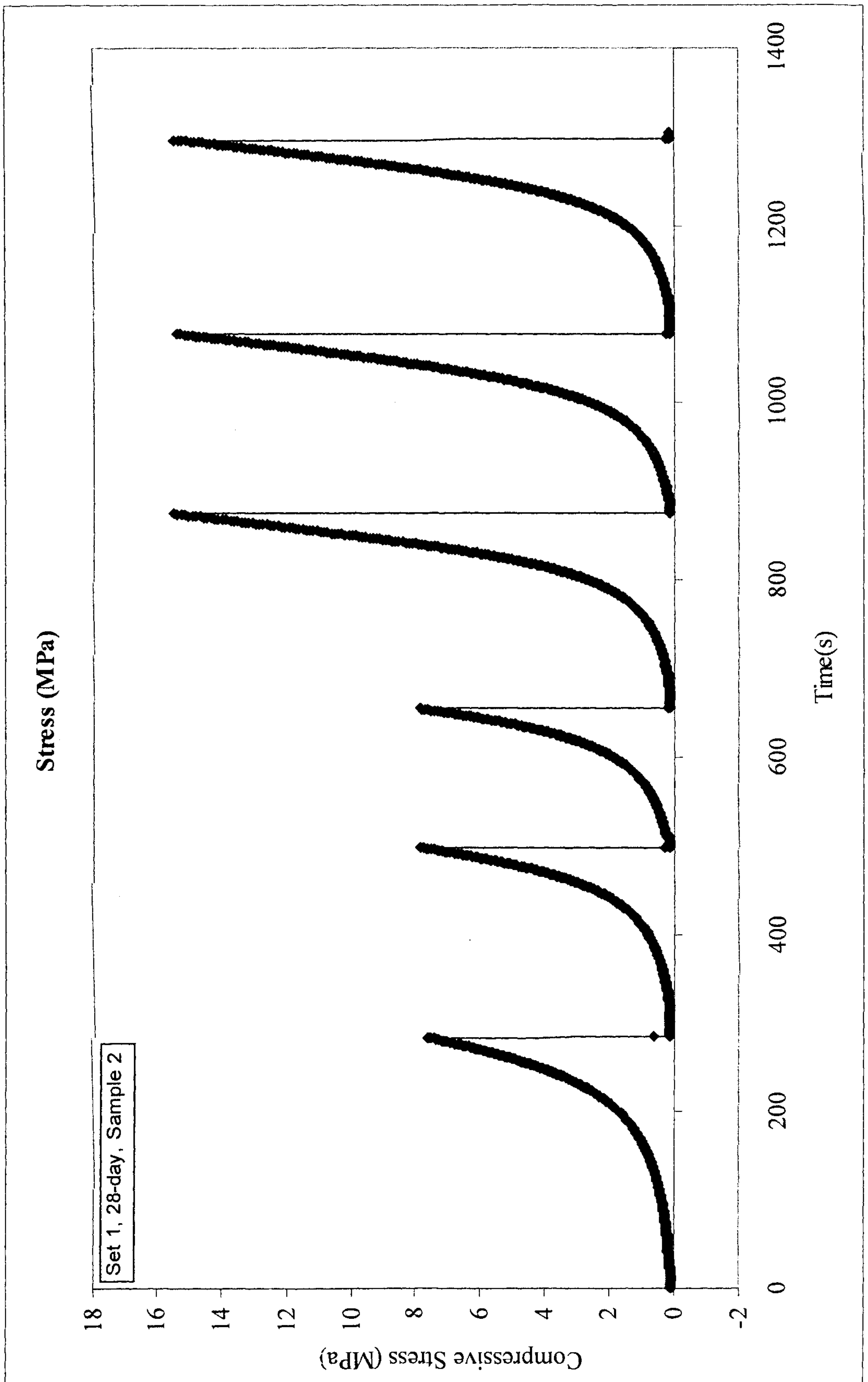
M_{immers} = mass of immersed sample, kg

Concrete set of sample	Sample No.	M_{dry} (kg)	M_{immers} (kg)	Unit weight ω (kg/m^3)	
1	1	3.6726	2.066	2286	2283
	2	3.7092	2.082	2279	
2	1	3.7526	2.094	2263	2266
	2	3.7427	2.093	2269	
Average, (kg/m^3)				2274	

Appendix A-2: Modulus of elasticity and Poisson's ratio: displacement and loading charts (samples)


Longitudinal Displacement (mm)
Set 1, 28-day, Sample 2
Compression Displacement (mm)
0.05
0.00
-0.05
-0.10
-0.15
-0.20
-0.25
0
200
400
600
800
1000
1200
1400
Time(s)


Radial Displacement (mm)
Set 1, 28-day, Sample 2
Dilation Displacement (mm)
Time(s)
0.030
0.025
0.020
0.015
0.010
0.005
0.000
-0.005
0
200
400
600
800
1000
1200
1400



Stress (MPa)
Set 1, 28-day, Sample 2
Compressive Stress (MPa)
Time(s)
18
16
14
12
10
8
6
4
2
0
-2
0
200
400
600
800
1000
1200
1400

Appendix A-3: Modulus of elasticity and Poisson's ratio testing results

Elastic modulus and Poisson's ratio, set 1 of samples, 7 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 7 days	at 40% max load	12.107	-0.000508	1.030E-04
		12.136	-0.000534	1.053E-04
		12.079	-0.000540	1.072E-04
	at 0.00005 strain	0.999	-0.000050	3.842E-06
		0.742	-0.000050	3.457E-06
		0.657	-0.000050	4.994E-06
Sample 2, 7 days	at 40% max load	12.072	-0.000456	6.762E-05
		12.122	-0.000445	7.159E-05
		12.176	-0.000487	7.606E-05
	at 0.00005 strain	0.989	-0.000053	-6.806E-06
		2.212	-0.000031	7.560E-06
		0.866	-0.000054	2.091E-06

Sample 1, 7 days	$E_1 =$	24,228	23,702	MPa
	$E_2 =$	23,548		
	$E_3 =$	23,329		
	$\mu_1 =$	0.22	0.21	-
	$\mu_2 =$	0.21		
	$\mu_3 =$	0.21		
Sample 2, 7 days	$E_1 =$	27,325	26,085	MPa
	$E_2 =$	25,077		
	$E_3 =$	25,854		
	$\mu_1 =$	0.18	0.17	-
	$\mu_2 =$	0.16		
	$\mu_3 =$	0.17		

Elastic modulus and Poisson's ratio, set 1 of samples, 14 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 14 days	at 40% max load	13.586	-0.000572	8.885E-05
		13.891	-0.000602	9.262E-05
		13.897	-0.000606	9.083E-05
	at 0.00005 strain	1.454	-0.000051	5.931E-06
		0.901	-0.000049	4.601E-06
		0.825	-0.000051	2.150E-06
Sample 2, 14 days	at 40% max load	13.938	-0.000591	7.684E-05
		13.820	-0.000596	8.100E-05
		13.849	-0.000600	8.265E-05
	at 0.00005 strain	1.246	-0.000050	-2.169E-06
		0.879	-0.000050	4.709E-07
		0.804	-0.000050	1.458E-06

Sample 1, 14 days	$E_1 =$	23,262	23,431	MPa
	$E_2 =$	23,534		
	$E_3 =$	23,498		
	$\mu_1 =$	0.16	0.16	-
	$\mu_2 =$	0.16		
	$\mu_3 =$	0.16		
Sample 2, 14 days	$E_1 =$	23,468	23,624	MPa
	$E_2 =$	23,707		
	$E_3 =$	23,698		
	$\mu_1 =$	0.15	0.15	-
	$\mu_2 =$	0.15		
	$\mu_3 =$	0.15		

Elastic modulus and Poisson's ratio, set 1 of samples, 28 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 28 days	at 40% max load	15.070	-0.000631	9.158E-05
		15.061	-0.000634	9.405E-05
		15.039	-0.000631	9.354E-05
	at 0.00005 strain	1.367	-0.000052	2.042E-06
		0.965	-0.000050	1.823E-06
		0.942	-0.000051	2.129E-06
Sample 2, 28 days	at 40% max load	15.048	-0.000559	8.610E-05
		15.059	-0.000569	8.845E-05
		14.990	-0.000573	8.905E-05
	at 0.00005 strain	1.868	-0.000050	2.531E-06
		1.546	-0.000050	2.589E-07
		1.524	-0.000050	1.817E-06

Sample 1, 28 days	E_1 =	23,584	23,995	MPa
	E_2 =	24,128		
	E_3 =	24,274		
	μ_1 =	0.15	0.16	-
	μ_2 =	0.16		
	μ_3 =	0.16		
Sample 2, 28 days	E_1 =	25,914	25,907	MPa
	E_2 =	26,050		
	E_3 =	25,756		
	μ_1 =	0.16	0.17	-
	μ_2 =	0.17		
	μ_3 =	0.17		

Elastic modulus and Poisson's ratio, set 1 of samples, 90 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 90 days	at 40% max load	16.175	-0.000541	7.911E-05
		15.963	-0.000563	8.269E-05
		15.989	-0.000587	8.553E-05
	at 0.00005 strain	2.414	-0.000053	6.958E-06
		1.853	-0.000049	7.654E-06
		1.450	-0.000050	7.411E-06
Sample 2, 90 days	at 40% max load	16.004	-0.000465	6.949E-05
		15.901	-0.000506	8.111E-05
		16.075	-0.000468	8.483E-05
	at 0.00005 strain	2.623	-0.000050	-4.230E-06
		1.589	-0.000050	5.718E-07
		1.122	-0.000051	8.819E-07

Sample 1, 90 days	E_1 =	28,047	27,538	MPa
	E_2 =	27,486		
	E_3 =	27,082		
	μ_1 =	0.15	0.15	-
	μ_2 =	0.15		
	μ_3 =	0.15		
Sample 2, 90 days	E_1 =	32,238	33,137	MPa
	E_2 =	31,385		
	E_3 =	35,788		
	μ_1 =	0.18	0.19	-
	μ_2 =	0.18		
	μ_3 =	0.20		

Elastic modulus and Poisson's ratio, set 2 of samples, 7 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 7 days	at 40% max load	10.004	-0.000436	4.997E-05
		10.031	-0.000370	5.481E-05
		10.089	-0.000467	5.537E-05
	at 0.00005 strain	1.139	-0.000051	2.425E-06
		1.377	-0.000049	2.099E-06
		0.619	-0.000051	5.533E-07
Sample 2, 7 days	at 40% max load	9.997	-0.000432	5.792E-05
		10.093	-0.000453	6.539E-05
		10.071	-0.000499	6.818E-05
	at 0.00005 strain	1.361	-0.000050	-4.901E-06
		0.826	-0.000050	-4.311E-06
		0.438	-0.000051	-2.033E-06

Sample 1, 7 days	E_1 =	22,983	24,231	MPa
	E_2 =	27,014		
	E_3 =	22,695		
	μ_1 =	0.12	0.14	-
	μ_2 =	0.16		
	μ_3 =	0.13		
Sample 2, 7 days	E_1 =	22,613	22,349	MPa
	E_2 =	22,974		
	E_3 =	21,459		
	μ_1 =	0.16	0.16	-
	μ_2 =	0.17		
	μ_3 =	0.16		

Elastic modulus and Poisson's ratio, set 2 of samples, 14 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 14 days	at 40% max load	11.332	-0.000482	6.793E-05
		13.489	-0.000593	9.651E-05
		13.306	-0.000591	9.880E-05
	at 0.00005 strain	1.407	-0.000051	-1.405E-06
		1.127	-0.000052	4.361E-06
		0.950	-0.000050	6.109E-06
Sample 2, 14 days	at 40% max load	13.304	-0.000531	8.597E-05
		13.216	-0.000543	8.752E-05
		13.342	-0.000552	8.829E-05
	at 0.00005 strain	1.947	-0.000049	4.112E-06
		1.348	-0.000051	3.005E-06
		1.436	-0.000051	2.778E-06

Sample 1, 14 days	$E_1 =$	22,960	22,851	MPa
	$E_2 =$	22,755		
	$E_3 =$	22,838		
	$\mu_1 =$	0.16	0.17	-
	$\mu_2 =$	0.17		
	$\mu_3 =$	0.17		
Sample 2, 14 days	$E_1 =$	23,620	23,792	MPa
	$E_2 =$	24,054		
	$E_3 =$	23,702		
	$\mu_1 =$	0.17	0.17	-
	$\mu_2 =$	0.17		
	$\mu_3 =$	0.17		

Elastic modulus and Poisson's ratio, set 2 of samples, 28 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 28 days	at 40% max load	14.171	-0.000557	8.970E-05
		14.171	-0.000585	9.343E-05
		14.190	-0.000599	9.279E-05
	at 0.00005 strain	1.314	-0.000052	6.968E-06
		0.916	-0.000051	5.684E-06
		0.823	-0.000050	2.979E-06
Sample 2, 28 days	at 40% max load	14.168	-0.000550	8.684E-05
		14.113	-0.000581	9.589E-05
		14.199	-0.000598	9.762E-05
	at 0.00005 strain	1.658	-0.000050	6.524E-06
		1.147	-0.000050	5.827E-06
		1.013	-0.000051	5.724E-06

Sample 1, 28 days	E_1 =	25,380	24,821	MPa
	E_2 =	24,754		
	E_3 =	24,328		
	μ_1 =	0.16	0.16	-
	μ_2 =	0.16		
	μ_3 =	0.16		
Sample 2, 28 days	E_1 =	25,031	24,515	MPa
	E_2 =	24,434		
	E_3 =	24,079		
	μ_1 =	0.16	0.17	-
	μ_2 =	0.17		
	μ_3 =	0.17		

Elastic modulus and Poisson's ratio, set 2 of samples, 90 days test

		Long. Stress (MPa)	Long. Strain	Radial Strain
Sample 1, 90 days	at 40% max load	16.382	-0.000462	1.002E-04
		16.382	-0.000462	1.002E-04
		16.441	-0.000537	1.145E-04
	at 0.00005 strain	2.910	-0.000050	1.297E-05
		2.910	-0.000050	1.297E-05
		1.832	-0.000049	1.395E-05
Sample 2, 90 days	at 40% max load	16.497	-0.000620	1.134E-04
		16.424	-0.000659	1.215E-04
		14.581	-0.000600	1.138E-04
	at 0.00005 strain	1.727	-0.000049	9.128E-06
		0.893	-0.000050	7.962E-06
		0.936	-0.000050	1.089E-05

Sample 1, 28 days	E_1 =	32,662	31,773	MPa
	E_2 =	32,662		
	E_3 =	29,994		
	μ_1 =	0.21	0.21	-
	μ_2 =	0.21		
	μ_3 =	0.21		
Sample 2, 28 days	E_1 =	25,927	25,417	MPa
	E_2 =	25,506		
	E_3 =	24,817		
	μ_1 =	0.18	0.19	-
	μ_2 =	0.19		
	μ_3 =	0.19		

Appendix A-4: Coefficient of thermal expansion of PCC calculations

Metallic frame calibration factor (C_f) using stainless steel cylinder:

C_f = correction factor (10^{-6}/°C), $C_f = (\Delta L_f/L_{cs})/\Delta T$

ΔL_f = length change of the aparatus (mm), $\Delta L_f = \Delta L_a - \Delta L_m$

ΔL_a = actual length change of the specimen (mm), $\Delta L_a = L_{cs} \times \alpha_c \times \Delta T$

L_{cs} = cylinder height, 207.5 mm

α_c = CTE of stainless steel, 17.3×10^{-6}/°C

ΔL_m = measured length change of the specimen (mm), $\Delta L_m = ABS(D_{fin} - D_{ini})$

D_{fin} = final deformation (mm)

D_{ini} = initial deformation (mm)

ΔT = measured temperature change (°C), $\Delta T = ABS(T_{fin} - T_{ini})$

T_{fin} = temperature corresponding to final deformation (°C)

T_{ini} = temperature corresponding to initial deformation (°C)

Concrete set of sample	**Sample No.**	**T_{ini} (°C)**	**T_{fin} (°C)**	**D_{ini} (mm)**	**D_{fin} (mm)**	**C_f (10^{-6}/°C)**	
1	1	9.0	49.2	54.85	-13.75	8.28E-06	8.21E-06
		49.2	9.2	-13.75	55.65	8.13E-06	
	2	9.2	49.4	55.65	-12.65	8.32E-06	8.17E-06
		49.4	9.3	-12.65	57.87	8.01E-06	
	3	9.3	49.3	57.87	-13.05	7.93E-06	8.08E-06
		49.3	9.1	-13.05	55.95	8.23E-06	
Average, (10^{-6}/°C)						**8.15E-06**	

CTE = coefficient of thermal expansion of concrete (10^{-6}/°C), CTE = $(\Delta L_a/L_{cs})/\Delta T$

ΔL_a = actual length change of the specimen (mm), $\Delta L_a = \Delta L_m + \Delta L_f$

ΔL_m = measured length change of the specimen (mm), $\Delta L_m = ABS(D_{fin} - D_{ini})$

D_{fin} = final deformation (mm)

D_{ini} = initial deformation (mm)

ΔL_f = length change of the aparatus (mm), $\Delta L_f = C_f \times L_{cs} \times \Delta T$

C_f = correction factor, 8.15×10^{-6}/°C

L_{cs} = cylinders heights, 201.24, 200.71, 203.40, and 202.60 mm

ΔT = measured temperature change (°C), $\Delta T = ABS(T_{fin} - T_{ini})$

T_{fin} = temperature corresponding to final deformation (°C)

T_{ini} = temperature corresponding to initial deformation (°C)

Set of sample	**Sample No.**	**T_{ini} (°C)**	**T_{fin} (°C)**	**D_{ini} (mm)**	**D_{fin} (mm)**	**CTE (10^{-6}/°C)**	
1	1	9.5	49.4	17.06	-16.97	1.280E-05	1.281E-05
		49.4	9.6	-16.97	17.17	1.282E-05	
	2	49.3	9.7	-18.74	16.03	1.295E-05	1.291E-05
		9.7	49.2	16.03	-18.11	1.287E-05	
Set 1 average, (10^{-6}/°C)						1.286E-05	
2	1	49.4	9.4	-4.63	25.02	1.215E-05	1.236E-05
		9.4	49.1	25.02	-7.52	1.257E-05	
	2	49.5	9.7	-44.81	-12.11	1.260E-05	1.269E-05
		9.7	49	-12.11	-45.71	1.278E-05	
Set 2 average, (10^{-6}/°C)						1.252E-05	
Average, (10^{-6}/°C)						**1.269E-05**	

Appendix A-5: Heat capacity of PCC calculations

Calorimeter calibration (crushed glass <25 mm):

$M_c = [(c_s \times M_s \times T + c_l \times M_0 \times T + c_b \times M_{ssb} \times T)/c_l \times T_l] - M_l$

c_s = glass heat capacity, 837.0 J/kg×°C

M_s = mass of the sample, 1.0004 kg

T = average temperature change of the sample, °C

c_l = water heat capacity, 4186.8 J/kg×°C

M_0 = mass of water carry-over, $M_0 = M_{ssbg} - M_{ssb} - M_s = 0.0525$ kg

M_{ssbg} = mass of drip basket & glass, 1.3196 kg

M_{ssb} = mass of stainless steel basket, 0.2667 kg

c_b = stainless steel heat capacity, 500.0 J/kg×°C

T_l = average temperature change of the water, °C

M_l = mass of water in the calorimeter, 1.0000 kg

Cold	$T_{isample}$	T_{iwater}	T_{final}	$T_l = T_{iwater} - T_{final}$	$T = T_{final} - T_{isample}$	M_{ccold}
1	1.9	21.8	17.9	4.0	16.0	0.1482
2	1.8	23.6	19.3	4.4	17.5	0.1406
3	1.8	22.4	18.4	4.0	16.6	0.1800
4	1.9	22.5	18.4	4.1	16.5	0.1443
5	2.0	22.6	18.5	4.1	16.5	0.1443
6	1.9	22.3	18.1	4.2	16.2	0.0934
7	1.8	22.2	18.2	4.1	16.4	0.1479
8	1.9	22.2	18.2	4.1	16.3	0.1409
Average Cold, (°C)				4.1	16.5	
Hot	$T_{isample}$	T_{iwater}	T_{final}	$T_l = T_{final} - T_{iwater}$	$T = T_{isample} - T_{final}$	M_{chot}
1	51.3	22.2	27.9	5.7	23.4	0.1673
2	51.3	22.2	27.7	5.6	23.6	0.2091
3	51.9	21.8	27.7	5.9	24.2	0.1663
4	51.7	21.0	26.8	5.8	24.9	0.2207
5	51.6	20.8	26.7	5.9	24.9	0.2000
6	51.7	21.0	27.0	6.0	24.8	0.1828
7	51.6	20.9	27.0	6.1	24.7	0.1585
8	51.5	21.0	26.9	5.9	24.6	0.1856
Average Hot, (°C)				5.8	24.4	
Average, (°C)				**5.0**	**20.4**	

Water Equivalent of the Calorimeter, M_c = 0.1679 kg

Heat capacity for set 1 of concrete samples:

$c_s = [(M_l + M_c)\times c_l \times T_l - (M_0 \times c_l + M_{ssb} \times c_b) \times T]/M_s \times T$

c_s = concrete heat capacity, J/kg×°C

M_l = mass of water in the calorimeter, 1.0000 kg

M_c = water Equivalent of the Calorimeter, 0.1679 kg

c_l = water heat capacity, 4186.8 J/kg×°C

T_l = average temperature change of the water, °C

M_0 = mass of water carry-over, $M_0 = M_{ssbg} - M_{ssb} - M_s = 0.0827$ kg

M_{ssbg} = mass of drip basket & glass, 1.3500 kg

M_{ssb} = mass of stainless steel basket, 0.2667 kg

c_b = stainless steel heat capacity, 500.0 J/kg×°C

T = average temperature change of the sample, °C

M_s = mass of the sample, 1.0006 kg

Sample 1: Heat capacity calculation

Cold	$T_{isample}$	T_{iwater}	T_{final}	$T_l = T_{iwater} - T_{final}$	$T = T_{final} - T_{isample}$	C_{scold}	
1	1.5	25.9	20.1	5.8	18.6	1044.59	1006.09
2	1.7	29.0	22.6	6.5	20.9	1032.49	
3	1.8	22.0	17.3	4.7	15.5	1002.55	
4	1.7	21.2	16.8	4.4	15.1	944.71	
Average Cold, (°C)				5.3	17.5		
Hot	$T_{isample}$	T_{iwater}	T_{final}	$T_l = T_{final} - T_{iwater}$	$T = T_{isample} - T_{final}$	C_{shot}	
1	51.8	20.9	27.7	6.8	24.1	899.59	887.14
2	52.0	23.2	29.5	6.3	22.5	889.05	
3	52.0	21.6	28.2	6.6	23.8	875.90	
4	51.5	21.7	28.2	6.5	23.3	884.01	
Average Hot, (°C)				6.6	23.4		
Average				**5.9**	**20.5**	**946.61**	

Sample 2: Heat capacity calculation

Cold	$T_{isample}$	T_{iwater}	T_{final}	$T_1 = T_{iwater}-T_{final}$	$T = T_{final}-T_{isample}$	C_{scold}	
5	1.7	21.6	16.4	5.2	14.7	1208.47	1049.66
6	1.7	21.7	17.0	4.7	15.3	981.04	
7	1.8	21.4	16.6	4.8	14.8	1064.75	
8	1.7	22.3	17.6	4.8	15.9	944.37	
Average Cold, (°C)				4.9	15.2		
Hot	$T_{isample}$	T_{iwater}	T_{final}	$T_1 = T_{final}-T_{iwater}$	$T = T_{isample}-T_{final}$	C_{shot}	
5	52.1	21.7	28.3	6.6	23.8	835.07	856.71
6	51.8	21.4	28.1	6.8	23.7	912.55	
7	52.1	21.5	28.2	6.7	23.9	849.84	
8	52.3	21.8	28.4	6.6	23.9	829.40	
Average Hot, (°C)				6.7	23.8		
Average				**5.8**	**19.5**	**953.19**	

Concrete Heat Capacity, c_s= 949.90 J/kg°C

Heat capacity for set 2 of concrete samples:

$c_s = [(M_l + M_c)\times c_l \times T_l - (M_0 \times c_l + M_{ssb} \times c_b)\times T]/M_s \times T$

c_s = concrete heat capacity, J/kg×°C

M_l = mass of water in the calorimeter, 1.0000 kg

M_c = water Equivalent of the Calorimeter, 0.1679 kg

c_l = water heat capacity, 4186.8 J/kg×°C

T_l = average temperature change of the water, °C

M_0 = mass of water carry-over, $M_0 = M_{ssbg} - M_{ssb} - M_s = 0.0924$ kg

M_{ssbg} = mass of drip basket & glass, 1.3600 kg

M_{ssb} = mass of stainless steel basket, 0.2667 kg

c_b = stainless steel heat capacity, 500.0 J/kg×°C

T = average temperature change of the sample, °C

M_s = mass of the sample, 1.0009 kg

Sample 1: Heat capacity calculation

Cold	$T_{isample}$	T_{iwater}	T_{final}	$T_1 = T_{iwater}-T_{final}$	$T = T_{final}-T_{isample}$	C_{scold}	
1	2.0	25.0	19.9	5.2	17.9	974.01	929.32
2	2.1	22.8	18.3	4.5	16.2	906.01	
3	2.2	19.8	16.0	3.8	13.8	909.31	
4	2.2	20.3	16.4	4.0	14.2	927.95	
Average Cold, (°C)				4.3	15.5		
Hot	$T_{isample}$	T_{iwater}	T_{final}	$T_1 = T_{final}-T_{iwater}$	$T = T_{isample}-T_{final}$	C_{shot}	
1	51.7	22.8	28.7	5.9	23.0	816.62	835.61
2	52.3	21.2	27.7	6.5	24.6	854.53	
3	51.7	21.9	28.0	6.1	23.7	820.86	
4	51.9	23.6	29.5	5.9	22.4	850.42	
Average Hot, (°C)				6.1	23.4		
Average				**5.2**	**19.5**	**882.46**	

Sample 2: Heat capacity calculation

Cold	$T_{isample}$	T_{iwater}	T_{final}	$T_1 = T_{iwater} - T_{final}$	$T = T_{final} - T_{isample}$	C_{scold}	
5	52.3	21.6	28.3	6.7	24.0	943.71	893.92
6	52.3	20.5	27.2	6.7	25.1	867.84	
7	52.1	22.0	28.4	6.4	23.7	898.96	
8	51.7	20.5	27.0	6.5	24.7	865.19	
Average Cold, (°C)				6.6	24.4		
Hot	$T_{isample}$	T_{iwater}	T_{final}	$T_1 = T_{final} - T_{iwater}$	$T = T_{isample} - T_{final}$	C_{shot}	
5	1.8	20.7	16.6	4.1	14.8	933.21	878.91
6	1.8	21.3	17.2	4.1	15.4	880.28	
7	1.8	21.4	17.4	4.0	15.6	832.10	
8	1.9	22.0	17.8	4.2	15.9	870.06	
Average Hot, (°C)				4.1	15.4		
Average				**5.3**	**19.9**	**886.42**	

Concrete Heat Capacity, c_s= 884.44 J/kg°C

Appendix A-6: Thermal diffusivity of PCC calculations

TD = thermal diffusivity of PCC, m^2/sec, TD = $(2.58064E\text{-}05)\times M/(T_2 - T_1)$

$M = 60\times\log(T_2/T_1)/(5.783/r^2 + \pi^2/l^2)$

2.58064E-05 = conversion factor from ft^2/hr to m^2/sec

M = factor depending on the size and shape of the sample, ft^2

T_2 (11) = time when the temperature difference is 11°C, hr

T_1 (44) = time when the temperature difference is 44°C, hr

r = concrete sample radius, ft

l = concrete sample length (height), ft

	Set 1				Set 2			
	Sample 1		Sample 2		Sample 1		Sample 2	
l	0.061	0.061	0.061	0.061	0.062	0.062	0.062	0.062
r	0.015	0.015	0.016	0.016	0.016	0.016	0.016	0.016
T_1(44)	0.05	0.05	0.05	0.05	0.07	0.06	0.06	0.06
T_2(11)	0.19	0.18	0.18	0.18	0.22	0.21	0.22	0.21
T_2-T_1 (hr)	0.13	0.13	0.13	0.13	0.15	0.15	0.15	0.15
M (ft^2)	0.00279	0.00292	0.00289	0.00290	0.00263	0.00284	0.00278	0.00285
TD (m^2/sec)	5.43E-07	5.74E-07	5.90E-07	5.70E-07	4.53E-07	4.92E-07	4.67E-07	4.82E-07
Average TD (m^2/sec)	**5.58E-07**		**5.80E-07**		**4.73E-07**		**4.74E-07**	
	5.69E-07				**4.74E-07**			
	5.21E-07							

Concrete Thermal Diffusivity = 5.21E-07 m^2/sec