Optimisation of the Crack Pattern in Continuously Reinforced Concrete Pavements

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Optimisation of the Crack Pattern in Continuously Reinforced Concrete Pavements

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Summary

Recent field investigations on several new Continuously Reinforced Concrete Pavements (CRCP) in Belgium indicate that its crack pattern is characterized by low mean crack spacing along with a high percentage of clusters of closely spaced cracks. Field surveys also indicate that it is difficult to significantly reduce the probability of a non-uniform crack pattern - such as closely spaced cracks, meandering, and Y-cracks - by only slightly adjusting the amount of longitudinal steel. Non-uniform crack patterns are inevitable and common in conventional CRCPs. Extensive distress data analyses of many CRCP sections in the United States have shown that the majority of punchouts develop at short spaced transverse cracks. Moreover, a non-uniform crack pattern and a high variability in transverse crack spacing were found to have a higher probability of punchout development. It is generally understood that the long-term performance of a CRCP is largely determined by it early age behaviour. Previous experiences have shown that the early entry method can eliminate the clusters of closely spaced cracks and a more regular crack pattern is achieved. In the present study a new early entry method, partial surface notch, is proposed to improve the crack pattern of CRCP. The primary objective of this study is to optimize the crack spacing pattern of CRCP through an active crack control method.

To realize the research objective, predicting the pavement temperature at early age is a good starting point to understand the early age behaviour of CRCP. This study thus firstly provides a procedure to predict the early age temperature development of a concrete pavement based on concrete mixture composition, the thermal characteristics of the concrete and the underneath pavement layers, the environmental conditions, and the construction time and curing methods. Available heat flux models for the pavement surface are initially reviewed and adjustments to improve the accuracy of the predicted early age concrete pavement temperature are suggested. The proposed model enables to simulate the use of blended slag cement and the plastic sheet curing for the Belgium CRCP practice. This temperature model is verified with field measured data of two projects in Belgium, and the result is quite satisfactory. Lastly, an approach is proposed to generate reliable and real-time climatic inputs by using limited weather forecasting climate data for the temperature prediction model during the construction phase. This allows the contractor to optimize construction operations, especially the time of saw cutting.

Because notches are made at early age to induce transverse cracks at the designated locations, in addition to the mechanical properties of tensile strength and elastic modulus, the evolution of the fracture energy has to be known to evaluate the cracking tendency of the notched concrete pavement. A deformation-controlled
uniaxial tensile test on unnotched specimens is performed for the typical CRCP concrete mixture used in Belgium. Experimental results show that the applied unnotched parabolic shape concrete specimens, the used tension set-up with three hinges, and the applied test procedures succeed in obtaining the complete softening curves for the Belgium CRCP concrete mixtures ranging from 24 hours to 90 days. In order to correlate the concrete properties in field and laboratory conditions for accurately predicting the cracking in a concrete pavement, degree of hydration based descriptions of the early age concrete properties are given based on the experimental results of the tension tests.

Using the proposed temperature prediction model and measured early age concrete properties, the concrete stress history at early age is calculated by the superposition principle (through a step-by step numerical method). The time dependent relaxation of the early age concrete, which was described as a function of the degree of hydration is considered as well. The zero stress temperature, peak pavement temperature, built-in temperature gradient, and the cracking time are determined by the estimated early age temperature and stress development. Extensive parametric simulations have shown that the early age concrete temperature and stress development are closely related with various environmental and construction conditions, such as time on the day of concrete placement, construction season, plastic sheet curing, concrete placement temperature etc. A fracture mechanics based procedure is developed to calculate the saw cut depth and saw cut timing for the active crack control method. The estimated final set time gives the lower limit for the saw cutting operation to avoid ravelling while the predicted cracking time indicates the upper limit of the saw cutting window before initiation of randomly occurring natural cracks. Theoretical analyses demonstrate that the applied saw cut depth and saw cut length is appropriate for the current CRCP conditions in Belgium.

Extensive field investigations were conducted on two recently constructed CRCP sections in Belgium to evaluate the effect of longitudinal reinforcement percentage and active crack control methods on the crack pattern of CRCP. The crack pattern development, crack width, and crack width movement due to daily temperature variation, were regularly investigated. Field evidences have shown that the proposed active crack control method is very effective in inducing cracks. Moreover, the transverse cracks in the active crack control sections are much straighter and more regular spaced. The active crack control method significantly reduces the percentage of short spaced cracks and cluster cracks and thus reduces the risk of punchout development in the long-term of CRCP.
Samenvatting

Recente metingen aan verschillende nieuwe Doorgaand Gewapende Betonverhardingen (DGB) in België geven aan dat hun scheurpatroon wordt gekarakteriseerd door een kleine gemiddelde scheurafstand en een groot percentage clusters van scheuren op korte onderlinge afstand. De onderzoeken geven ook aan dat de kans op een onregelmatig scheurpatroon – zoals scheuren op korte afstand, kronkellende en Y-scheuren – moeilijk significant is te reduceren door slechts de hoeveelheid langswapening in geringe mate te wijzigen. Onregelmatige scheurpatronen zijn onvermijdelijk in conventionele DGBs. Uitgebreide analyses van schadegegevens van veel DGB secties in de Verenigde Staten hebben aangetoond dat de meeste punchouts optreden tussen scheuren op korte afstand. Bovendien bleken een onregelmatig scheurpatroon en grote variatie in scheurafstand een grotere kans op punchouts te geven. Het is algemeen geaccepteerd dat het lange termijn gedrag van een DGB in hoge mate wordt bepaald door het gedrag van de jonge verharding. Eerdere ervaringen hebben aangetoond dat het vroeg aanbrengen van dwarse zaagsneden de clusters van scheuren kan elimineren waardoor een regelmatiger scheurpatroon ontstaat. In deze studie wordt een nieuwe methode voorgesteld om het scheurpatroon van een DGB te verbeteren, nl. de korte dwarse zaagsnede als scheurinleider. Het belangrijkste doel van deze studie is om het scheurpatroon van een DGB te optimaliseren door middel van deze zgn. actieve methode.

Voorspelling van de temperatuur is een goed startpunt om het gedrag van de jonge DGB te begrijpen en daarmee de doelstelling van het onderzoek te realiseren. Daarom is eerst een procedure ontwikkeld om de temperatuur in de jonge verharding te voorspellen op basis van de betonsamenstelling, de thermische eigenschappen van het beton en de onderliggende lagen, de omgevingscondities, het tijdstip van aanleg en de curing methode. De beschikbare warmtestroom modellen voor het verhardingsoppervlak zijn geëvalueerd en veranderingen zijn voorgesteld om de temperatuur in de jonge verharding nauwkeuriger te voorspellen. Met het voorgestelde model kan de toepassing van hoogovencement en plastic folie voor curing worden gesimuleerd voor de Belgische DGB praktijk. Dit temperatuurmodel is geverifieerd door metingen aan twee projecten in België en de overeenstemming is zeer goed. Tenslotte is een methodiek voorgesteld om betrouwbare en realtime klimaatgegevens voor het temperatuurmodel te genereren met beperkte, algemeen beschikbare weersvoorspellingsgegevens voor de periode van uitvoering van de betonverharding. Dit stelt de aannemer in staat om het uitvoeringsproces te optimaliseren, met name het tijdstip van aanbrengen van de zaagsneden.

Omdat de korte dwarse zaagsneden kort na het aanbrengen van beton worden gemaakt om dwarsscheuren te krijgen op de gewenste locaties, moet naast de
ontwikkeling van de treksterkte en elasticiteitsmodulus ook die van de breukenergie bekend zijn om de scheurgevoeligheid van de betonverharding met deze scheur-inleiders te evalueren. Hiertoe zijn verplaatsingsgestuurd uniaxiale trekproeven op proefstukken zonder zaagsnede van typische Belgische DGB betonmengsels uitgevoerd. De resultaten laten zien dat de combinatie van de toegepaste parabolische proefstukken, trekopstelling met drie scharnieren en testprocedure de complete 'softening curve' voor de Belgische DGB betonmengsels, met een ouderdom variërende van 24 uur tot 90 dagen, heeft opgeleverd. Op basis van de trekproefresultaten zijn de eigenschappen van het jonge beton beschreven als functie van de hydratatiegraad om de eigenschappen van het beton in de praktijk en onder laboratoriumcondities te kunnen correleren.

Op basis van het voorgestelde temperatuurvoorspellingsmodel en de gemeten eigenschappen van het jonge beton wordt de ontwikkeling van de betonspanningen berekend volgens het superpositie beginstel (met een stap-voor-stap numerieke methode). Hierbij is de tijdsafhankelijke relaxatie van het jonge beton, beschreven als functie van de hydratatiegraad, ook in beschouwing genomen. Met de berekende ontwikkeling van de temperatuur en spanning in het jonge beton zijn de spanningsvrije temperatuur, de maximale temperatuur, de ingebouwde temperatuurgradiënt en het tijdstip van scheuren bepaald. Uitgebreide parametrische simulaties tonen dat de temperatuur en spanning in het jonge beton sterk gerelateerd zijn aan verschillende omgevings- en uitvoeringscondities, zoals het tijdstip op de dag en het seizoen van aanleg, de curing met plastic folie, de temperatuur van het beton bij aanleg, etc. Op basis van de breukmechanica is een procedure ontwikkeld om de diepte van de korte dwarse zaagsnede en het tijdstip van zagen voor de actieve methode te berekenen. Het geschatte einde van de groene fase geeft de ondergrens voor het zagen, om rafeling te voorkomen, terwijl het berekende tijdstip van scheuren de bovengrens vormt, om random optredende natuurlijke scheuren te voorkomen. Theoretische analyses laten zien dat de toegepaste diepte en lengte van de korte zaagsneden geschikt zijn voor de huidige DGB condities in België.

Uitgebreid veldonderzoek is uitgevoerd aan twee recent aangelegde DGB secties in België om de invloed van het percentage langswapening en de actieve methode op het scheurpatroon te evalueren. De ontwikkeling van het scheurpatroon, de scheurwijdte alsmede de verandering van de scheurwijdte als gevolg van de dagelijkse temperatuurvariatie zijn regelmatig gemeten. Uit het onderzoek is gebleken dat de voorgestelde actieve methode zeer effectief is met betrekking tot het inleiden van scheuren. Bovendien zijn de dwarsscheuren dan veel rechter en is het scheurpatroon veel regelmatiger. De actieve methode verkleint significant het percentage korte scheurafstanden en clusters van scheuren en daarmee het risico van punchouts in DGBs op de lange termijn.
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Introduction

Continuously Reinforced Concrete Pavement (CRCP) contains continuous longitudinal steel reinforcement. It does not have transverse joints except where necessary for construction purposes. It allows transverse cracks to occur in a random crack pattern, mainly caused by volumetric changes in the concrete that are a result of temperature and moisture variations. Those cracks are held tightly closed by the longitudinal reinforcement. CRCP can be an excellent pavement solution for heavily loaded roads, and is expected to provide long life performance with minimal maintenance (Hall et al. 2007; Rasmussen et al. 2009). Extensive distress data analyses of many CRCP sections in the United States have shown that the majority of punchouts develop at short spaced transverse cracks. Moreover, a non-uniform crack pattern and a high variability in transverse crack spacing were found to have a higher probability of punchout development. It is generally understood that the long-term performance of a CRCP is largely determined by its early age behaviour. Therefore, the primary objective of this study is to optimize the crack spacing pattern of CRCP through an active crack control method.

1.1 BACKGROUND

The Importance of Crack Pattern on CRCP Performance

In Belgium, continuously reinforced concrete pavements are being used on a large scale for more than forty years. The sustainability, e.g. durability and low maintenance of CRCP, lead to long lasting applications in Belgium (Verhoeven 1993). However, the crack pattern still shows a cluster formation as shown in Figure 1.1(a), which may lead to the development of punchouts (localized slab portion broken into several pieces), reducing the potential durability. According to field observations of several newly constructed CRCPs under the current design concept in Belgium, the crack pattern is characterized as low mean crack spacing, with cracks approximately 1.0 m apart after 2 years in service, along with a high
percentage of clusters of closely spaced cracks (Verhoeven 1993; Feys 2010; Rens 2010; Rens and Beeldens 2010; van Avermaet and van Weyenberge 2011; Ren et al. 2013). The analysis of an extensive field and laboratory testing of 23 in-service CRCP roads in United States has shown that the majority of punchouts occur when transverse cracks are spaced from about 0.3 to 0.6 m, as indicated in Figure 1.1(b), and especially in clusters of closely spaced cracks (Selezneva et al. 2003). In Belgium, CRCP under the current standard design concept behaves excellent and is barely subjected to deterioration, mainly because of the good base support condition (Rens 2010). However, occasionally punchouts do occur (as shown in Figure 1.1(c)) and therefore more research is needed to investigate how to obtain a more regular crack pattern in CRCP.
Figure 1.1 The distress of punchout in CRCP, (a) cluster of closely spaced cracks and Y-cracks observed on E17, De Pinte, 2011; (b) frequency of punchouts as a function of crack spacing according to LTPP database, after Selezneva et al. (2003); (c) punchout observed on a motorway in Belgium, after Fuchs and Jasienshi (1997).

**History of Improving CRCP Crack Pattern**

The first attempt to improve the crack pattern, especially reducing the occurrence of clusters of closely spaced cracks, may be achieved by optimizing the design or construction variables. In fact, the standard CRCP design and construction in Belgium underwent several changes over time with regard to longitudinal reinforcement percentage, position of the rebar, presence of an asphalt interlayer, pavement thickness, concrete mix, surface finishing, and lane width (Ren et al. 2013). Field findings have indicated that these attempts can let the average crack spacing and mean crack width fall into a favourable range, but it is difficult to significantly reduce the probability of a non-uniform crack pattern, such as “Y” shaped cracks and closely spaced transverse cracks, by adjusting the amount of longitudinal steel, primarily because of the variability of material properties, construction factors, and environmental conditions that are to some extent outside the contractor’s control (Rasmussen et al. 2009).

An alternative solution is active crack control. Actually, it is not a new idea. Active crack control or induced cracking is being used extensively for concrete pavements, mainly in jointed plain concrete pavements (JPCP) and jointed reinforced concrete pavements. Zollinger et al. (1998); McCullough and Dossey (1999); Kohler and Roesler (2004) adopted the idea of active crack control for CRCP. Their results of full-scale field test sections have revealed that the active crack control technique achieved transverse cracks occurring sooner, straighter, and at the intended regular interval relative to the passive crack control. Therefore, it can
significantly reduce the probability of a non-uniform crack pattern and eventually prevent punchout development. However, there are still some limitations existing in the active crack control method that is applied in US. Firstly, the tape insertion method poses a difficulty during construction. Secondly, the presence of a transverse saw cut or a crack initiation through the whole width of the concrete slab may not only reduce the aggregate interlock and eventually decrease the load transfer efficiency, which will reduce the life of the pavement, but also cause some surface defects, like spalling which could decrease the riding quality.

In 2012, Rens proposed a modified active crack control method in Belgium attempting to achieve a better crack pattern, especially with the aim to reduce the number of closely spaced cracks (Rens and Beeldens 2013). A partial surface saw cut was applied in the reconstruction project of motorway E313 near the city of Herentals, Belgium, as shown in Figure 1.2. However, it should be noted that the adopted saw cut timing and the selected saw cut size in the test section E313 were actually based on the operator’s experience. The determination of the appropriate timing and depth at which the saw cut should be made is still problematic in practice, and it should be assessed from a theoretical point of view. Several attempts have been made to predict early age stress development in Portland cement concrete pavement (4C-Temp&Stress 1998; TMAC2 2007; HIPERPAV 2009). However, many of those do not consider saw cutting. The software package HIPERPAV is the most advanced and practical model for the early age concrete pavement behavior (Schindler et al. 2002). HIPERPAV recently added a feature to address the timing of saw cutting. However, there are several limitations when it is used for the Belgium conditions. For instance, the cement hydration model in HIPERPAV was developed according to the cement that is commonly used in United States, and thus the use of this model for the blended blast furnace slag cement that is applied in Belgium should be validated. Besides, limitations also exist for the heat flux models for both the upper and bottom boundary conditions of the pavement structure, such as the lack of a model to quantify the effect of the plastic sheet curing method, and refinements are needed for the adjustment of environmental data, etc. Several methodologies have been developed to evaluate the saw cutting requirements for concrete pavement (Okamoto et al. 1994; Zollinger et al. 1994; Gaedicke et al. 2007; Raoufi et al. 2008). However, none of those models covers the notch shape used in the E313. Therefore, adjustments are required for the application of those available early age concrete pavement behaviour prediction models and saw cut models for the Belgium CRCP conditions.
1.2 OBJECTIVES

The primary objective of this study is to provide a tool that will help to achieve a uniform crack spacing pattern in CRCP through an active crack control method. In order to achieve the primary objective, the following secondary objectives are required during this study:

1) Develop a prediction model to quantify the early age temperature development of concrete pavements under Belgium field and construction conditions for CRCP. It should account for the plastic sheet curing method and cover the blend blast furnace slag cement.

2) Develop models, based on experimental results with respect to the degree of hydration, for the early age concrete Young’s modulus, uniaxial tensile strength, and fracture energy for the evaluation of the risk of thermal cracking in the early age of concrete pavement and the subsequent determination of the saw cut timing.

3) Quantify the early age stress development as a function of the ambient conditions and construction practices of typical CRCP roads in Belgium accounting for the effect of concrete relaxation. Develop a saw cut model to determine the timing and geometry of the partial surface notches for CRCP based on fracture mechanics.
4) Evaluate the effectiveness of the active crack control method on the early age CRCP behaviour as compared to field survey results of conventional CRCP sections.

1.3 SCOPE AND ORGANIZATION OF THE THESIS

In total, this dissertation is composed of eight chapters. After this introductory Chapter, Chapter 2 presents the literature review, which serves to provide the necessary background for the present study. Firstly, an overview of the history of CRCP in Belgium and the Netherlands is presented. The relationship between the early age crack pattern of CRCP and its long-term performance is identified, with special focus on the distress of punchout that is generally regarded as the most severe distress type in CRCP. The factors influencing the early age crack pattern of CRCP are reviewed. More especially, the possible causes of the clusters of closely spaced cracks are concluded from the factors concerning the design, construction, and material properties of CRCP. This section further provides a summary of current active crack control methods for CRCP. The experiences from the previous active control CRCP sections are summarized.

Chapter 3 provides detailed information of the field investigations on two recently constructed CRCP sections in Belgium, E17 at De Pinte and E313 at Herentals. The intention of the De Pinte test section on E17 is to evaluate the effect of the longitudinal reinforcement percentage on the transverse crack pattern of CRCP. The test section in Herentals on E313 is used to identify the effect of an active crack control method on the transverse crack pattern. The test section sites, material properties, special features of construction, field instrumentations and survey schemes are presented in detail. In order to obtain the required parameters for the proposed theoretical analysis procedure of the active crack control method, several experimental programs are further addressed.

Chapter 4 describes in detail the development of an early age concrete pavement temperature prediction model. It firstly describes the selected hydration model for blended blast furnace slag cement, and the procedure to obtain the relevant parameters of this model through an isothermal calorimetry conduction test. Critical reviews of the heat flux models for the pavement surface are presented. Those models are evaluated and reasons for the selection of the recommended models are provided. Besides, adjustments are suggested when necessary, for instance, an extension of an existing model to quantify the effect of the plastic sheet curing method is introduced. The numerical implementation procedure for the proposed temperature prediction model is solved by the finite difference method. The Matlab code for this simulation is included in the Appendix I. This proposed temperature prediction model is verified with the field-measured data of two projects in Belgium. The accuracy of the temperature
prediction model is then assessed. Lastly, a prediction model for concrete pavement temperatures during construction using limited weather forecasting data is presented.

Chapter 5 is dedicated to deformation controlled uniaxial tensile tests to obtain the concrete fracture energy, modulus of elasticity and uniaxial tensile strength at early age for the concrete mixtures used on E17 and E313. Unnotched specimens with a parabolic shape are tested in a tension set-up developed at TU Delft. Unlike the notched specimens for conventional concrete fracture tests, the gradual change in the specimen shape used in the present study does not lead to extreme stress concentrations that obscure the actual tensile strength. The most significant feature of the TU Delft tension test set-up is that it is built with three hinges to accommodate the alignment of the specimens. The specimen preparation, test conditions, and the TU Delft tension test set-up are explained in detail. In the last part of this chapter, in order to correlate the concrete properties in field and laboratory conditions for accurately predicting the cracking in concrete pavements, degree of hydrations based descriptions of the early age concrete properties are given based on experimental results.

Chapter 6 addresses the early entry partial surface saw cut method as applied in the test section on E313 with the purpose of improving the crack spacing distribution and thus reducing the risk of punchout development. Based on the proposed temperature model and measured early age concrete properties in previous chapters, a procedure to calculate the thermal stress development for the CRCP slab at the early age is developed. It includes a degree of hydration based relaxation model. The saw cutting window is established by the estimation of the concrete final set time (the earliest allowable time) and the potential thermal cracking time (the latest time to prevent random crack initiation) through an analysis of the early age concrete stresses. Extensive parametric simulations are conducted in terms of various environmental condition and curing methods under the Belgium CRCP conditions. A saw cut depth model proposed by Zollinger is modified to determine the geometry of the partial surface notch used in Belgian CRCP projects.

Chapter 7 characterizes the CRCP behaviour in terms of the measured crack spacing and crack width for newly constructed CRCP roads according to the new standard design concept in Belgium. The effectiveness of the active crack control method for CRCP through partial surface notches is assessed based on extensive field condition surveys.

Finally, Chapter 8 summarizes observations and findings of this study, and recommendations are presented here as well.
2.1 CRCP IN THE NETHERLANDS AND BELGIUM

2.1.1 Belgium CRCP History

Belgium has a long history of concrete road construction. The Avenue de Lorraine in Brussels, built in 1925 with just a 150 mm thick concrete slab and directly placed on the subgrade, remained in service until 2003 when it received a concrete overlay. It provided good service for 78 years and without any significant maintenance costs ever. The Avenue de Lorraine is not unique. Brussels has many examples of concrete roads that have served traffic for 50 years or more (Gilles and Jasienski 2006). Concrete pavements make up 17% of all roads in Belgium. The Belgian motorway network comprises in total 1700 km and 40% of this network has a concrete pavement and most of them are CRCP (Hall et al. 2007). The Belgian Road Authorities favours CRCP due to its low maintenance requirements and now it is an accepted form of pavement structure for heavily trafficked roads in Belgium.

In Belgium, CRCP is being used on large scale for more than forty years. Actually, Belgium’s CRCP design was adapted from the United States. CRCP was first applied in 1950s on several experimental sites in Belgium. However, the technique did not definitely break through until the late sixties. The first large-scale construction was started in 1970. Initially, the Belgium standard CRCP structure between 1970 and 1977 consisted of a 0.85% longitudinal reinforcement placed at a depth of 60 mm and a 200 mm thick concrete slab. A 60 mm asphalt interlayer was placed between the lean concrete base and the concrete slab. Extremely low average crack spacing for this design concept 1 was found which was not expected under the recommended design practice at that moment (Verhoeven 1993).
Due to economic reasons, the Belgium design concept 2 for CRCP was applied between 1981 and 1991. The reinforcement percentage was reduced to 0.67%. The depth of the reinforcement was changed from 60 to 90 mm while the concrete slab and lean concrete base thicknesses remained 200 mm but the bituminous interlayer was eliminated. A large research in 1992 revealed that the distribution of cracks with concept 2 was much more regular than with concept 1. The average crack spacing ranged from 1.4 to 2.4 m and nearly 70% of the crack spacing was within the range 0.8 to 3.0 m (Verhoeven 1993). However, it was found that CRCP constructed under concept 1, with an apparently unfavourable crack pattern and many clusters of closely spaced cracks, still behaved perfectly. By contrast, CRCP constructed under concept 2 rapidly exhibited punchout problems due to erosion of the base layer (Rens 2010).

Because of the failure of concept 2 CRCPs and in view of the increasing traffic loads, the standard structure design concept 3 of CRCP for main roads in Belgium was adapted in 1990s. The 60 mm asphalt interlayer was reintroduced and the slab thickness was increased to 230 mm and later to 250 mm. The reinforcement percentage was increased to 0.75% and the depth of the steel reinforcement was changed from 90 to 80 mm (Rens 2010).

As compared to the widely used pavement concrete in other countries, high strength concrete is commonly used in the Belgian pavement construction. The mean compressive strength at 28 days is more than 50 MPa (Tayabji et al. 1998a; Gilles and Jasienski 2006; Hall et al. 2007). The cement used is either Portland cement or a blast furnace slag cement of strength class 42.5, always with a low alkali content. Another feature is that exposed aggregate surfaces are now used on all motorways made of concrete in Belgium.

Verhoeven (1993) investigated the crack pattern and corrosion of the longitudinal steel reinforcement bars for various types of structures used in Belgium to that date. A total of 20 stretches of roads were selected, 4 from the oldest experimental phase of 1966 and 1967, 5 from the period 1971 to 1973 constructed according to design concept 1, and 11 stretches built between 1977 and 1985 according to design concept 2. Table 2.1 shows the summary of the crack pattern results for some of the concept 1 and concept 2 CRCPs. The crack pattern of design concept 2 consisted of much less closely spaced cracks but more widely spaced cracks than the crack pattern of design concept 1. As mentioned above, it was found that CRCP constructed under concept 1 with an apparently unfavourable crack pattern and many clusters of closely spaced cracks, still behaves perfectly after 40 years of service. By contrast, CRCP constructed under concept 2 rapidly exhibited punchout problems due to erosion of the base layer.
Chapter 2 Literature Review

Table 2.1 Characteristics of crack spacing of design concepts 1 and 2 (adapted from K. Verhoeven, 1993)

<table>
<thead>
<tr>
<th>Design concept</th>
<th>Road Section</th>
<th>Age(^2) (year)</th>
<th>Average crack spacing (m)</th>
<th>Thickness</th>
<th>Percentage of longitudinal reinforcement (%)</th>
<th>Crack spacing spectrum (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Bertem A</td>
<td>21</td>
<td>0.57</td>
<td>207</td>
<td>0.82</td>
<td>32.6 32.1 35.3 NA</td>
</tr>
<tr>
<td></td>
<td>Turnhout A</td>
<td>20</td>
<td>0.39</td>
<td>221</td>
<td>0.77</td>
<td>64.2 30.1 5.7 NA</td>
</tr>
<tr>
<td></td>
<td>Retie</td>
<td>19</td>
<td>0.61</td>
<td>225</td>
<td>0.75</td>
<td>27.4 33.5 39.1 NA</td>
</tr>
<tr>
<td>2</td>
<td>Pecq</td>
<td>15</td>
<td>1.17</td>
<td>222</td>
<td>0.60</td>
<td>5.2 13.1 78.4 3.3</td>
</tr>
<tr>
<td></td>
<td>Lamain</td>
<td>12</td>
<td>1.35</td>
<td>232</td>
<td>0.58</td>
<td>5.2 12.4 66.3 16.1</td>
</tr>
<tr>
<td></td>
<td>Recht</td>
<td>10</td>
<td>1.36</td>
<td>211</td>
<td>0.64</td>
<td>3.5 9.4 85.1 2.0</td>
</tr>
<tr>
<td></td>
<td>Vaux</td>
<td>10</td>
<td>1.86</td>
<td>211</td>
<td>0.64</td>
<td>3.0 3.9 67.8 25.3</td>
</tr>
</tbody>
</table>

Note: 1. Each section is 200 m long.
2. Age is from the time of construction to the time of survey.

Feys (2010) conducted a similar investigation on motorway E40 at Affigem, which was built under the current design concept 3. It was concluded that the crack pattern of design concept 3 was characterized as several groups of cracks closely spaced together with in between a large crack spacing that was similar to that of design concept 1. It was also noted that no conclusion could be made for the long-term durability of the pavement due to the early age of the pavement. Feys proposed that a possible solution was to decrease the longitudinal reinforcement percentage so that a more regular crack pattern will be formed.

Table 2.2 Characteristics of crack spacing of current design concept 3 in Belgium.

<table>
<thead>
<tr>
<th>Road</th>
<th>Age (day)</th>
<th>Length of test Sections (m)</th>
<th>Average crack spacing (m)</th>
<th>Percentage of longitudinal reinforcement (%)</th>
<th>Crack spacing spectrum (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E40-Affligem</td>
<td>184</td>
<td>600</td>
<td>1.69</td>
<td>0.75</td>
<td>11.1 38.3 17.5 9.0 24.1</td>
</tr>
<tr>
<td>E17-Kruishoutem</td>
<td>158</td>
<td>500</td>
<td>1.36</td>
<td>0.75</td>
<td>15.1 24.6 16.5 25.7 18.1</td>
</tr>
<tr>
<td>E17-Kruishoutem</td>
<td>158</td>
<td>500</td>
<td>1.44</td>
<td>0.70</td>
<td>13.9 28.1 17.7 17.7 22.6</td>
</tr>
<tr>
<td>E17-Kruishoutem</td>
<td>158</td>
<td>500</td>
<td>2.08</td>
<td>0.65</td>
<td>15.9 14.2 7.7 18.5 43.7</td>
</tr>
</tbody>
</table>

According to the findings by Feys, three percentages of longitudinal reinforcement, 0.75%, 0.70%, and 0.65%, respectively, were applied in the rehabilitation project of E17 at Kruishoutem in August 2010. van Avermaet and van Weyenberge (2011) periodically investigated the crack pattern of this project. Based on the results of the monitoring and an analysis by means of the MEPDG (ARA Inc. 2004) they concluded that it would be acceptable to reduce the steel content from 0.75 to 0.70%. In addition, it was proposed to add steel fibers to CRCP with reduced traditional reinforcement that could be a solution to keep cracks tightly closed. Table 2.3 shows a summary of design concepts and performance of CRCP in Belgium.
## Table 2.3 Summary of design concepts and performance of CRCP in Belgium

<table>
<thead>
<tr>
<th>Design concept</th>
<th>Concept 1</th>
<th>Concept 2</th>
<th>Concept 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal reinforcement</td>
<td>0.85%, Φ18@150</td>
<td>0.67%, Φ16@150</td>
<td>0.75%, Φ20@150</td>
</tr>
<tr>
<td>Concrete cover depth</td>
<td>60 mm</td>
<td>90 mm</td>
<td>80 mm</td>
</tr>
<tr>
<td>Slab thickness</td>
<td>200 mm</td>
<td>200 mm</td>
<td>230 mm</td>
</tr>
<tr>
<td>Interlayer</td>
<td>asphalt interlayer</td>
<td>no asphalt interlayer, directly on lean concrete</td>
<td>asphalt interlayer</td>
</tr>
<tr>
<td>Air-entraining agent</td>
<td>no</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>transversely grooved</td>
<td>transversely grooved or exposed aggregate</td>
<td>exposed aggregate</td>
</tr>
<tr>
<td>Performance</td>
<td>average crack spacing: 0.4 to 0.6 m about 18 years after construction. Many clusters of closely spaced cracks</td>
<td>average crack spacing: 1.4 to 2.4 m about 18 years after construction. About 70% of the crack spacings 0.8~3 m</td>
<td>average crack spacing: about 1.0 m after 2 years with clusters of closely spaced cracks (few data available)</td>
</tr>
</tbody>
</table>

## 2.1.2 The Netherlands CRCP History

The roadway network of the Netherlands consists of 139000 km, and 2300 km of them are motorways. Five percent of the motorway mileage in the Netherlands is concrete pavement, about half is CRCP and the other half is Jointed Plain Concrete Pavement (JPCP). Overall, concrete pavements make up about 4% of all the roads in the Netherlands. In addition to roadways for motorized traffic, the Netherlands also has about 20000 km of bicycle paths, 10% of which have a concrete pavement (Hall et al. 2007).

The Netherlands currently uses a mechanistic-design software package called VENCON for concrete pavement design (Houben 2006). Typical cross sections and other details for pavements for different types of road are available in the Dutch Cement Concrete Pavement Manual-Basic Structures (CROW 2005). The Delft tension bar model developed in Delft University of Technology is used to determine the required longitudinal reinforcement content for CRCP (van Breugel et al. 1998). In recent years, almost all new concrete pavements on motorways have been built as CRCP. The Netherlands is the most densely populated country in Europe. Noise is a big concern in the design and construction of roads. Concrete pavement generally produces more noise than asphalt concrete. The motorway agency is thus using porous asphalt concrete surfacing on concrete pavement. Concrete in the C35/45 strength class is used for concrete paving in the Netherlands (Hall et al. 2007).

Recent field surveys on several CRCPs with a porous asphalt surfacing in the Netherlands have shown that over 10 years after construction the CRCP is in good condition (Ren and Houben 2014; Ren and Houben 2015). No punchouts are observed. However, reflective cracks are observed at some sections and they are
believed to be attributed to the crack pattern of the underneath CRCP slab as shown in Figure 2.1.

Figure 2.1 Cluster of closely spaced cracks in CRCP slab and reflective crack in Porous Asphalt Overlay on A5 during the field investigation on 28/02/2015.

2.2 PUNCHOUT AND EARLY AGE CRACK PATTERN

2.2.1 Punchout

Punchout is recognized as the major structural distress in CRCP. National Cooperative Highway Research Program (NCHRP) Project 1-37A provides the following description for punchout: Punchout develops between two closely spaced transverse cracks because of crack load transfer efficiency loss and a longitudinal fatigue crack that defines the punchout segment along the pavement edge (ARA Inc. 2003a), as illustrated in Figure 2.2.

Figure 2.2 A typical punchout in CRCP, after ARA (2003).

The isolated small piece of concrete slab settles down into voids created by erosion due to repetitive traffic loads. A fully formed punchout results in the loss
of ride quality and presents serious hazards that could lead to fatal road accidents (Selezneva 2002). Considering the difficulty of full depth repair for concrete pavement, controlling the development of punchout is the focus of the CRCP design. The mechanism of punchout development summarized by Selezneva (2002) is schematically shown in Figure 2.3. The general stages of pavement deterioration leading to punchout are indicated as ① through ⑤. Among those, the formation of specific longitudinal cracks between two closely spaced cracks was commonly considered as the indication of punchout development.

![Figure 2.3 Mechanism of punchout development, after Selezneva (2002).](image)

LaCourseiere et al. (1978) evaluated the mechanism of edge punchout based on field investigations and numerical simulations in Illinois. As can be seen in Figure 2.4(a), the critical loading condition was believed to be a truck wheel near the slab edge. The concrete stress was calculated by ILLI-SLAB, a finite element computer program for concrete slabs on Winkler foundation that was developed at the University of Illinois. LaCourseiere and Darter observed that the maximum stress becomes very large as the crack spacing shortens and load transfer is lost, as indicated in Figure 2.4(b). It also indicates that the short crack spacing in CRCP may not be a problem if the load transfer is maintained. LaCourseiere and Darter observed that the crack spacing has a significant effect on the magnitude of the critical tensile lateral stress at the top of the slab. The maximum tensile stress at the top of the slab due to a 40 kN wheel load increases from 0.7 MPa at a crack spacing of 2 m to 5.1 MPa at a crack spacing of 0.3 m. The maximum tensile stress occurs 0.9 m from the slab edge as can be seen in Figure 2.4(c). It agrees well with the results of extensive field surveys that about 90% of all punchouts occur on segments bound by a pair of transverse cracks spaced at 0.6 m or less (Tayabji et al. 1999; Selezneva et al. 2003). Besides, the locations of the short longitudinal cracks of those punchouts are mostly located from 0.6 m to 1.4 m from the slab edge as
shown in Figure 2.5. This location corresponds to about half of the truck axle length.

Figure 2.4 Maximum tensile stress at the top of concrete slab in CRCP under traffic loading, (a) loading type used for analysis; (b) the effect of load transfer; (c) the effect of crack spacing with zero load transfer across cracks. (1 in=25 mm, 1 ft=0.3 m, 1 psi=6.895 kPa), after LaCourseiere and Darter (1978).
Data analysis has shown that no correlation exists between the mean crack spacing and the size of the segment that develops a punchout. Regardless of the mean crack spacing, punchouts always develop on narrow CRCP segments (Selezneva et al. 2003). Selezneva et al. further concluded that punchout prediction cannot be based on mean crack spacing alone but rather should take into account the fact that punchouts are likely to develop on individual narrow concrete segments. High transverse crack spacing variability results in a greater probability that narrow CRCP panels will be positioned next to wide CRCP panels. This situation may lead to wider transverse crack openings and accelerated deterioration of load transfer across the transverse cracks surrounding the narrow crack segments, leading to accelerated punchout development on the isolated segments (Kohler and Roesler 2004). A large variability in the crack spacing could also be a sign of poor subgrade and concrete uniformity and poor overall quality control in construction (McGovern et al. 1996). Zollinger (1989) reported that punchouts were invariably accompanied by severe base erosion and loss of support. Some other types of punchout in CRCP were also reported to be caused by the undesirable patterns of Y-cracking (Ley et al. 2012), meandering cracking (McCullough et al. 2000), and horizontal cracking (Choi and Won 2004).

![Figure 2.5 Frequency distribution of longitudinal crack locations with punchouts obtained from LTPP GPS-5 program, after Selezneva (2003).](image)

Figure 2.5 Frequency distribution of longitudinal crack locations with punchouts obtained from LTPP GPS-5 program, after Selezneva (2003).

![Figure 2.6 Crack shapes and patterns associated with defective passive cracks, after Kohler and Roesler (2004).](image)

Figure 2.6 Crack shapes and patterns associated with defective passive cracks, after Kohler and Roesler (2004).
2.2.2 Crack Spacing

Figure 2.7 shows the schematic view of some factors influencing CRCP behavior. It includes the crack spacing and crack width, and they are regarded as the primary early age CRCP performance indicators. Transverse crack spacing is the most frequently used performance indicator of the structural response of CRCP in previous literatures, mainly because it is the most visible one. The effect of design, construction, and construction variables on crack spacing has been well-documented (Won 1990; Suh et al. 1992; Zollinger et al. 1998). Initially, the AASHTO 86/93 guide suggests that the crack spacing should be between 1.0 m and 2.4 m. It states that to minimize the incidence of crack spalling, the maximum spacing should be no more than 2.4 m. While, to minimize the potential for the development of punchouts, the minimum desirable crack spacing to be used for design is 1.0 m (Rasmussen et al. 2009). It is commonly regarded nowadays that the shorter crack spacing is not desirable, as it will increase the potential of punchouts. However, many field investigations show that the slab support, not crack spacing, is more responsible for punchouts (Zollinger 1989; Verhoeven 1993; Won 2009). The new CRCP design procedure described in the AASHTO Interim MEPDG guide does not provide recommendations on the control of minimum crack spacing (ARA Inc. 2004). Finally, it should be noted that with respect to crack spacing, cluster cracking and Y-cracking are a unique case of short crack spacing that could be problematic in terms of their contribution to punchouts. Thus, the efforts to evaluate the causes of cluster cracking in CRCP, and subsequently methods to reduce them are beneficial to improve the CRCP performance.

Figure 2.7 Schematic representation of some factors influencing CRCP behaviour, after Rasmussen et al. (2009).

Causes of cluster cracks

Rasmussen et al. (2009) consider that cluster cracking is mainly caused by localized weak support or because of inadequate concrete consolidation. In other words, cluster cracking is regarded to be more construction related, and not as much a design issue. However, recent research evidences have indicated that cluster...
cracking can be related to several structural and construction issues that are summarized as follows:

*Material related*
A crack will occur when and where the concrete stress exceeds the tensile strength of concrete. If the concrete slab is assumed to be homogenous, the new crack will occur at the center of two previously formed transverse cracks because of the maximum concrete stress at the center (Won 1990). However, the tensile strength of field concrete normally varies from location to location. Won et al. have used the Monte Carlo method to evaluate the effect of variation of the tensile strength on the crack development along the pavement length for CRCP, as shown in Figure 2.8. It should be noted that not only the tensile strength of the concrete varies along the pavement length, but the thermal stress changes as well, which will result in a more random distributed crack pattern in CRCP. Besides the variation of environmental induced loading, Jackson (1988) considered that the variation of the wheel load stresses due to variation in the base support might be another factor that attributes to the causes of cluster crack in CRCP.

Figure 2.8 Monte Carlo methodology for crack spacing prediction due to the variation of the tensile strength, after Won et al. (1990).
Structure related

Figure 2.9(a) shows the calculated concrete stress and steel stress in CRCP through a three-dimensional finite element model (Nishizawa et al. 2013). It shows that the maximum tensile stress at top of the slab occurs at midspan but a small peak appears near the crack when the bond strength and stiffness between concrete and steel are high. Nishizawa also found that a similar movement of the location of the maximum tensile stress at top of the slab when using a low stiffness of the base course, such as asphalt interlayer. It might be one of the causes of the ‘secondary’ crack that occurs about 0.3 to 0.5 m away from the ‘primary’ crack, which is very often observed in actual CRCP. Figure 2.9(a) also indicates that as the crack spacing decreases, both the concrete stress and steel stress decrease. Moreover, the small peak of tensile stress near the primary cracks disappears when the crack spacing becomes short.

![Figure 2.9](image)

(a) effect of crack spacing  
(b) effect of base stiffness

Figure 2.9 (a) Effect of crack spacing on the steel stress and concrete tensile stress at the top of pavement slab; (b) effect of base stiffness on the concrete tensile stress at top of the pavement slab ($E_b$ in the right figure is the stiffness of the asphalt interlayer), after Nishizawa et al. (2013).

Transverse rebars related

Field investigations have shown that many transverse cracks in CRCP occur at the location of the transverse steel bars, about 35% to 50% for single layered reinforcement, and higher values ranging from 50% to 65% for double-layered reinforcement reported by Suh et al. (1992). Even higher values varying from 60% to 80% were also reported by Choi et al. (2015). Besides, with regard to the test section with active crack control method, nearly all the random transverse cracks were found over the transverse steel bars (Kohler and Roesler, 2004). Hasson et al. (2005) stipulated that the transverse steel reinforcement is beneficial in forming a regular transverse crack pattern for CRCP. However, a contrary and well-accepted opinion
by researchers in the field of CRCP, including the present author, is that the right angle placed transverse steel rebars could lead to a random crack pattern, mainly because of the variation of the concrete strength and concrete stress. In the current standard design concept of CRCP in Belgium, the transverse reinforcement bars are placed at an angle of 60 degrees to the longitudinal steel due to concerns that the transverse bars could be crack inducer and could thus influence the crack pattern (FEBELCEM, 2006). Several explanations on the effect of transverse steel bars on crack pattern of CRCP were proposed by previous studies. Suh et al. (1992) considered that the higher probability of this type of transverse cracks was perhaps owing to the reduction of the cross section area. However, the amount of the transverse steel typically used in CRCP is quite small, in the range of 0.05% to 0.11% of the cross-sectional area (Choi et al. 2015). Al-Qadi and Elseifi (2006) concluded that the tensile stress concentration near the transverse steel bars might be the dominant factor attributing to the initiation of transverse cracking in CRCP from three-dimensional finite element simulations, as shown in Figure 2.10.

![Figure 2.10 Calculated concrete longitudinal stresses on top of the transverse steel bars at different times of the day, after Al-Qadi and Elseifi (2006).](image)

Construction related
The current available crack spacing data are mostly obtained on an individual lane. However, there are normally more than three or even more lanes in each carriageway in practice, and they are often paved separately. Field investigations of crack pattern in Belgium have shown that some transverse cracks might be induced by existing cracks in the earlier paved adjacent lanes. As can be seen in Figure 2.11(a), many transverse cracks in the emergency lane and the shoulder are located at the same location. The emergency lane and the shoulder were not tied. Although the times of occurrence of these cracks in the emergency lane and the shoulder are unknown, some transverse cracks in the emergency lane are believed to be induced by an existing crack in the shoulder, as shown in Figure 2.11(b). The
crack in the shoulder at the saw cut joint is expected to be initiated first. Transverse cracks initiation by an adjacent lane paved separately were also found in the field investigation of a CRCP section on the motorway A5 in the Netherlands (Ren and Houben, 2015). Gatti (2011); Mu et al. (2012) have studied the premature transverse cracking due to paving adjacent lanes separately by finite element calculations. They concluded that the thermal incompatibility is the cause for the initiation of transverse cracks by adjacent lanes paved separately.

Figure 2.11 Crack induced by the existing crack in the adjacent lane. (a) and (b) in E17, De Pinte, Belgium; (c) CRCP roundabout in Park Area Genk, Belgium.

2.2.3 Crack Width

Crack width traditionally has been the controlling factor in the design of CRCP (AASHTO Guide 86/93; VENCON). Crack widths affect CRCP performance in several ways. Excessive crack widths may lead to: a) water infiltration that can reduce the concrete slab support condition and cause rusting of the longitudinal reinforcement steel. b) loss of load transfer efficiency that results in increased slab deflections (possibly lead to faulting) and stresses, and higher stresses in the concrete, in turn, lead to spalling, additional cracking, and punchouts. c) infiltration of incompressible material, causing spalling and blow-ups.
A maximum allowable crack width of 1.0 mm at the pavement surface was suggested in the AASHTO 86/93 guide based on the considerations of spalling and water penetration. However, a crack width of 0.6 mm or less has also been reported to be effective in reducing water penetration at 0 °C, thus minimizing corrosion of the steel and maintaining a high load transfer efficiency (McCullough and Dossey 1999). Recently, the AASHTO Interim MEPDG design guide (ARA Inc. 2004) specifies a maximum crack width of 0.5 mm at steel depth to minimize the possibility of corrosion of the reinforcement. In Europe, according to EN 1992 and EN 206, an allowable crack width of 0.4 mm for CRCP is proposed according to the environment classes that the pavements generally experience. Different models have been proposed to predict the crack width in CRCP (Vetter 1933; Reis et al. 1965; Palmer et al. 1988; Sato et al. 1989; Won 1990; Jiménez et al. 1992; van Breugel et al. 1998; Kohler and Roesler 2005; Kohler and Roesler 2006). The present study does not attempt to discuss all the above-mentioned crack width models, but rather investigate the crack width measurement methods and how different factors are affecting the measured crack width.

**Crack width measurement**

Despite the fact that the crack width is widely recognized as a vital variable influencing the performance of CRCP, the value of most of the available crack width data is limited by lacking a clear explanation how the measurements are done, the location of the crack width measurement and at what temperature condition the measurement are done. Almost all the crack widths reported were measured on the pavement surface by graduated-eyepiece microscope (Suh et al. 1992; Kohler and Roesler 2005; Nam et al. 2007). The used microscopes generally have a resolution of 0.025 mm to 0.05 mm. However, it is quite difficult to obtain an accurate and representative crack width on the slab surface. It is because the crack width varies along the crack, and crack faces are quite rough under the microscope. Van Avermaet and Van Weyenberge (2011) also adopted a microscope to measure the crack width on the surface. They found that the crack width was not constant over the depth. The variation of the crack width at the surface is remarkably larger, varying from 0.1 mm to 20 mm. They observed that there was a small variation of crack width at just a few millimetres below the surface. As shown in Figure 2.12, Van Avermaet and Van Weyenberge measured the crack width about 5 mm below the pavement surface. Suh et al. (1992) recommended the crack width measurements to be performed by one operator to reduce the measurement error by eliminating operator variance. Suh et al. (1992) concluded that there was no significant difference between the average of the crack widths measured at three locations (near both edges and the center) and the crack width measured at the center. Braam and Frenay (2004) measured crack width on the pavement surface at more measurement points along each individual transverse
crack. The measurement points were equally spaced at 67.5 mm that is half of the longitudinal reinforcement bar spacing. Braam and Frenay found that there was no statistically significant difference between both mean crack widths and individual crack width measurement. However, it should be noted that Braam and Frenay used a plastic card with lines of different thickness to measure crack widths and the measured crack width were rounded to the nearest 0.05 mm.

![Image of crack measurement](image)

**Figure 2.12** Crack width measurement using microscope: (a) on the pavement surface, after Nam (2005); (b) at a small distance below the surface, after Van Avermaet and Van Weyenberge (2011).

Most of the crack width models for reinforced tensile members define a crack width at the location of the reinforcement. In order to obtain the actual crack width at the depth of the steel reinforcement, Kohler and Roesler (2006) developed a procedure called the load spectra test to evaluate the crack width at different depth and under various temperature conditions. The crack width data were obtained from full-scale CRCP test sections at the Advanced Transportation Research and Engineering Laboratory at the University of Illinois. As shown in Figure 2.13, the crack width movement was captured by LVDTs between studs at each side of the crack.
Cores taken from CRCP sections have indicated the variation of crack width through the depth of the concrete. Cores taken from sections in Illinois indicated that the cracks were widest at the surface. In some cores, the crack width decreased with slab depth and became almost non-existing at the bottom of the core. However, other cores showed that the cracks become discontinuous in the intermediate vicinity of the reinforcing bar and widened towards both the top and bottom surfaces (Kohler and Roesler, 2004). It is consistent with numerical simulation results by Nishizawa et al. (2013). As shown in Figure 2.14, the calculated crack width is smallest at the location of the longitudinal reinforcement, and is widest at the pavement surface. The simulated results of Nishizawa also indicate that a larger crack spacing would result in a larger crack width.
Effect of construction and design variables

There are several construction and design variables significantly affecting the crack width in CRCP, ranging from the geometrical parameters to material properties. Kohler (2004) elaborated that the long list of factors implies that there is more than one solution for problems related to crack width, and it is of great importance to find the most cost effective solutions. Kim et al. (2003) concluded that the curing temperature and the thermal expansion coefficient of concrete (CTE) are the most sensitive design variables through a sensitivity analysis of design variables for CRCP using a mechanistic model, the CRCP-10 program. During the TxDOT study 0-1244 (Suh et al. 1992), the effect of placement season and aggregate type on the crack width at the pavement surface were evaluated in four field test sections, as shown in Figure 2.15. The crack width for the summer construction was much larger than those associated with winter construction, and it was considered a consequence of the high curing temperature of summer placement. With respect to the effect of the aggregate type, the use of siliceous aggregate resulted in larger crack width than the use of limestone aggregate. Zollinger et al. (1998) proposed a set of construction guidelines for CRCP based on coarse aggregate type and weather conditions. Aggregates were divided in categories according to their CTE, and different recommendations were made for concrete placement under different weather conditions.

![Figure 2.15](image1.png)

(a) effect of construction season
(b) effect of aggregate type

Figure 2.15 Measured crack width at the surface of CRCP slab in the TxDOT study 0-1244, (a) effect of construction season; (b) effect of aggregate type (SRG = siliceous river gravel, LS = limestone) and concrete temperature, after Suh et al. (1992).

Selecting the adequate longitudinal reinforcement percentage to limit the crack width is the starting point for the current CRCP design procedures (ARA 2004; VENCON 2004). The other design variables related to reinforcement steel include the longitudinal bar diameter and depth of concrete cover. In terms of crack spacing, a longitudinal steel percentage of 0.55% to 0.70% has provided suitable CRCP performance (Zollinger et al. 1998). Bar size has an influence on crack
development in the way that the restraint of the longitudinal steel depends on the bond area provided by the reinforcing bar. For the same percentage of longitudinal steel, the smaller bar size results in a larger steel surface area, which increases stress transfer from the steel to the concrete and results in tighter cracks. The depth of cover of the longitudinal steel bars also affects the crack pattern because of the volumetric changes are greatest at the pavement surface and decrease with depth. If the steel is placed near the surface of the slab, the restraint to induced movements increases that leads to a tighter crack width at the pavement surface. It is consistent with the numerical simulations by Nishizawa et al. (2013), as shown in Figure 2.14. The longitudinal reinforcement is normally placed in the centre of the concrete slab in the Netherlands (Houben 2006), while it is placed above the centre of the slab in Belgium. The concrete cover of the longitudinal steel is 80 mm for a 250 mm thick slab in the current typical CRCP design in Belgium (Rens 2010).

**Effect of crack occurrence time**

Field investigations have revealed that cracks that form within the first few days after pavement construction tend to be wider than cracks that form later during the life of the pavement (Suh et al. 1992), as shown in Figure 2.16(a). Similar findings were also reported by Kohler and Roesler (2004), Nam (2005), and Won (2009). One possible explanation for this behaviour is due to the bond strength development at the concrete-steel interface. At earlier ages, the bond strength is weaker than that at a later age. Therefore, the bond strength does less restrain the movement of the crack at earlier ages as it does later when this bond is stronger and the restraint to movement is higher. This phenomenon results in tighter cracks. Furthermore, concrete drying shrinkage also increases with time after placement. Because drying shrinkage is one of the factors that govern the contraction of the concrete, the increase of drying shrinkage with time will also affect the crack width. Figure 2.16(b) shows the measured drying shrinkage of actual field paved concrete by Nam (2005). For a crack that occurred at 2 days after concrete placement, there will be additional 200 micro-strains of drying shrinkage at 30 days. On the other hand, if the crack occurred at 14 days after concrete placement, additional drying shrinkage at 30 days will be about 70 micro-strains, and drying shrinkage of about 180 micro-strains (in this case) up to 14th day was absorbed by the creep of concrete. The resulting crack width of the crack that occurred at 14 days will be smaller compared with the width of the crack that occurred at 2 days, if the crack spacing is comparable. Depending on the environmental condition and the effectiveness of curing operations, some cracks may form quite late (Won 2009). According to Zollinger et al. (1998), primary cracks form within the first 3 to 7 days after concrete placement, and it is between 6 months and 3 years before the final crack spacing distribution is completed with the secondary cracks.
(a) after McCullough 1992

(b) after Won 2009

Figure 2.16 (a) Effect of time of crack occurrence on crack width, after Suh et al. (1992); (b) measured concrete shrinkage in the field curing condition, after Won (2009). (1 in = 25.4 mm).

Effect of crack spacing

It is generally accepted that the greater the crack spacing, the wider the crack width. However, Figure 2.17(a) shows the plot of the crack width versus crack spacing observed in TxDOT research project 0-1244, indicating no significant correlation between crack spacing and crack width. It should be mentioned that the crack width data in Figure 2.17(a) includes cracks that occurred at various age. As pointed above by Suh et al. and Moon, the crack occurrence time has a substantial effect on crack width for CRCP.

Figure 2.17 Relationship between crack spacing and crack width of four test sections in the study of TxDOT 0-1244, after Suh et al. (1992). (1 in = 25.4 mm, 1 ft = 12 inch).

Figure 2.17(b) shows the slab movement at different distance from the adjacent transverse cracks by periodically measuring the length between Demac points embedded in the concrete in the study of TxDOT 0-1244. Demac points were installed continuously for over 3 m in a longitudinal direction. Suh concluded that
the slab movements are confined near the transverse crack area. In other words, only the concrete adjacent to the transverse cracks instead of all the concrete between two transverse cracks contributes to the crack width. Another field test on the effect of crack spacing on crack width was conducted in a subsequent TxDOT research study 0-1700 through measuring the steel strain (Nam 2005). Steel strain gages were installed at 0 m, 0.15 m, 0.30 m, 0.45 m, and 0.60 m from an induced transverse crack, as illustrated in Figure 2.18. It was found that the steel strain remains almost zero beyond 0.30 m from the transverse crack. Won (2009) thus concluded that the concrete volume changes contributing to the crack width at the steel depth is limited to about 0.30 m from a transverse cracks.

![Layout of steel strain gage installation](image)

**Figure 2.18** Steel strain measurements in CRCP of the TxDOT study 0-1700, after Nam and Won (2005). (1 in = 25.4 mm).
2.3 CONCRETE PAVEMENT TEMPERATURE

2.3.1 Existing Climatic Models for Concrete pavement

Several numerical models are available for evaluating the temperature and/or moisture distribution in concrete pavements, such as the well-known Enhanced Integrated Climatic Model (Dempsey et al. 1986), High performance concrete paving (Schindler et al. 2002), Temperature and Moisture Analysis for Curing Concrete (Yang 1996; Ye 2007).

Enhanced Integrated Climatic Model (EICM) is a one dimensional coupled heat and moisture flow model initially developed for FHWA and later adopted in the Mechanistic Empirical Pavement Design Guide developed under NCHRP project 1-37A (ARA Inc. 2003b). However, the EICM model does not include the internal heat of hydration, and it is thus not capable to simulate the temperature development during concrete hardening phase. High performance concrete paving (HIPERPAV), a concrete paving software sponsored by FHWA, enables users to evaluate the development of concrete strength and stress resulting from varying temperature and moisture within the pavement during early ages (first 72 hours after concrete placement). The core module of HIPERPAV is a general early age concrete pavement behaviour model that predicts the development of concrete temperature, shrinkage and the strength as a function of various material composition, construction, and climatic conditions. HIPERPAV can help contractors in managing concrete temperature based on mix designs and specific climatic and construction conditions. The first version of HIPERPAV I was released in 1996, the latest version is HIPERPAV Version 3.2 after extensive modelling enhancements. Many validation tests in the United States have shown that HIPERPAV was found to predict crack formation with an accuracy of approximately 5 hours.

Temperature and Moisture Analysis for Curing Concrete (TMAC2) is a finite element program developed in Texas Transportation Institute to predict the concrete temperature and moisture during early age. The features of this program are that it first includes heat loss due to evaporation, back calculated thermal conductivity and moisture diffusivity, and coupled thermal and moisture analysis (Yang and Zollinger, 1996; Ye and Zollinger, 2007).

Others numerical temperature prediction models either are also available for the hardening concrete pavement (Ge 2005) or for hardened concrete pavement (Bentz 2000; Qin 2011). In addition to the specialized models for concrete pavements, numerous models that well predict the early age behaviour of concrete are available in Europe. These include, but not limited to, FEMMASSE, HYMOISTRUC, and Ghent Model. These models are similar in some aspects to the above-mentioned specialized models for concrete pavement, but also have differences in the theory and application, such as the adopted type of cement hydration model and the treatment of the heat transfer boundary conditions. FEMMASSE is an
abbreviation of Finite Element Modules for MAterial Science and Structural Engineering. FEMMASSE was started in the early 1980’s at Delft University of Technology, and was continued at Swiss Federal Institute of Technology in Lausanne. FEMMASSE enables the user to calculate the temperature, moisture, stress and cracking in the pavement under different material properties, construction sequence, and climatic conditions. Raoufi et al. (2008) have used FEMMASSE to investigate the stress and cracking behaviour of concrete pavements under a variety of saw cutting sequences and environmental conditions.

Another well-known numerical model called HYMOISTRUC is based on microstructure development and is developed by van Breugel (1991) at Delft University of Technology. HYMOISTRUC focuses on the microscopic level. It enables to predict adiabatic and isothermal hydration curves for concrete as a function of the particle size distribution, chemical composition, water to cement ratio, and curing temperature.

Lastly, a finite element method for the prediction of thermal stresses due to heat of hydration in hardening concrete structure has been proposed by De Schutter (1997) at the Magnel Laboratory for Concrete Research, University of Ghent, entirely based on degree of hydration, including basic creep and basic shrinkage. One significant feature of the De Schutter model is that it is valid for both Portland and blended slag cement.

Although each of the above-mentioned concrete pavement temperature models were considered the best option at the time of their development, new developments are now available that could increase the accuracy of prediction, especially for the specific concrete pavement structure geometry, the construction, and curing sequences. Many studies have shown that the temperature distribution through the pavement slab primarily depends on the climatic condition, the adoption of upper and bottom boundary conditions, and adequately describing the heat of hydration under field condition for specific cement type and mixtures (Schindler et al. 2002; Ge 2005; Yoen 2011). Detailed discussions about the heat flux boundary conditions for concrete pavement will be further illustrated in Chapter 4. A review of the heat of hydration tests and the available concrete hydration models are given in the subsequent sections.

2.3.2 Degree of Hydration

Heat evolution is a fundamental parameter to predict the pavement strength and stresses during the early age. The rate of heat hydration under field condition varies with concrete age and curing temperature and it is generally calculated by the equivalent-age maturity method. The fundamental parameter ‘degree of hydration’ has to be determined in the maturity method. The degree of hydration for a concrete mixture is defined as the cement fraction that has reacted and it can experimentally be determined by a number of direct and indirect techniques. Two
indirect methods, amount of chemically bound water and amount of heat generated during hydration, are commonly used to determine the degree of hydration (van Breugel 1991). The latter method, amount of heat generated, is considered as a more practical way because it is easy to measure. A linear correlation has been reported between the heat of hydration and the amount of non-evaporable water, from which it is concluded that these two methods are equivalent to reflect the degree of hydration (van Breugel 1991).

![Figure 2.19 Schematic of heat of hydration development in the field condition of a concrete pavement, after Schindler et al. (2002).](image)

The commonly used methods to determine the cement hydration by the amount of heat generation include isothermal (conduction) calorimetry, semi-adiabatic and adiabatic calorimetry.

- Since it is impossible to achieve an adiabatic environment, the calorimeter is considered as adiabatic as long as the temperature loss of the sample is not greater than 0.02 K/h. The advantage of the adiabatic calorimetry method is that it enables to measure the heat evolution of an actual concrete mixture. However, one major drawback of an adiabatic calorimeter is that the effect of the curing temperature on the rate of hydration is measured implicitly. The degree of hydration has to be computed based on the heat transfer principle. The activation energy is required to convert the results to the isothermal reference temperature. The results, thus, are affected by the inaccurate assumption of the activation energy and the material thermal properties. The most important is that the adiabatic calorimetry method cannot represent the two peaks feature of the curve of heat generation rate of blast furnace slag cement under field condition.

- The semi-adiabatic calorimetry method is similar to the adiabatic method except that a known amount of heat loss is allowed to occur over time. The maximum heat loss should be less than 100 J/h/K. The temperature development is not as high as with the adiabatic calorimeter test. The additional disadvantage of the semi-adiabatic method is that the true adiabatic calorimetry test has to be determined and the heat losses associated with the test have to be accounted for.
• Isothermal calorimetry is conducted at a constant temperature and the heat of hydration is directly measured by monitoring the heat flow from the specimen. Isothermal calorimeters are usually used for paste samples mixed in the lab or the mortar samples sieved from the concrete mixture. The total heat evolution can be determined by the summation of the measured heat over time. The first drawback of the isothermal calorimetry test is that it does not take into account the cement reactivity change due to the change of temperature. Another disadvantage is that the duration of this test is normally limited to 7 days due to the signal resolution. After about 7 days, it is hard to distinguish the signal from its background (Wang et al. 2009; Sedaghat et al. 2012). Isothermal calorimeters are more widely used for studying the kinetic reaction of cement pastes (Wang et al. 2008).

2.3.3 Heat of Hydration of GGBFS Cement

The blended GGBFS slag cements are widely used in concrete pavement construction. Unlike the hydration of ordinary Portland cement that is well described, the hydration process of blended cement is considerably more complex. It involves reaction of pozzolanic materials in addition to the hydration of Portland cement (Pane and Hansen 2005).

![Figure 2.20 Effect of slag on hydration, measured by cement paste sample with a water to cement ratio of 0.5 at 20 °C. ‘OPC’ is the Ordinary Portland Cement, adopted from Kishi and Maekawa (1994).](image)

Figure 2.20 shows the effect of replacement levels of slag on the heat generation rate for blended cement under isothermal condition (Kishi and Maekawa 1994). There are two peaks for blended slag cement. The first one is caused by the Portland cement hydration and the second is due to slag reaction. The slag cement reaches the first peak at the same time as the pure ordinary Portland cement. It suggests that adding GGBF slag into cement does not delay the reaction of cement. The second peak is unaffected by the amount of slag as shown in Figure 2.20. Kishi and Maekawa (1994) explained that the slag react independently as long as sufficient calcium hydro-oxide is released from the cement hydration, but at higher
slag replacement, the reaction of slag is stagnant because of shortage of calcium hydro-oxide in pore solution. However, it should be noted that the reactivity of slag can largely vary from one slag to the other (Chen 2006), which indicates that the second peak of heat of hydration might not be so obvious for certain types of blended slag cement.

Figure 2.21 Isothermal heat of hydration (a) at 23 °C and (b) at 34 °C. ‘45’ and ‘35’ denotes that the water to cement ratio of the measured sample is 0.45 and 0.35, respectively; ‘1 to 4’ represents 100% ordinary Portland cement I (OPC I), 75% OPC I + 25% Fly ash, 75% OPC I + 25% GGBFS, 90% OPC I + 10% silica fume, respectively, adopted from Hansen (2006).

In general, the degree of hydration is defined as the ratio of the amount of liberated hydration heat $Q(t)$ at time $t$ relative to the maximum amount of heat $Q_\infty$ potentially liberated, which requires $Q_\infty$ being determined. For Portland cement, $Q_\infty$ can be calculated from the Bogue equation according to the cement clinker composition, but no equivalent of Bogue's formula exists for blast furnace slag cements (Bogue 1947). Many researchers have used the isothermal calorimeter to study the heat of hydration of blended cements containing granulated blast furnace slag (De Schutter and Taerwe 1995; Xiong and van Breugel 2001; Pane and Hansen 2005; Wang et al. 2006). De Schutter and Taerwe (1995) used the measured total heat of hydration at the end of the reaction (about 7 days) to calculate the degree of reaction instead of degree of hydration for the blended slag cement.
because \( Q_\infty \) is unknown. Pane and Hansen (2005) performed isothermal calorimetry tests on blended slag cement with a test duration of 3 weeks. Figure 2.21 illustrates the measured isothermal heat of hydration for 4 mixture at 23 and 34 °C, and two w/c ratios of 0.35 and 0.45, respectively. Pane and Hansen found that the total heat of hydration measurement up to 21 days may still not be sufficient to accurately determine \( Q_\infty \). It should be noted that Pane and Hansen concluded from the measurement that the maximum heat of hydration for the same mixture is temperature dependent. However, many others studies suggested that the maximum heat of hydration at different temperature are close, as can be seen in the results of Pane and Hansen themselves in Figure 2.21.

Many mathematical models have been proposed to predict the heat of hydration based on the hydration-maturity relationship (Hansen and Pedersen 1977; Byfors 1980; Knudsen 1982; Roelfstra 1989; De Schutter and Taerwe 1995; Maekawa et al. 1999). Detailed descriptions of those models are outside the scope of this study, and comprehensive discussions of those models used for the concrete pavement temperature prediction are well documented in the relevant literatures (Schindler 2002; FEMMASSE, 2005; Wang et al. 2006). Hereafter, only two models proposed by Hansen and Pederson (1977) and De Schutter (1995) are discussed.

The exponential function (here called FHP model) proposed by Hansen and Pedersen (1977) has been proven to be a suitable equation to represent the degree of hydration of ordinary Portland cement, and it is expressed as follows:

\[
\alpha(t) = \alpha_u \cdot \exp \left( - \left[ \frac{\tau}{t} \right]^\beta \right) 
\]

(2.1)

Where,
- \( t \) = time, [hour];
- \( \alpha_u \) = ultimate degree of hydration, [-];
- \( \tau \) = hydration time parameter, [hour];
- \( \beta \) = hydration slope parameter, [-].

The FHP model has been successfully applied to characterize the hydration process of concrete with ordinary Portland cement. This model has been selected as the default heat of hydration and degree of hydration model in numerous programs, such HIPERPAV, FEMMASSE, TMAC\(^2\), etc. However, isothermal tests clearly show a second peak that relates to the hydration of slag in the concrete mixes, as shown in Figure 2.20. Such a heat evolution peak is however not observed from the generated heat of hydration curve based on the FHP hydration model. De Schutter and Taerwe (1995) proposed a new hydration model for granulated blast furnace slag cements, which is based on isothermal and adiabatic hydration tests. The heat of hydration is calculated as function of the degree of hydration and the temperature. De Schutter and Taerwe have verified the accuracy of the temperature simulation by the temperature rise in an adiabatic test for the concrete sample made of granulated blast furnace slag cement. A detailed
description of the model and the determination of the model parameters is presented in Chapter 4.

2.4 EXPERIENCES ON ACTIVE CRACK CONTROL FOR CRCP

2.4.1 Texas Experiences
In the early 1990s, the Texas Department of Transportation constructed several CRCP test sections to investigate crack control methods to eliminate clusters of closely spaced cracks.

State Highway (SH) 290 in Cypress, Texas
The CRCP test section on SH 290 in Cypress, Texas was constructed in August 1992. It consisted of a 330 mm thick pavement, and a double layer of longitudinal reinforcement. Four different coarse aggregate mixtures were applied and the air temperature during construction ranged from 32 to 38 °C. Moreover, although the temperature was quite high, the Cypress test section also included the polyethylene sheet curing method. The length of the transverse crack control section is approximately 365.8 m. Two crack control methods were applied in the Cypress test section: the early-age shallow saw cutting technique and crack inducers placed at the interior of the pavement slab, as shown in Figure 2.22. In the early-age saw cutting technique the shallow surface notches with a nominal depth of 25.4 mm were made between initial and final setting of the concrete (in this case it was about 4 hours after concrete placement). The notches were made at intervals of 0.9 m, 1.2 and 1.5 m combinations, 1.8 m, and 2.7 m. In the second type of crack control method, metallic crack inducers were placed in both single and stacked double layer configurations and were anchored to the double layer of longitudinal reinforcement to provide support against the flow of fresh concrete during the paving operations.

![Figure 2.22 Crack control method used in Cypress test section, (a) early-age saw cutting techniques; (b) metallic crack inducer located on top of the longitudinal rebars, after McCullough et al. (2000).](image-url)
Figure 2.23 shows the cracking development at saw cut location in the Cypress test section. It is noted that nearly 100% cracking occurred in the notches spaced at 0.9 m and 1.2/1.5 m notch combinations approximately 3 days after paving. The longer saw cut interval sections, 1.8 m and 2.7 m, took longer time to reach 100% cracking at the notches after placement. It was also found that secondary cracking occurred in the 2.7 m saw cut interval sections.

![Crack development at saw cut locations.](image1)

(a) Crack development at saw cut locations.

![Percentage of cracks initiated at crack inducer.](image2)

(b) Percentage of cracks initiated at crack inducer.

Figure 2.23 The effect of crack control method on crack pattern in Cypress test section, Texas, after McCullough et al. (2000).

Experiences on crack control methods for CRCP obtained from the Cypress test section are summarized as follows:

- Surface crack initiation using the early age notching method is more effective than the interior crack inducer in controlling the crack pattern, and the single interior crack inducer is found to be less effective. The short saw cut or crack inducer interval results in faster and more crack induction at the designated locations.
- Timing of saw cutting plays an important role to achieve the goal of crack induction at shallow notch depths.
- Although the notch width is larger than the initial random crack width, the notches were not sealed. The distress of spalling due to the unsealed notch-
es has not been observed until the authors have written the report, 6 years after the construction (Zollinger et al. 1998).

**State Highway (SH) 225 in LaPorte, Texas**

The LaPorte test section on State Highway 225 in Texas was constructed in November 1992 (Tang et al. 1994). The thickness of CRCP slab in the LaPorte test sections is 330 mm and with two layers of longitudinal reinforcement. The longitudinal reinforcement percentage was 0.60%. The total length of this test section was 777 m, and was sub-divided into nine sub-sections to evaluate the effect of different curing techniques, saw cut techniques and placement of skewed transverse steel on the performance of CRCP. Figure 2.24 shows the layout of the test sections by different curing types and crack control methods. The crack control method through sawcut was used in sub-section 6 and 9, and each sub-section had a length of 76.2 m.

![Figure 2.24 Layout of test sub-sections in LaPorte test section in SH 225, Texas, after Tang et al. (1994).](image)

Figure 2.24 Layout of test sub-sections in LaPorte test section in SH 225, Texas, after Tang et al. (1994).

![Figure 2.25 Crack pattern development in the sub-section 6 of LaPorte test section, after Tang et al. (1994).](image)

Figure 2.25 Crack pattern development in the sub-section 6 of LaPorte test section, after Tang et al. (1994).
The saw cut was made by ‘dry sawing’ after the concrete and curing compound had solidified to a point where it could support the saw and operator, about 7 hours after concrete placement. Four saw cut machines, ‘Soff-cut’, were used simultaneously in order to meet the pavement saw cutting schedule. The saw produced a shallow transverse surface notch by cutting in a single pass, and it had a depth of 25.4 mm and a width of 3.2 mm. The transverse notches were sawed at 0.9 and 1.5 m intervals. Figure 2.25 showed the crack pattern development in the sub-section 6 during the first two weeks after concrete placement. The following conclusions on the crack control method were made by the investigators of this project according to periodical crack pattern surveys until 125 days after concrete placement:

- Field practice has shown that the early age saw cutting can be performed without ravelling when the pulse velocity of concrete reaches 1520 m/s;
- In the sawcut test section, most transverse cracks were initiated from the sawcut and the rest of the cracks were initiated from transverse steel rebars. At the age of 8 days after concrete placement, in total 13 transverse cracks was observed and they were located at the location of a sawcut. After that, another 14 new transverse cracks initiated between 8 days and 15 days, five of them were at the sawcut and the rest of the cracks were initiated above transverse steel rebars.
- Cores taken at the random transverse cracks in the sub-sections without saw cuts showed that there was delamination present. There were no indications of delamination in the cores taken at the transverse cracks initiated by transverse saw cuts.
- Slight spalling was observed, mostly along the wheel paths.

Farm-to-Market Road (FM) 528 in Friendswood, Texas
The CRCP test section on FM 528 in Friendswood, Texas, was constructed in November 1993 (McCullough et al. 2000). The thickness of the CRCP slab is 254 mm. The purpose of this section was to determine the best method to active control the crack pattern in CRCP under cool weather paving conditions using river gravel coarse aggregate. Figure 2.26 shows the layout of the test sections in Friendswood on FM 528. The Friendswood test sections consisted of four crack control methods: three crack inducers and the saw cut method. The type I crack inducer consisted of L-shaped angled metal steel placed at 1.5 m interval and served as a support for the longitudinal reinforcement steel that eliminated the need for transverse rebars, as shown in Figure 2.27(a). In addition, transverse sawcut notches with 1.5 m and 0.75 m intervals were made and aligned with some of the type I crack inducers, and those sawcut notches were made about 7 hours after concrete paving. In the other three subsections in Friendswood, the conventional transverse rebars were used to support the longitudinal reinforcement at standard 0.75 m intervals. The
type III crack inducer, as shown in Figure 2.27(b), consisted of corrugated metal sheets placed on top of the longitudinal reinforcement. The type II crack inducer was plastic deboning inserts. The crack inducers were placed at 1.5 m intervals, and coinciding with alternating transverse rebars.

Figure 2.26 Layout of the test sections on FM 528 at Friendswood, Texas, after McCullough et al. (2000). (1 ft=0.30 m).

Figure 2.27 Crack inducers used in Friendswood test sections.

Figure 2.28 Percentage of crack occurrence at (a) Type I crack inducer, (b) Type III crack inducer with or without notches, after McCullough et al. (2000).
The following conclusions on the crack control method were made by the investigators of this project according to periodical crack pattern surveys until 125 days after concrete placement:

- Type I crack inducer and Type III crack inducer may be more efficient in crack initiation than saw cutting under cool weather construction conditions, as indicated in Figure 2.28;
- The type II crack inducer did not function as a crack inducer;
- Some of the cracks were induced by existing cracks in the adjacent lane;
- It appeared from drilled cores that the position of the type III crack inducers, located on top of the longitudinal rebar, was disturbed by the paving operations.

2.4.2 Illinois Experiences

In December 2001, the University of Illinois constructed 10 CRCP sections to compare the performance of different structural designs under accelerated loadings and to characterize the crack development in CRCP with and without active crack control methods. Two 150 m long parallel lanes with a thickness of 250 mm were built at the Advanced Transportation Research and Engineering Laboratory in Illinois. The material composition, material properties, and design variables for this test section can be found in the relevant documents (Kohler et al. 2002). Two crack induction methods were applied, sawing and tape insertion, and the test sections were not loaded with traffic.

- **Sawing**

  Early age entry saw cut was used to cut a notch with a depth of 37.5 mm on top of the CRCP slab, as shown in Figure 2.29. The saw cuts were made approximately four hours after concrete placement, based on the work by Jeong et al. (2001).

![Sawing](image)  
(a) Soff Cut  
(b) 1.2 m interval and 37.5 mm depth

Figure 2.29 Early age entry saw cut used in the Illinois test section, Soff-Cut, after Roesler (2005).
• **Tape Insertion**

Kohler and Roesler (2004) applied an automated crack-inducing device to create a weakened plane by means of a plastic film inserted in the fresh concrete at the top part of the CRCP slab. The thickness of the used plastic film is only 0.07 mm, and the depth of tape inserted into the concrete is 75 mm. The automated crack-inducing device was reported to be able to insert the tape without jamming or tearing the tape, as shown in Figure 2.30(a). However, Kohler and Roesler (2006) also pointed out one problem that the tape inserter disrupted the concrete surface because it bumped into the reinforcing steel, as shown in Figure 2.30(b). A finisher had to spend some time to re-finish the concrete surface in the area of the inserted tape, as can be seen in Figure 2.30(c) and (d). The induced crack spacing was set at 0.6, 1.2, or 1.8 m with sawing and tape insertion used in an alternating pattern. Figure 2.31 shows the layout of the two crack control methods at each section.

![Figure 2.30](image)

**Figure 2.30** Execution of crack induction by automated tape insertion, after Kohler and Roesler (2006).
According to crack pattern surveys over 18 months after concrete placement, as shown in Figure 2.31, Kohler and Roesler concluded the following experiences on crack control method from the CRCP test sections in Illinois:

- Both the saw cutting and tape insertion method in the Illinois test section did successfully achieve transverse cracks sooner, straighter, and at the intended regularly spaced intervals relative to the natural crack sections, as can be seen in Figure 2.32(a);
- The tape insertion method developed cracks slightly faster than the saw cutting method, and were more effective, as shown in Figure 2.32(b);
- The active control methods for CRCP eliminated the occurrence of undesirable types of cracks, such as, Y-cracks, divided cracks, or meandering cracks;
- Induced cracks only occurred in the winter seasons;
- A few natural cracks occurred in the active crack control section, and most of them were found directly over the transverse rebars;
Kohler and Roesler concluded that the CRCP with an active crack control system should have a better long-term performance because of a uniform crack pattern.

![Graph showing crack development on the Illinois test section](image)

**Figure 2.32** Crack development on the Illinois test section. a) total number of cracks in passive and active crack control lanes; b) percentage of induced cracks over the first 5 months according to crack induction type, after Kohler and Roesler (2006).

### 2.4.3 Saw Cutting Requirements for Concrete Pavement

Concrete sawing operations are performed at pre-determined locations within the first 24 hours to prevent random cracking in jointed plain concrete pavements (Okamoto et al. 1994). The determination of the approximate timing and the depth has proven to be a difficult task in practice. The saw cut should be deep enough to ensure that the cracks initiate at those pre-determined joints in JPCP. However, a deep saw cut is not applicable for the case of active crack control method for CRCP because of the sufficient concrete cover requirement for the longitudinal reinforcement in CRCP. Besides, the deep saw cut in CRCP could lead to distresses initiation around those joints that common found in the case of JPCP. It would require maintenance that goes against the original intention of the CRCP design to eliminate the transverse joints to reduce the maintenance works and improve ride...
quality. Shallow and partial saw cut may be preferred for CRCP. With respect to
the timing of saw cutting, currently the timing of the saw cutting operation is
mainly determined based on the saw operator’s experience, the project length, and
the contractor’s resources (Thier 2005). When a saw cut is placed in concrete
pavement, the classical strength of materials approach to predict the average
section stress fails to capture the stress concentration that develops at the tip of a
saw cut. Thus, in several studies the principles of fracture mechanics are used to
describe the cracking behaviour cracking behaviour of concrete pavements with
saw cuts (Zollinger et al. 1994; Ioannides 1997; Jeong et al. 2001; Gaedicke et al.
2007; Raoufi et al. 2008).

Zollinger et al. (1994) proposed an approach for estimating the appropriate
sawcut depth and sawcut implementation timing by using fracture mechanics for
JPCP. The fracture mechanics-based model uses the size effect model to calculate
the required notch depth at a given time and applied far-field stress. The fracture
properties for the specific aggregate type used in Texas were determined from
laboratory notched three point bending beam fracture tests. Modified linear
fracture mechanics was applied to determine a sufficient notch depth to ensure
controlled cracking. Figure 2.33 illustrates the estimated sawcut depth guidelines
at the anticipated sawcut location for a given set of climatic conditions. Zollinger et
al. (1994) concluded that it was reasonable to use the shallow notch depth of about
25 mm to initiate cracking at the pavement surface. It is significantly less than the
traditional one third or one fourth of the slab depth, because the shallow notch can
take advantage of the greater change in moisture and temperature in the concrete
at the pavement surface to initiate a greater incidence of cracking at the notches.

Figure 2.33 Determination of sawcut depths, after Zollinger et al. (1994).

Gaedicke et al. (2007) first admired the approach proposed by Zollinger as a
solid basis to determine the appropriate saw cutting for concrete pavement.
However, considering the difficulty to obtain the early age concrete fracture properties by using the three points bending beam test, since the specimen’s weight significantly contributes to the peak load in the first 24 hours, Gaedicke and Roesler adopted the Wedge Splitting test to determine the early age concrete fracture properties. Gaedicke and Roesler measured the fracture properties as early as at an age of 6 hours after the concrete mixing. They concluded that the saw cutting depth requirements on a given project significantly vary depending on the timing and concrete properties. As shown in Figure 2.34, the required saw cut depth increases with concrete age, with a dramatic increase after 10 to 12 hours. If the saw cut is performed at an earlier age, for instance, 6 to 8 hours, then the depth of the sawcut can be much smaller than the typically specified one third or one fourth of the slab depth. It is also observed that a larger maximum aggregate size requires a larger saw cut depth.

![Figure 2.34 Saw cut depth ratio versus age for slab thickness 190 mm (left) and 380 mm (mm). ‘555.44’ denotes that a concrete mixture having a cementitious content of 270 kg/m$^3$ with a water to cementitious ratio of 0.44, and the used maximum aggregate size is 38 mm; ‘688.38st’ denotes a concrete mixture having a cementitious content of 349 kg/m$^3$ with a water to cementitious ratio of 0.38, and the used maximum aggregate size is 25 mm, after Kohler and Roesler (2006).](image)

Raoufi et al. (2008) proposed a procedure to estimate the end of the saw cutting window using the average stress from an uncut pavement, the tensile strength, and a strength reduction factor. Figure 2.35(a) shows the development of tensile strength and stress in a typical uncut concrete pavement, and the cracking will occur around 40 hours when the stress in the pavement reaches the tensile strength. Because of the stress concentration around the notch tip, the introduction of a saw
cut reduces the stress that is required to cause cracking in a pavement. The strength reduction factor is introduced by dividing the stress at the age of failure by the tensile strength of concrete based on the principle of fracture mechanics. Figure 2.35(b) illustrates the stress and tensile strength development for the same pavement. However, when a saw cut with a depth of one third of the pavement thickness is made 6 hours after the concrete pavement, the cracking will occur after about 20 hours, which is earlier as compared to the pavement without sawcut. Figure 2.36 illustrates the calculated the strength reduction factors for different saw cut depth ratios. It should be noted that the principle of the strength reduction factor is only applicable when the saw cut size is larger than a critical value that is believed to depend on the used aggregate size (Weiss 1999).

![Figure 2.35](image1.png)

(a)

![Figure 2.35](image2.png)

(b)

Figure 2.35 Schematic view of stress and strength development of (a) uncut pavement; (b) pavement with saw cut of one third of depth of pavement slab, placed at 6 hours after concrete placement, after Raoufi et al. (2008).
Figure 2.36 The effect of saw cut to depth ratio on the strength reduction factor, after Raoufi et al. (2008).

Houben (2008a); Houben (2008b) proposes a model to describe the cracking process in no-jointed plain concrete pavement due to the temperature and moisture-induced loading. It enables to determine the time of occurrence of cracks and its crack width as a function of the pavement structure, material properties, and the time of construction. However, only time of occurrence of the first series of cracks can be obtained for CRCP by using Houben’s model. The most of the required material properties in Houben’s calculation are estimated according to the standards, and the temperature variation of the concrete pavement is produced by very limited historical climate values.

2.5 SUMMARY

Based on the literature review, the following can be summarised:

- Field investigations of recently constructed CRCPs under the current design concept in Belgium indicate that the crack pattern is characterized as low mean crack spacing along with a quite high percentage of clusters of closely spaced cracks. Besides, it is difficult to reduce the probability of those non-uniform cracks by slightly adjusting the amount of longitudinal reinforcement steel. Field surveys have also indicated that these undesirable cracks can lead to the development of punchouts. However, it should be noted that CRCPs built under the current design concept in Belgium behave excellently and are barely subjected to deterioration, mainly because of the good support condition. However, occasional punchouts occur and therefore more research is needed to obtain the ‘ideal’ crack pattern of CRCP.

- Field surveys have shown that the PA/CRCP composite pavements generally have provided good performance in the Netherlands. However, some reflec-
tive cracking was commonly found in the PA overlay after 5 to 10 years. Those reflective cracks are believed to be attributed to the crack pattern in the CRCP slab that is characterized as large mean crack spacing along with a high percentage of extremely long slab segments. It is thought to be caused by the low percentage of the longitudinal reinforcement and the isolating effect of the asphalt overlay.

- The crack pattern of CRCP is found to be significantly influenced by the concrete pavement temperature development during the hardening phase. An excessive temperature may result in reduced performance in the long-term of CRCP. The heat flux on the top boundary condition and the heat hydration of cement has a dominant effect on the concrete temperature under field condition. Slag cement and the plastic sheeting curing method are commonly used in the CRCP construction in Belgium. However, the current available temperature models need improvements to estimate the temperature accurately when the slag cement and plastic sheeting curing method are adopted.

- Previous experiences on crack control method for CRCP have shown that it can significantly improve the crack pattern and thus result in better long-term performance. The early age surface shallow saw cut along the whole width has proven to be a practical solution for crack control to CRCP. However, more research is required to determine the appropriate saw cut timing and the saw cut geometry for Belgian conditions.
This chapter presents the overview of the field and laboratory experimental programs, including the field instrumentation and data collection, material, and laboratory test plan.

3.1 OVERVIEW OF FIELD SECTIONS

The field experiments include two CRCP test sections in Belgium. The first test section was constructed on the motorway E17 near De Pinte, in August 2011, and the other test section was constructed on the motorway E313 near Herentals, in September 2012. The objective of the De Pinte test section on E17 is to evaluate the effect of the percentage of longitudinal reinforcement on the transverse crack pattern of CRCP. The test section on E313 near Herentals is to identify the effect of an active crack control method to improve the transverse crack pattern.

3.1.1 E17, De Pinte

The test section near De Pinte is a part of an 11 km long stretch of the motorway E17 that was reconstructed in mid-2011. In 2011, the Flemish Road Authorities decided to increase the concrete pavement design thickness from 230 to 250 mm for the highest traffic class. This change was immediately applied for the continuation of the worksite on the E17. Initially it was considered to reduce the steel reinforcement percentage to 0.70%. Finally, however, a more conservative approach was followed by maintaining the percentage 0.75%. It was mainly because there still was some doubt about the correctness of the crack width measurements since different results were obtained depending on the depth of the measurement. This worksite also included two trial sections: one with a steel reinforcement content of 0.70% and another with a reinforcement content of 0.65% plus 20 kg/m³ of steel fibers.
Within this stretch, the 250 mm thick concrete slab was applied upon an asphalt interlayer (50 mm), a roller compacted concrete base layer (150 mm) and a sub-base of unbound aggregates obtained through crushing the old lean concrete, as shown in Figure 3.1. The old jointed plain concrete pavement was recycled into the new base and sub-base of the CRCP. The longitudinal reinforcement steel (BE 500 S) consists of bars with diameter 20 mm spaced at 170 mm, 180 mm, 190 mm for the sections with longitudinal reinforcement percentages of 0.75%, 0.70% and 0.65%, respectively. The position of the longitudinal steel reinforcement is 80 ± 10 mm (the centre of the bars) below the pavement surface. The transverse reinforcement (BE 500 S) consists of bars with diameter 12 mm. They are supported by steel chairs spaced at 0.70 m, which are placed on the asphalt interlayer. The transverse reinforcing bars are placed at an angle of 60 degrees to the longitudinal steel.

![Figure 3.1](image1.png)

**Figure 3.1** The existing jointed plain concrete pavement (left) and new CRCP (Mu et al.) on E17, De Pinte.

![Figure 3.2](image2.png)

**Figure 3.2** CRCP Test section on E17, De Pinte, Belgium.
3.1.2  **E313, Herentals**

Inspired by an interesting finding during a field inspection of CRCP roundabouts in Belgium, Rens and Beeldens (2013) found some transverse (radial) cracks that looked like induced by the contraction joint of the adjacent inner circle of the roundabout. Besides, based on American experiences of the shallow sawcut method in active crack control for CRCP, Rens proposed a new active crack control procedure for CRCP that was firstly applied in the reconstruction project of motorway E313.

The reconstruction project E313 near Herentals was conducted in September 2012. The test sections on E313 were constructed according to the current standard CRCP practice in Belgium: 250 mm thick CRCP slab laid on a 50 mm asphalt interlayer and a 200 mm lean concrete base. The longitudinal reinforcement steel amounts 0.75%, and the position of the longitudinal steel reinforcement is 90 mm (the centre of the bars) below the pavement surface. Besides, due to the noise reduction requirement, two-lift construction was adopted for the concrete slab, the thickness of the top and the bottom layer is 50 mm and 200 mm, respectively. During hardening of the concrete partial surface notches were sawn at the outer side of the pavement slab, the length of the notch is 400 mm, and the spacing is 1.20 m, as shown in Figure 3.3. Initially, the notch depth was 30 mm and later it was increased to 60 mm.

![Figure 3.3 Active crack control section in E313, Herentals, 2012: (a) at the inner lane; (b) at the emergency lane.](image-url)

3.1.3  **CRCP Construction in Belgium**

Figure 3.4 illustrates several features of CRCP design and construction practices in Belgium. The use of the asphalt interlayer is believed to reduce the potential of punchout development according to the Belgium historical experiences on CRCP, as shown in Figure 3.4(a). In the majority of the rehabilitation projects, also on E17 and E313, concrete is placed without interruption, 24/24 h, 7 days a week, due to the very tight execution schedule, as seen in Figure 3.4(b). It also prevents the daily...
Construction joints. Extra width of the concrete slab is commonly used to avoid the edge stresses, and concrete gutters are applied along both edges of the pavement to improve the drainage of water and thus reduces the potential of the erosion of the base layer under traffic loading, as shown in Figure 3.4(c). Reduction of traffic noise is becoming increasingly important in Belgium and therefore the so-called exposed aggregate surface is nowadays commonly used in Belgium. Immediately after pouring of the concrete, a liquid retarder (which retards the concrete hardening) is spread at the concrete, which then has to be covered, both at the top and at the sides of the pavement, with a plastic sheet to prevent drying out of the fresh concrete, as seen in Figure 3.4(d). Figure 3.4(e) and Figure 3.4(f) show the exposed aggregate surface with a maximum aggregate size of 20 mm on E17, and with a maximum aggregate size of 6.3 mm on E313, respectively.

Figure 3.4 CRCP construction practice in Belgium.
3.2 MATERIALS

Table 3.1 shows the concrete mixture proportions used on the E17 and E313. The typical one layer system for concrete pavement in Belgium is used in the test section on E17. Two-lift construction is used on the E313 test section.

- **One layer concrete**

Exposed aggregate concrete 0/20 is applied on the E17 section. Porphyry aggregate with 3-grain size ranges are utilized: 4-6.3 mm, 6.3-14 mm, and 14-20 mm. The amount of the 4-6.3 mm aggregate is at least 20% of the total granular mix (sand and coarse aggregate). Figure 3.5(a) shows the ‘reference’ aggregate gradation curve used for typical concrete pavement mixtures in Belgium for maximum aggregate sizes of 20 with the exposed aggregate surface treatment (Ployaert and van Audenhove 2010). The Fuller gradation curves (n=0.5) based on the corresponding maximum aggregate sizes for the typical paved concrete mixtures in Belgium are illustrated in Figure 3.5 as well.

- **Two-lift construction**

Two-lift construction is used on the E313 test section. The bottom layer with a thickness of 200 mm is made of limestone aggregate with a maximum grain size of 32 mm. The fractions were 2-6.3 mm (172 kg/m³), 6.3-20 mm (575 kg/m³), and 20-31.5 mm (402 kg/m³). The ‘reference’ aggregate gradation curve for the maximum aggregate size of 32 mm is illustrated in Figure 3.5(b). The top layer with a thickness of 50 mm consists of a fine-grained concrete that is placed wet on wet with slipform pavers. The fine-grained concrete is made of only one fraction of coarse aggregate, a porphyry aggregate of 4-6.3 mm. Figure 3.5(c) shows the ‘reference’ aggregate gradation curve used for the concrete mixture of the top layer with the exposed aggregate surface treatment. Although, exposed aggregate concrete with fine aggregates in the top layer in this two-lift construction pavement is a little more expensive (±10%) than conventionally exposed aggregate concrete 0/20. However, two-lift construction for CRCP provides good friction, durability and noise reduction (Hendrickx 2006).

Table 3.1 Concrete Mixture Composition

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>E17</th>
<th>E313 Top</th>
<th>E313 Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>kg/m³</td>
<td>400</td>
<td>425</td>
<td>375</td>
</tr>
<tr>
<td>Cement type</td>
<td>--</td>
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<td>III/A 42.5N/</td>
<td>III/A 42.5N/</td>
</tr>
<tr>
<td>Water</td>
<td>kg/m³</td>
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<td>175</td>
<td>165</td>
</tr>
<tr>
<td>Water to cement</td>
<td>--</td>
<td>0.43</td>
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<td>0.44</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>kg/m³</td>
<td>1331</td>
<td>1030</td>
<td>1149</td>
</tr>
<tr>
<td>Coarse aggregate type</td>
<td>--</td>
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<td>porphyry</td>
<td>limestone</td>
</tr>
<tr>
<td>Fine aggregate</td>
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<td>474</td>
<td>664</td>
<td>726</td>
</tr>
<tr>
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<td>0</td>
</tr>
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<td>1.28</td>
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<td>Air content</td>
<td>%</td>
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<td>3</td>
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</table>
During the initial phase of this study, a number of specimens were casted in the concrete plant of the reconstruction project E17 and tested to characterize the fundamental concrete mechanical properties. In the sequent phase, the concrete specimens for the uniaxial tensile tests were casted in the laboratory using the collected aggregate fractions from the concrete plant of the reconstruction project E313. In order to obtain the same gradation curves of the concrete used in E313, the three limestone aggregate fractions of the bottom layer were further sieved into 7 fractions, 22.4-31.5 mm, 16.0-22.4 mm, 11.2-16.0 mm, 8.0-11.2 mm, 5.6-8.0 mm, 4.0-5.6 mm, 2.0-4.0 mm. The Table 3.2 lists the sieving results of each fraction of aggregates used on E313. Finally, the actual applied gradation curves in E313, as defined as the ‘measured’ curves in Figure 3.5, were then calculated according to the weights and the corresponding particle size distribution of each fraction. This ‘measured’ gradation curves in Figure 3.5 were used for laboratory prepared specimens.

Table 3.2 Particle size distribution of each fraction of the aggregates and sands used in E313.

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>20.0-31.5 limestone</th>
<th>6.3-20.0 limestone</th>
<th>2.0-6.3 limestone</th>
<th>4.0-6.3 porphyry</th>
<th>0-4 sand</th>
<th>0-2 sand</th>
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<td>63.0</td>
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Table 3.3 Chemical composition and Fineness of CEM III/A 42.5N/LA.

<table>
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<th>Parameter</th>
<th>Percentage by mass (%)</th>
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</tr>
<tr>
<td>Al₂O₃</td>
<td>8.0</td>
</tr>
<tr>
<td>MgO</td>
<td>3.9</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>2.8</td>
</tr>
<tr>
<td>Blaine fineness (m²/kg)</td>
<td>460</td>
</tr>
</tbody>
</table>
CEM III/A 42.5 N/LA, produced by Holcim, with a limited alkali content to prevent alkali-aggregate reaction, are used on E17 and E313. It contains 36% to 65% granulated blast furnace slag. The oxide composition and the fineness of the CEM III/A 42.5 N/LA are given in Table 3.3 (Holcim 2011).

![Gradation curves of concrete mixtures for motorways in Belgium](image)

Figure 3.5 The gradation curves of the concrete mixtures for motorways in Belgium as a function of the used maximum aggregate size together with the exposed aggregate surface treatment.
3.3 FIELD INSTRUMENTATION AND DATA COLLECTION

3.3.1 Field Concrete Temperature Measurement

Typical low cost type K thermocouples have been used to measure the concrete temperature during the first few days after concrete placement. To provide a vertical temperature distribution throughout the pavement slab, a set of thermocouples have been installed prior to concrete paving to measure concrete temperature at different depths within the concrete slab. To place the thermocouples at the desired locations, a steel rod was driven into the asphalt interlayer and the thermocouples were then attached on the steel rod, as shown in Figure 3.6(a). Being afraid of the top thermocouple being damaged by the spreading auger of the splitform paver during construction on E17, the top thermocouple was located at 50 mm below the pavement surface as suggested by the contractor. The other temperature measurement locations are at the depth of longitudinal reinforcement (100 mm below the pavement surface), and 25 mm above the surface of the asphalt interlayer, as indicated in Figure 3.6(b). The cables of the embedded thermocouples were lead through a groove in the asphalt interlayer to the verge of the road, as shown in Figure 3.6(c). In the test section of E313, the upper thermocouple is installed at 75 mm below the pavement surface because of the two-lift construction method applied in this project, and the other locations of the installed thermocouples on E313 are 100 mm (the location of the longitudinal rebars), 175 mm, and 225 mm below the pavement surface.

Figure 3.6 Thermocouple installation in CRCP.

The temperatures were manually collected by a digital temperature gauge, Kane-May 330, at every 30 minutes during the first 24 hours after concrete placement, and afterwards every 2 hours until the first few days. Figure 3.7 gives the recorded concrete temperatures at different depths during hardening.
Figure 3.7 Measured concrete temperature during the first few days after concrete placement for E17 and E313.
3.3.2 Crack Pattern Survey

Regularly crack pattern surveys, including crack width and crack spacing, were performed to evaluate the crack pattern progression on E17 and E313 to investigate the effect of various percentages of longitudinal reinforcement and the active crack control method. Table 3.4 summarizes the crack pattern surveys for both projects. Table 3.5 lists the location and the length of the surveyed sections on E17 and E313.

Table 3.4 Summary of time of each survey for E17 and E313, respectively.

<table>
<thead>
<tr>
<th>Road</th>
<th>Time</th>
<th>Crack Pattern Survey</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Date</td>
<td>1st</td>
</tr>
<tr>
<td>E17</td>
<td>18-22/08/11</td>
<td>18/10/11</td>
</tr>
<tr>
<td>E313 60mm</td>
<td>11-15/09/12</td>
<td>15-11-12</td>
</tr>
<tr>
<td>E313 30mm</td>
<td>–</td>
<td>15-11-12</td>
</tr>
</tbody>
</table>

Table 3.5 Summary of surveyed sections on E17 and E313, respectively.

<table>
<thead>
<tr>
<th>Road</th>
<th>Location</th>
<th>Length (m)</th>
<th>Percentage of longitudinal reinforcement (%)</th>
<th>Active crack control</th>
</tr>
</thead>
<tbody>
<tr>
<td>E17</td>
<td>km 44.7-km 45.2</td>
<td>500</td>
<td>0.75%</td>
<td>Not</td>
</tr>
<tr>
<td></td>
<td>km 45.2-km 46.2</td>
<td>1000</td>
<td>0.70%</td>
<td>Not</td>
</tr>
<tr>
<td></td>
<td>km 46.7-km 46.7</td>
<td>500</td>
<td>0.65%</td>
<td>Not</td>
</tr>
<tr>
<td>E313</td>
<td>km 24.0-km 22.9</td>
<td>1100</td>
<td>0.75%</td>
<td>60 mm depth</td>
</tr>
<tr>
<td></td>
<td>km 25.0-km 24.5</td>
<td>500</td>
<td>0.75%</td>
<td>30 mm depth</td>
</tr>
</tbody>
</table>

The crack spacing survey was conducted by slowly walking along the edge of the pavement (emergency lane), and record the location, the shape of cracks, and define the category of each crack as shown in Figure 3.8. In this study, different devices have been adopted to evaluate the crack width on the pavement surface. It includes the optical microscope, digital microscope, and image analysis, as shown in Figure 3.9.

Figure 3.8 Crack spacing survey conducted by using a distance measurement wheel.
3.4 LABORATORY TESTS

During the initial phase of this study, a number of specimens were casted at the concrete plant of the reconstruction project E17 and tested to characterize the fundamental concrete mechanical properties. In the sequent phase, the concrete specimens for isothermal calorimetry tests and uniaxial tensile tests were casted in the laboratory using the collected aggregate fractions from the concrete plant of the reconstruction project E313.

3.4.1 Compressive Strength, Modulus, and Split Tensile Strength

The following tests, including compressive strength, modulus of elasticity, split tensile strength and concrete coefficient of thermal expansion, were performed for the hardened concrete that was casted in the field of the E17 reconstruction project.

At the concrete plant, six concrete slabs with dimensions of 150*240*500 mm, and nine prisms with dimensions of 100*100*400 mm were cast on August 18, 2011, using the same concrete as in the test sections of E17, as illustrated in Figure 3.10. The concrete specimens were prepared according to NEN-EN 12350-1 (2009). After one day of curing at the concrete plant, the casted slabs and prisms were transported to Delft University of Technology. The specimens were demoulded and stored in a curing room with a constant temperature around 25 °C and 100% humidity until the time of testing. Compressive strength, modulus of elasticity, and split tensile strengths tests were performed at the age of 7, 28, and 90 days. Cylindrical specimens with a diameter of 100 mm and a height of 200 mm were used for those tests. At each test age, six cylinder specimens were drilled from the individual field casted slabs, and were ground and sawn to the required tolerances of the specimen.
size of NEN-EN 12390-1 (2000). Three specimens were tested for the compressive strength, modulus of elasticity, and split tensile strength at each age. Among those, three specimens were first loaded to one third of the ultimate compressive load to determine the static modulus of elasticity under compression according to ASTM C469 (2010), and they were then used for the measurement of the compressive strength according to NEN-EN 12390-3 (2009). The specimen and the test set-up are shown in Figure 3.11. Another three specimens were tested for the split tensile strength according to NEN-EN 12390-6 (2009). The test set-up for the compressive strength and split tensile strength are shown in Figure 3.12.

Figure 3.10 Field casted specimens at E17, 18/08/2011.

Figure 3.11 Experimental set-up for static modulus of elasticity in compression.
Table 3.6 summarizes the measured test results obtained for the concrete mixtures of E17, i.e. the average value and the standard deviation. High cement contents, low water-cement ratios, and the use of air entraining agent yields a very durable, high-strength concrete in Belgium CRCP roads.

Table 3.6 Measured mechanical properties of concrete mixtures used in E17.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Age of the concrete (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>38.85 (±0.52)</td>
</tr>
<tr>
<td>Split tensile strength (MPa)</td>
<td>3.54 (±0.11)</td>
</tr>
<tr>
<td>Elastic Modulus (MPa)</td>
<td>36750 (±3182)</td>
</tr>
</tbody>
</table>

3.4.2 Shrinkage

The unrestrained shrinkage tests of the concrete mixtures used in E17 and E313 were conducted in accordance with the method of Xuan (2012). The length changes that are produced in the absence of externally applied forces and temperature changes in concrete specimens, which exposed to controlled conditions of temperature and moisture, are measured using dial gauges. The test specimens for the E17 mixture were casted in the field and the specimens for the E313 mixture were prepared in the laboratory. The dimension of the test specimen is 100*100*400 mm and casted by steel mould. After casting, all specimens were covered by plastic sheet and stored in room condition, and were demoulded at age of 1 day. After that, 4 curing conditions were chosen to investigate the influence of the curing conditions: no further sealing by aluminium foil (here defined as CR1), sealing for 3 days (CR2), sealing for 7 days (CR3), and sealing for the whole duration of the measurement (CR4). The specimens were kept undisturbed for the
entire duration of the test in a room at 20±2 °C and 50±5% relative humidity. Actually, the curing condition CR1 is close to the Belgian CRCP curing procedure in practice with plastic sheet and curing compound.

The dial gauge with an accuracy of 0.01 mm was used to measure the length change between two fixed points at a distance of 300 mm. A pair of stainless steel clamps was glued on the center of the specimen for the connection of gauges and a stainless steel bar, as shown in Figure 3.13. Two dial gauges were subsequently installed on the opposite sides of each specimen. The shrinkage specimens were kept undisturbed for the entire duration of the test. After the initial readings were made, the length of the specimen was manually recorded at the specified time during 1 year. The measurements frequency was twice per day during the first week, and it was subsequently reduced to twice per week in the later age. It should be noted that the shrinkage measured by this method is the average shrinkage of the concrete specimens.

![Figure 3.13 Sealed and unsealed drying shrinkage measurement specimens.](image)

Figure 3.14 shows the measured shrinkage for the specimens made of the E313. As indicated in Figure 3.14, the largest shrinkage is obtained under the curing condition of CR2 as expected. Besides, it also demonstrates that a longer period of the specimen under the sealed condition leads to a lower drying shrinkage for the early age concrete. There exist many models associated with predicting shrinkage of concrete, such as ACI 209R-92 (ACI Committee 209 1992), B3 Model (RILEM 1995), CEB MC90-99 (fib 1999) and Eurocode 2 (Eurocode 2 2004). Figure 3.15 presents the experimental results of the unrestrained shrinkage under the sealed condition that is similar to the drying condition of the field-cured concrete in a concrete pavement, for the three types of concrete mixture applied on E17 and E313. The shrinkage prediction model in Eurocode 2, including the autogenous shrinkage and drying shrinkage, is used for the prediction of the average shrinkage of the whole cross section of the specimen as a function of time,
composition of concrete mixtures, and drying conditions. Figure 3.15 shows that the Eurocode 2 model can well catch the characteristics of the development of shrinkage of concrete at early age for both non-sealed and sealed drying conditions. The following parameters are used: mean compressive strength of 50 MPa, normal type of cement, notional size of the cross section of the sample is 50 mm, and the end of the plastic sheet curing is chosen as the age of the concrete at the beginning of the drying shrinkage. It should be noted that the specimen sealed by foil does not mean that it was completely without moisture loss. The simulation results show that the drying condition with 85% relative humidity matches well the specimen sealed by aluminium foil.

Figure 3.14 The measured unrestrained shrinkage for the mixture of E313 top layers as a function of various curing conditions.

Figure 3.15 Comparison of shrinkage between Eurocode 2 model and test results for concrete mixtures used in E17 and E313.
3.4.3 Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) of hardened concrete of E17 is determined by measuring the change of the length of the specimen when changing the temperature from 1 °C to 50 °C (Xuan 2012). Three field casted concrete prisms with dimensions of 100*100*400 mm were stored in a room at 20 °C and 100% relative humidity for 2 years. Those three specimens were then dried at room conditions about one week prior to the CTE measurement. The tested specimen was cooled in a temperature chamber at 1 °C during 12 hours. Meanwhile, the LVDT gauges and their connection bars were pre-installed in the adjacent chamber having a constant temperature of 50 °C. The cooled specimen was quickly moved to the temperature chamber of 50 °C and the LVDT gauges were installed within 30 seconds. The length change of the specimen was recorded by a data acquisition system with a frequency of 1 minute and continuously measured for at least 6 hours. Three duplicated tests were performed for each specimen. As shown in Figure 3.16, it is observed that the measured displacement of the specimen becomes constant after about 3 hours, when the concrete specimen reaches thermal equilibrium with the internal environment of the chamber. The CTE is calculated through the measured mean displacement (after 6 hours) divided by the temperature difference (49 °C) and the height of the specimen (400 mm).

![Figure 3.16 Repeatability of CTE tests performed on the same specimen by a given operator at different times.](image)

Table 3.7 summarizes the measured CTEs of the hardened concrete specimens made of the mixture of E17. A value of (10.6±0.6)*10^-6 is found for the typical paving concrete in Belgium made of porphyry aggregate.
Table 3.7 Measured coefficient of thermal expansion (CTE) of hardened concrete mixture of E17.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test cycle 1</th>
<th>Test cycle 2</th>
<th>Test cycle 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>CTE-1</td>
<td>11.3</td>
<td>10.4</td>
<td>10.0</td>
</tr>
<tr>
<td>CTE-2</td>
<td>11.0</td>
<td>11.5</td>
<td>10.3</td>
</tr>
<tr>
<td>CTE-3</td>
<td>10.0</td>
<td>10.4</td>
<td>--</td>
</tr>
</tbody>
</table>

3.4.4 **Isothermal Conduction Calorimetry Tests**

The cement hydration heat curve is a fundamental input for the prediction of the pavement strength and stresses during the early age. The isothermal calorimetry test results are used as input data for the proposed concrete temperature and stress analysis of the early age CRCP. The isothermal calorimeter, TAM Air 3114/3236, manufactured by Thermometric Inc. (see Figure 3.17) was used in this study. The calorimeter is located in a room with a temperature of 20±2°C and 50±5% relative humidity. The heat of cement hydration is directly measured by monitoring the heat flow from the specimen. This calorimeter contains eight separate channels. The isothermal calorimetry tests were performed for paste samples to study the hydration behaviour of the blended cement containing granulated blast furnace slag (ASTM C1702 2009). Test conditions and test results of the isothermal calorimetry tests are further discussed in Chapter 4.

Figure 3.17 Isothermal calorimeter TAM Air 3144/3236.

3.4.5 **Uniaxial Tensile Test**

In case of a saw cut is made in a concrete pavement, the classical strength of material approach fails to capture the stress concentration that develops at the tip of the saw cut. The cracking behaviour of concrete pavements can therefore be better described using the principle of fracture mechanics. In the present study, a
deformation controlled uniaxial tensile test on unnotched specimens is used to obtain the evolution of the fracture energy during hardening, as shown in Figure 3.18 (Erkens 2002). The uniaxial tensile test will be addressed in detail in Chapter 5.

Figure 3.18 The TU Delft Uniaxial tensile test set up.
Early Age Concrete Pavement Temperature

Prediction of the pavement temperature at early age is a good starting point to understand the behaviour of CRCP. Fluctuations in temperature produce expansion and contraction movements in a concrete pavement, and when they are restrained, they lead to the development of stresses and possibly cracking, which may significantly affect the pavement’s early age and long-term performance (Suh et al. 1992; Schindler et al. 2002). As described previously in Chapter 2, numerous numerical models are available to predict the concrete pavement temperature distribution. However, those models are limited to ordinary Portland cement concrete. The use of blended slag cement has attracted much attention because of the low cost along with many engineering advantages, such as low heat release, low permeability and good durability (Glasser 1991). For instance, more than 50% of the cement delivered to the market in the Netherlands is CEM III, a mixture of Portland cement clinker with blast furnace slag (Chen 2006). In Belgium, the cement used for concrete pavements is either Portland cement (CEM I) or a blast furnace slag cement (CEM III). Thus, a heat of hydration model valid for the blended slag cement is required in the proposed temperature prediction model for the Belgium CRCP conditions. Besides, some improvements are also required in the treatment of the heat flux models at the boundary conditions and the determination of the climatic inputs. The chapter describes in detail the development of an early age concrete pavement temperature prediction model that enables to simulate the use of blended slag cement and the plastic sheet curing method. This proposed model is then verified with field-measured data of two projects in Belgium. Lastly, the approaches to generate the required climatic data for the
Chapter 4 Early Age Concrete Pavement Temperature

temperature prediction in the construction phase (real-time) by using limited weather forecasting climate data is presented.

4.1 EARLY AGE CONCRETE PAVEMENT TEMPERATURE PREDICTION MODEL

In the proposed early age concrete pavement temperature prediction model, the following steps are considered: heat transfer governing equation, rate of heat hydration, boundary and initial conditions, numerical implementation, and the required inputs.

4.1.1 Heat Transfer Governing Equation

The geometry of a concrete pavement is characterized as relatively large in the transverse direction (width) compared to the vertical direction (slab thickness), and therefore it is reasonable to assume that the concrete temperature only varies in the vertical direction. Field measurements have also shown that the temperature variation in the width direction is not significant except for that at the very edge surface (Nam 2005). Thus, a one dimensional temperature prediction model is adequate to predict the temperature distribution within a concrete pavement. The non-stationary heat conduction problem of the thermal behaviour of concrete during hardening can be described by the well-known Fourier equation (Narasimhan 1999):

\[ \rho c \frac{\partial T}{\partial t} = \lambda \frac{\partial^2 T}{\partial x^2} + q(x, t) \] (4.1)

Where,
- \( c \) = specific heat capacity of the concrete, [J/kg/°C];
- \( \rho \) = density of the concrete, [kg/m³];
- \( \lambda \) = thermal conductivity of the concrete, [W/m/°C];
- \( T \) = concrete temperature, [°C];
- \( t \) = time, [s];
- \( x \) = distance from the pavement surface, [m];
- \( q(x, t) \) = rate of heat of cement hydration at location \( x \) at time \( t \), [W/m³].

Incorporating the known boundary and initial conditions, the time-dependent temperature distribution inside the concrete pavement can be obtained through solving Equation (4.1).

4.1.2 Rate of Cement Heat Hydration

The used cement type is the dominant influence on the concrete pavement temperature during the early age. As mentioned previously, blended slag cement is very often used for the concrete pavements in Europe, especially, in Belgium. To simulate more adequately the early age temperature in the concrete pavement, a
general hydration model proposed by De Schutter and Taerwe (1995). This model has been verified by temperature measurements on hardening massive concrete elements made with blended cement. The description of this hydration model and the determination of the corresponding parameters are presented in the following sections.

**Isothermal Calorimeter Test**
A commercial isothermal calorimeter TAM air manufactured by Thermometric Inc. is used in the present study. TAM air is an eight-channel isothermal heat conduction calorimeter and can measure the heat flow in the mill watt range. All channels are of the twin type, consisting of a sample and a reference vessel, each with a volume of 20 ml. The mixing water, cement, vessels, and the mixing tools were kept at a temperature-controlled room with a constant temperature of 20°C prior to mixing, in where the TAM air device was also placed. The cement paste was mixed externally of the calorimeter during 30 seconds, and then the required amount of paste, 10 g is adopted in this study, was transferred to the testing vial. Lastly, the vial was then placed into the calorimeter for data collection. The readings were recorded every 60 seconds for a period of 7 days by an automated data acquisition program. The recorded heat flow caused by the cement hydration was converted from the measured voltage to the liberated heat by the cement hydration of the cement in the sample. Two blended–granulated blast furnace slag cement, CEM III 42.5N/LA from Holcim and CEM III 42.5N from ENCI with four w/c ratios (0.41, 0.43, 0.45, and 0.50) were tested in the present study, which are commonly used for the mixtures for concrete pavements in Belgium and the Netherlands. A typical heat evolution curve for a paste mixture made of the CEM type III/A obtained by this isothermal calorimetry test is illustrated in Figure 4.1. The measured original data of the rate of heat, after being normalized by the cement weight, is expressed in J/g/h.

![Figure 4.1 Isothermal calorimetry test of Blast furnace slag cement at 20 °C.](image-url)
**Pre-processed data**

There exists a peak at the initial stage as shown in Figure 4.1, which is partly due to the rapid heat evolution of the reaction between Tricalcium Silicate and gypsum (Byfors 1980), and partly attributed to the temperature equilibrium at the test setup (Wang et al. 2007). The latter part is the measurement error instead of the real material behaviour and it is difficult to account for. However, the contribution of the integrated measured first peak only accounts for a relative small percentage of $Q_{\text{max}}$. What’s more, as in practice concrete is not cast immediately after mixing with water, the corresponding amount of heat of the first peak can never be produced inside the concrete structure (De Schutter and Taerwe 1995). Thus, the first peak is removed from the heat computation, as shown in Figure 4.2. A straight line is used to connect the origin to the valley point after the first peak, and the chosen valley point in the present study is the value corresponding to 2 hours after the addition of water to the concrete mixture.

![Figure 4.2 Correction of initial isothermal calorimetry test result of Blast furnace slag cement.](image)

**Maximum potential of heat $Q_{\text{max}}$**

Isothermal conduction calorimetry tests with a testing duration of 3 or 7 days to obtain the heat of hydration of Blended slag cement have been conducted by many researchers (Xiong and van Breugel 2001; Wang et al. 2006; Castro and Rey 2011; Sedaghat et al. 2012). Pane and Hansen (2005) performed isothermal conduction calorimetry tests up to 21 days. Pane and Hansen consider that the heat data up to 21 days are still not measured long enough to allow for an accurate determination of the total amount of heat due to the complete reaction. But, Wang et al. (2009) observed that the heat of hydration up to 7 days shows a larger variation as compared to that for 3 days in the isothermal calorimetry tests. Sedaghat et al. (2012) have recently pointed out that the noise of the signal of the calorimeter is approximately 0.008 mW for the common commercial conduction calorimeters at a
measurement age of 7 days, while the heat signal from the cement paste sample is just an order of magnitude higher. Longer hydration times such as 28 days are therefore not necessary. Moreover, ASTM C1702-09, ‘Standard test method for measurement of heat of hydration of hydraulic cementitious materials using isothermal conduction calorimetry’, specifies a procedure for determining the total heat of hydration of cementitious materials up to 7 days.

As mentioned previously, the isothermal conduction calorimetry tests were performed up to 7 days in the present study. In order to obtain the maximum potential heat of hydration $Q_{\text{max}}$ under the isothermal condition, the contribution of the heat generation after 7 days is estimated by linear extrapolation utilizing the measured data between 5 to 7 days, as shown in Figure 4.1. Table 4.1 summarizes the cumulative heat evolution at several ages. A comparable heat of hydration is observed for the two types of cement under the isothermal condition. Moreover, the sample with the lowest w/c ratio exhibits a higher rate of heat evolution before the peak value. After that, the sample with the higher w/c ratio has the larger rate of heat evolution. This is consistent with previous studies (Wang et al. 2007). With regard to the CEM III/A 42.5N/LA that is commonly used in concrete pavements in Belgium, the cumulative heat of hydration up to 7 days is about 290 J/g, for the w/c ratio ranging from 0.41 to 0.44. Moreover, the corresponding maximum potential of heat hydration $Q_{\text{max}}$ is around 310 J/g. This is consistent with the results by De Schutter (1999) for the CEM type III.

<table>
<thead>
<tr>
<th>Cement</th>
<th>w/c</th>
<th>2</th>
<th>12</th>
<th>24</th>
<th>48</th>
<th>72</th>
<th>168</th>
<th>$\infty$</th>
</tr>
</thead>
<tbody>
<tr>
<td>III/A</td>
<td>0.41</td>
<td>21.7</td>
<td>59.7</td>
<td>134.6</td>
<td>203.8</td>
<td>231.9</td>
<td>289.4</td>
<td>305.5</td>
</tr>
<tr>
<td>42.5N/LA</td>
<td>0.43</td>
<td>21.8</td>
<td>58.1</td>
<td>131.2</td>
<td>201.9</td>
<td>231.3</td>
<td>292.4</td>
<td>309.7</td>
</tr>
<tr>
<td></td>
<td>0.44</td>
<td>24.3</td>
<td>59.1</td>
<td>130.9</td>
<td>200.9</td>
<td>229.1</td>
<td>286.6</td>
<td>311.2</td>
</tr>
<tr>
<td></td>
<td>0.50</td>
<td>24.2</td>
<td>55.8</td>
<td>123.9</td>
<td>200.1</td>
<td>230.7</td>
<td>300.6</td>
<td>340.8</td>
</tr>
<tr>
<td>III/A</td>
<td>0.41</td>
<td>25.4</td>
<td>86.9</td>
<td>149.5</td>
<td>221.9</td>
<td>255.0</td>
<td>302.1</td>
<td>322.3</td>
</tr>
<tr>
<td>42.5N</td>
<td>0.43</td>
<td>25.6</td>
<td>84.7</td>
<td>147.9</td>
<td>220.7</td>
<td>254.5</td>
<td>303.7</td>
<td>320.5</td>
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<td>0.44</td>
<td>24.7</td>
<td>85.2</td>
<td>149.7</td>
<td>223.9</td>
<td>257.4</td>
<td>306.2</td>
<td>317.7</td>
</tr>
<tr>
<td></td>
<td>0.50</td>
<td>24.4</td>
<td>84.1</td>
<td>150.5</td>
<td>226.7</td>
<td>260.5</td>
<td>312.5</td>
<td>331.8</td>
</tr>
</tbody>
</table>

**Hydration model for Granulated Blast Furnace Slag Cement**

The hydration of GBFS cement is more complex than that of Portland cement because its two constituents, Portland cement clinker and GBFS, hydrate at different rates and their reactions interfere with each other. Slag is activated by the alkalis and calcium hydroxide released by Portland cement hydration, and consumes a large amount of calcium hydroxide as well (Roy 1982). Furthermore, Roy (1982) has observed that those two reactions have different temperature factors, where the slag reaction is more sensitive than that of Portland cement.
clinker. In contrast to Portland cement, a single peak curve function, such as the common adopted FHP degree of hydration model based on exponential formulation, for describing the heat generation rate during the hardening phase does not exist for GBFS cement, as shown in Figure 4.1. A hydration model based on isothermal conduction calorimetry tests proposed by De Schutter and Taerwe (1995) has been shown to simulate the degree of hydration for the type of Blended GBFS cement quite well. Thus, this hydration model by De Schutter and Taerwe (1995) is adopted in the present study and is described as follows:

Because of the different temperature sensitivity and inconsistent initiation of the Portland reaction (P-reaction) and Slag reaction (S-reaction), the measured curve of the total heat production rate $q(t)$ under the isothermal condition is separated into two individual curves for both reactions based on the superposition principle, as shown in Figure 4.3.

$$q(t) = q_p(t) + q_s(t)$$

(4.2)

Where,

- $q_p$ = heat production rate of the P-reaction, [J/g/h];
- $q_s$ = heat production rate of the S-reaction, [J/g/h].

![Figure 4.3: Superposition of P-reaction and S-reaction for GBFS cement.](image)

Subsequently, the parameters of the maximum heat of hydration $Q_{max}$, $Q_{P, max}$ and $Q_{S, max}$ at the end of the reaction can be estimated by integrating the heat production curves for the total of P-reaction and S-reaction, respectively. The corresponding degree of both reactions $r_p(t)$ and $r_s(t)$ defined as the fraction of the heat hydration that has been released, can then be calculated as follows:

$$r_p(t) = \frac{Q_p(t)}{Q_{P, max}} = \frac{1}{Q_{P, max}} \int_0^t q_p(t)dt$$

(4.3)
\[ r_s(t) = \frac{Q_s(t)}{Q_{s,\text{max}}} = \frac{1}{Q_{s,\text{max}}} \int_0^t q_s(t) \, dt \quad (4.4) \]

A standardized curve of \( q_p((t))/q_{p,\text{max},20} \) for the P-reaction is obtained by transforming the time axis of \( t \) into the above-calculated degree of reaction \( r_p(t) \), as shown in Figure 4.4. The obtained curves can be described as follows:

\[ \frac{q_p(r_p(t))}{q_{p,\text{max},20}} = f_p(r_p(t)) = c_p \cdot [\sin(r_p(t)\pi)]^{a_p} \cdot \exp(-b_p r_p(t)) \cdot r_p \quad (4.5) \]

Where,

- \( q_{p,\text{max},20} \) = maximum heat production rate of the P-reaction at 20°C, \([\text{J/g/h}]\);
- \( r_p \) = degree of reaction of the P-reaction, [-];
- \( a_p, b_p, c_p \) = regression constants, [-].

![Figure 4.4 Standardized curves of P-reaction at 20 °C for a paste sample with w/c=0.43 made of CEM III/A 42.5N/LA.](image)

The constants of \( a_p, b_p, c_p \) in Equation (4.5) are determined by curve fitting using the standardized curve as shown in Figure 4.4. De Schutter (1999) explained that the sinusoidal function in Equation (4.5) accounts for the maximum value of the heat production rate \( q_{p,\text{max},20} \). The exponential part of Equation (4.5) results from the general Guldberg and Waage law describing the effect of the concentration of reagents on the rate of corrosion. De Schutter (1995) observed that the standardized curves of \( q_p(t)/q_{p,\text{max}} \) as function of the corresponding degree of reactions have a quite similar shape for different isothermal temperatures, and the maximum heat production rate occurs at the same degree of reaction for different test temperatures of the same material. De Schutter further concluded that the temperature influence on the heat production rate seems to be completely independent on the degree of reaction. The primary concept to understanding this model is that at the specific degree of reaction \( r_p(t) \) (in other words, the specific released heat state \( Q_p(t) \)), the standardized rate of heat generation \( q_p(t)/q_{p,\text{max}} \) is
identical no matter at what temperature state \( T \) it is. This has been experimentally indicated by Reinhardt et al. (1982) and Wadsö (2003). Lastly, the well-known Arrhenius function is adopted to describe the temperature effects on the magnitude of the maximum heat production rate under various isothermal conditions.

\[
q_{p, \text{max}, T} = q_{p, \text{max}, 20} \cdot \exp\left[\frac{E_p}{R} \left(\frac{1}{293} - \frac{1}{273 + T}\right)\right] \tag{4.6}
\]

Whereas, \( E_p \) is the apparent energy of the P-reaction. It can be determined by the measured maximum heat production rate through several calorimetry tests under different isothermal temperature conditions. Combination of Equation (4.5) and Equation (4.6) yields the hydration model for the P-reaction of the Blended cement as a function of the actual temperature \( T \) and the degree of the reaction \( r_p(t) \):

\[
q_p(r_p(t), T) = q_{p, \text{max}, 20} \cdot c_p \cdot [\sin(r_p(t)\pi)]^{a_p} \cdot \exp(-b_pr_p(t)) \\
\cdot \exp\left[\frac{E_p}{R} \left(\frac{1}{293} - \frac{1}{273 + T}\right)\right] \tag{4.7}
\]

Similarly, the standardized curve of \( q_s(t) / q_{s, \text{max}, 20} \) for S-reaction is also plotted as a function of its degree of reaction, as shown in Figure 4.5. It is clearly shown that the curve for the S-reaction is more symmetric which indicates the S-reaction ends rapidly. De Schutter (1999) interprets this phenomenon that the alkalis and lime released by the residual Portland cement are retrained in the hydration products of the slag fraction and do not seem to contribute to the hydration of slag after a certain hardening time. The following formulations are adopted to describe the standardized curve of the S-reaction:

\[
\frac{q_s(r_s(t))}{q_{s, \text{max}, 20}} = f_s(r_s(t)) = [\sin(r_s(t)\pi)]^{a_s} \tag{4.8}
\]

\[
q_{s, \text{max}, T} = q_{s, \text{max}, 20} \cdot \exp\left[\frac{E_s}{R} \left(\frac{1}{293} - \frac{1}{273 + T}\right)\right] \tag{4.9}
\]

Where, \( E_s \) is the apparent energy of S-reaction, and \( q_{s, \text{max}, 20} \) is the maximum heat generation rate of S-reaction at the temperature of 20°C. The constant \( a_s \) in Equation (4.8) can be determined by curve fitting using the standardized curve of S-reaction as shown in Figure 4.5. Again, combination of Equation (4.8) and Equation (4.9) yields the hydration model for the S-reaction of the Blended cement as a function of the actual temperature \( T \) and the degree of the reaction \( r_s(t) \):

\[
q_s(r_s(t), T) = q_{s, \text{max}, 20} \cdot [\sin(r_s(t)\pi)]^{a_s} \cdot \exp\left[\frac{E_s}{R} \left(\frac{1}{293} - \frac{1}{273 + T}\right)\right] \tag{4.10}
\]
Then, the overall heat production rate \( q(t, T) \) of Blended cement hydration under field temperature condition can be expressed as follows:

\[
q(t, T) = q_P(r_P(t), T) + q_S(r_S(t), T)
\]  

(4.11)

As shown in Figure 4.3, the S-reaction is activated when sufficient alkalis and calcium hydroxide is released by the Portland cement hydration. In other words, the S-reaction does not start immediately with water addition. This is implemented mathematically by keeping the second term of Equation (4.11) to zero as long as the degree of reaction of P-reaction has not yet reached a threshold value \( r_{P,B} \). The parameter \( r_{P,B} \) is determined by performing several isothermal calorimetry tests with different temperature for the same mixture. A value of 0.3635 for \( r_{P,B} \) is observed for the CEM III/A 42.5 N/LA at 20°C. Moreover, \( r_{P,B} \) is also found to be temperature dependent.

The values of the parameters of this hydration model obtained through curve fitting with the results of the isothermal conduction calorimetry tests at 20°C are summarized in Table 4.2 for CEM III/A 42.5N/LA that is often used in Belgium CRCP roads. Lastly, it should be noted that this hydration model by De Schutter is explicitly based on the fraction of the heat released. As shown in Equation (4.5) and (4.8), a very small initial value for \( r_P \) and \( r_S \), instead of zero, should be initialized in the numerical simulation procedure. Moreover, with an appropriate value for the initial degree of P-reaction \( r_{P,i} \), the induction period of the P-reaction can be simulated accurately. These initial values are also model parameters that can be changed to simulate, for example, the effect of retarding agents. De Schutter recommended a value of 0.0001 to both \( r_{P,i} \) and \( r_{S,i} \) for CEM III without additives. In the present study, an initial value of \( r_{P,i} \) is chosen as 0.01 at the concrete placement time that represents the degree of the P-reaction developed during the period of concrete delivery from the concrete plant to the worksite. The calculated
parameters for the De Schutter’s hydration model for another two types of blended cement in Belgium are also tabulated in Table 4.2 (De Schutter 1996).

<table>
<thead>
<tr>
<th>Parameters for De Schutter’s hydration model for blended slag cement.</th>
<th>CEM III/A 42.5N/LA</th>
<th>CEM III/B 32.5</th>
<th>CEM III/C 32.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>a_p (-)</td>
<td>0.892</td>
<td>0.667</td>
<td>0.500</td>
</tr>
<tr>
<td>b_p (-)</td>
<td>4.003</td>
<td>3.500</td>
<td>2.500</td>
</tr>
<tr>
<td>c_p (-)</td>
<td>3.518</td>
<td>2.846</td>
<td>2.143</td>
</tr>
<tr>
<td>q_p,max,20 (J/g/h)</td>
<td>6.68</td>
<td>4.80</td>
<td>1.42</td>
</tr>
<tr>
<td>Q_p,max (J/g)</td>
<td>285.6</td>
<td>251.0</td>
<td>167.0</td>
</tr>
<tr>
<td>a_s (-)</td>
<td>0.617</td>
<td>0.667</td>
<td>0.667</td>
</tr>
<tr>
<td>q_s,max,20 (J/g/h)</td>
<td>1.49</td>
<td>1.06</td>
<td>0.62</td>
</tr>
<tr>
<td>Q_s,max (J/g)</td>
<td>6.35</td>
<td>11.3</td>
<td>32.5</td>
</tr>
<tr>
<td>r_p_B (-)</td>
<td>0.3635</td>
<td>0.3793</td>
<td>0.3200</td>
</tr>
</tbody>
</table>

**Degree of hydration**

For Portland cement, a certain ultimate degree of hydration $\alpha_u$, which is the degree of hydration actually reached in practice (in other words corresponding to the end of the Portland cement reaction $r_p = 1$), depends on the w/c ratio of the concrete (Mills 1966). During the development of HIPERPAV II, Schindler et al. (2002) proposed the following ultimate degree of hydration model also valid for blended slag cement.

$$\alpha_u = \frac{1.031w/c}{0.194 + w/c} + 0.3P_{slag} \leq 1.0$$ (4.12)

Where, w/c is the water to total cementitious materials ratios, and $P_{slag}$ is the mass ratio replacement of the GBFS slag. The calibration ranges of cement compositions for Equation (4.12) are well documented in the relevant literature (Schindler et al. 2002). With this ultimate degree of hydration model by Schindler, the degree of the hydration of the Blended slag cement is calculated as follows:

$$\alpha(t) = \alpha_u \cdot \frac{Q(t)}{Q_{max}} = \alpha_u \cdot \frac{1}{Q_{max}} \int_0^t q(t)dt$$ (4.13)

Figure 4.6 plots the calculated degree of hydration based on the measured heat evolution curve with an isothermal test at 20°C for a paste sample with w/c=0.43 made of CEM III/A 42.5N/LA according to the estimated ultimate degree of hydration based on Equation (4.13). As a comparison, the hydration parameters of the FHP hydration model are also calculated ($\tau = 23.23$ hour, and $\beta = 1.11$) and the corresponding predicted degree of hydration at 20°C is presented in Figure 4.6. It is clearly shown that both the De Schutter model and FHP model agree well with the measured values in the acceleration stage and the deceleration stage of the hydration process (the definition of those stages is adopted from Byfors, 1980).
They all underestimate the degree of hydration at the steady stage. However, during the dormant stage, the FHP model once again underestimates the calculated degree of hydration according to the measured data, whereas the De Schutter model agrees quite well with the measured data. The discrepancies are even clearer when the predicted heat production rates of these two models at 20°C are compared with the measured heat evolution curve, as shown in Figure 4.7. The predicted heat generation rate according to the De Schutter model shows a good agreement with the measured values from the dormant stage to the deceleration stage, and the second heat evolution peak due to the S-reaction is included. However, the predicted curve based on the FHP model shows a quite large discrepancy at all hydration stages and cannot simulate the second peak of heat of hydration of the S-reaction.

Figure 4.6 Predicted and measured degree of hydration at 20°C for a paste sample with w/c=0.43 made of CEM III/A, 42.5N/LA.

Figure 4.7 Predicted and measured heat production rate at 20 °C for a paste sample with w/c=0.43 made of CEM III/A, 42.5N/LA.
Activation Energy

The activation energy $E_P$ and $E_s$ in the De Schutter hydration model define the temperature sensitivity of the heat production rate. In order to obtain $E_P$ and $E_s$, at least two isothermal conduction calorimetry tests for different test temperatures are required for the same material. The measured peak values of the heat production rate are utilized for computing the activation energy based on the Arrhenius Equation. Detailed descriptions of the determination of activation energy by isothermal tests are well documented by (Schindler et al. 2002; Wang et al. 2007). When no laboratory tests are available, a value of 48804 J/mol for both $E_P$ and $E_s$ is recommended by RILEM (1997) for slag cement. Figure 4.8 illustrates the sensitivity of the activation energy on the heat evolution rate and degree of hydration under different temperatures.

![Heat Production Rate and Degree of Hydration](image)

Figure 4.8 Sensitivity of activation energy on the heat production rate and the degree of hydration.
4.1.3 Boundary Conditions
Heat flux at the concrete pavement surface consists of the convection, irradiation, solar absorption, curing methods etc., as shown in Figure 4.9. Each heat transfer mechanisms and the selected models are presented and discussed in detail in the following sections.

![Heat transfer mechanisms between pavement and its surroundings](after Schindler, et al. 2002)

4.1.4 Heat Convection
Convection is heat transfer by mass of motion of a fluid, such as air or water, when the heated fluid is caused to move away from the source of heat, carrying energy with it. According to the heat transfer theory, heat convection consists of two parts: free and forced convections. Free convection is caused by air movement due to the density gradient while forced convection is the heat transfer between a moving fluid and the pavement surface. Generally, Newton’s law of cooling is used to express the heat convection \( q_{\text{conv}} \) at the surface (ASHRAE Handbook 1993):

\[
q_{\text{conv}} = h_{\text{conv}}(T_a - T_s)
\]  

(4.14)

Where,
\begin{align*}
q_{\text{conv}} & \quad \text{heat convection transfer coefficient, [W/m}^2]; \\
h_{\text{conv}} & \quad \text{heat convection transfer coefficient, [W/m}^2/{^\circ C}]; \\
T_a & \quad \text{air temperature, [°C]}; \\
T_s & \quad \text{temperature at the pavement surface, [°C]};
\end{align*}

Knowledge of the heat convection coefficient is crucial for an accurate determination of the concrete temperature development as this heat change can be 3 to 4 times higher than the longwave radiative heat exchange (Pinto and Hover 1999). An extensive review of convective heat transfer coefficient \( h_{\text{conv}} \) models imple-
mented in building energy simulation programs was well documented by Mirsadeghi et al. (2013). The convective heat transfer coefficient is influenced by several factors, such as the wind speed, surface to air temperature differences, pavement surface roughness, the geometry of the pavement slab. Considering the complexity involved in obtaining $h_{\text{conv}}$, empirical models based on both reduced-scale and full-scale tests are commonly adopted in building energy programs and concrete temperature prediction models, such as EICM, HIPERPAV, FEMMASSE, TMAC$_2$, etc. The specific convective heat transfer coefficient prediction model implemented in the above mentioned programs are briefly presented as follows, with special attention for the relevant experimental conditions behind those empirical models that are very often not clearly stated.

**McAdams Model**

FEMMASSE, HIPERPAV I, and TMAC$_2$ have all adopted a simple convective heat transfer coefficient prediction model which is proposed by McAdams (1954), as shown in Equation (4.15). This model is also the model most common used by the researchers in the field of concrete (van Breugel et al. 1998; Bentz 2000). This expression is based on wind tunnel experiments conducted by Jurges (1924) who heated a 0.5 m × 0.5 m smooth copper plate mounted vertically and flushed with airflow parallel to the copper plate.

$$h_{\text{conv}} = \begin{cases} 5.6 + 4.0 \cdot \frac{v_{\text{wind}}}{m} & \text{for } v_{\text{wind}} \leq 5 \text{ m/s} \\ 7.2 \cdot \frac{v_{\text{wind}}^{0.78}}{m} & \text{for } v_{\text{wind}} > 5 \text{ m/s} \end{cases} \quad (4.15)$$

Where, $v_{\text{wind}}$ is the average hourly wind speed in m/s. Moreover, the original expression of the model of $h_{\text{conv}}$ proposed by McAdams is as follows:

$$h_{\text{conv}} = 5.678 \left[ m + n \left( \frac{v_{\text{wind}}}{0.3048} \right)^p \right] \quad (4.16)$$

<table>
<thead>
<tr>
<th>Surface type</th>
<th>$v_{\text{wind}} &lt; 4.88 \text{ m/s}$</th>
<th>$4.88 \text{m/s} \leq v_{\text{wind}} &lt; 30.48 \text{m/s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth</td>
<td>$m$ 0.99 $n$ 0.21 $p$ 1</td>
<td>$m$ 0 $n$ 0.50 $p$ 0.78</td>
</tr>
<tr>
<td>Rough</td>
<td>$m$ 1.09 $n$ 0.23 $p$ 1</td>
<td>$m$ 0 $n$ 0.53 $p$ 0.78</td>
</tr>
</tbody>
</table>

Where $m$, $n$, $p$ are the roughness parameters for smooth and rough surface, and the range of the wind speed is up to 30 m/s as given in Table 4.3. It can be found that Equation 4.15 is derived based on the smooth surface condition. However, the criterion that should be used to classify a surface as either smooth or rough is not clear. Moreover, the calculated $h_{\text{conv}}$ by Equation (4.16) is about 6-10% higher for rough surfaces than for smooth surfaces, for wind speeds from 0 to 15 m/s. Mirsadeghi et al. (2013) state that the $h_{\text{conv}}$ model by McAdams seems logical, as in most others, that it does not include the effect of the longwave radiation. Whereas, Watmuff et al. (1977) regard that McAdams’ equation for $h_{\text{conv}}$ may include the
longwave radiation effects, and thus overestimates the value of the convective heat transfer coefficients. Figure 4.10 summarizes the calculated $h_{\text{conv}}$ for the current commonly used models in the field of concrete structures as a function of wind speed. As shown in Figure 4.10, McAdams’ model gives the largest value of the calculated convective heat transfer coefficients, especially in the strong wind condition.

Contrary to the above-mentioned model based on laboratory tests on a vertical plate, Test et al. (1981) measured the convective heat transfer coefficient through a horizontal plate of 1.22 m$x$ 0.81 m placed in the natural environment that is considered as more suitable to the pavement condition. Wind speed in this test was measured at 1 m above the plate. They proposed the following equation to calculate $h_{\text{conv}}$:

$$h_{\text{conv}} = 8.55 + 2.56 \cdot v_{\text{wind}}$$

(4.17)

![Figure 4.10 Comparison of different convective heat transfer coefficients for two temperature conditions; the first value in the bracket is the air temperature, and the second one is the surface temperature, in °C.](image)

Similar tests conducted by Lombaard and Kroger (2001) on a horizontal plate of 1 m $\times$ 1 m in the field condition gave values from 1.9 to 2.9 for the correlation parameter of wind speed. Kroger (2002) concluded that the value for the correlation parameter of wind speed appears to lie in the range of 1.9 to 3.3 for horizontal plates exposed to the natural environment. Those values are lower than the corresponding parameter of McAdams’ equation. The parameter 8.55 in Equation (4.17) may be considered as the free convection component, and is influenced by
the difference between the surface temperature to the air temperature and the size of the plate.

**EICM Model**

The heat convection coefficient prediction model adopted in the EICM model is based on the results of a field experiment conducted by Vehrencamp (1953). He obtained the convective heat transfer coefficient through a heat meter placed on the ground, and the heat meter plate had a disc shape with a relative small diameter of 110 mm. With the measured air temperature and the wind speed at the height of 2 m, Vehrencamp proposed the following equation to calculate $h_{conv}$ for the convective heat transfer between the air and the ground surface, including both free and forced convection (adapted for the SI units).

$$h_{conv} = 698.24 \cdot [0.00144 \left( \frac{T_s + T_a}{2} + 273 \right)^{0.3} \cdot v_{wind}^{0.7} + 0.00097 (T_s - T_a)^{0.5}]$$  \hspace{1cm} (4.18)

It should be noted that the formulation of $h_{conv}$ by Vehrencamp may be the local convective heat transfer coefficient for the laminar flow condition due to the relative small dimension of the measured plate. According to the heat convection theory, airflow over a large plate or concrete slab is not laminar but turbulent, and the average convective heat transfer coefficient for the turbulent flow condition is much larger than that of laminar flow. As shown in Figure 4.10, the Vehrencamp’s Equation adopted in EICM gives the lowest value of convective heat transfer coefficients and is insensitive to the air to surface temperature difference. More especially, the heat convection coefficient $h_{conv}$ calculated by Equation (4.18) gives an extreme low value when the wind velocity is zero. This calculated negligible heat convection coefficient goes against the accepted heat transfer theory of a free heat convection coefficient ranging from 5 to 7 W/m² for an infinite horizontal plate under normal environmental conditions (Jiji 2006). The adoption of the EICM heat convection model would over-predict the pavement temperature.

**HIPERPAV II Model**

Based on the fundamentals of heat transfer and mass transfer by Incropera and Dewitt (1990), a convective heat transfer coefficient model used in ASTM C680 and ASHRAE Handbook 1993 is available for the determination of $h_{conv}$ of a smooth horizontal plate, including both free and forced convection. HIPERPAV II further simplified the heat convection model and added the effect of surface roughness (ASHRAE Handbook 1993; Schindler et al. 2004). The $h_{conv}$ model adopted in HIPERPAV II is presented as follows:
\[ h_{\text{conv}} = 3.727 \cdot C \cdot (0.9 \cdot (T_s + T_a) + 32)^{-0.181} \cdot (T_s - T_a)^{0.266} \cdot \sqrt{1 + 2.857 \cdot v_{\text{wind}}} \]  

(4.19)

Where, \( C \) is a constant correction factor depending on the heat flow condition, it is chosen as 1.79 when the pavement surface is warmer than the air, and 0.89 when the pavement surface is cooler than the air. As shown in Figure 4.10, the calculated \( h_{\text{conv}} \) by HIPERPAV II gives a moderate value for the whole range of wind speeds from 0 to 10 m/s. Moreover, it agrees quite well with the results by Test et al. for a horizontal plate placed in the natural environment condition that is similar to that for a concrete pavement. Meanwhile, the HIPERPAV II \( h_{\text{conv}} \) model takes into account the influence of the air to surface temperature difference, and a larger heat convection coefficient is found for the warmer temperature condition. In view of the reliable prediction results, uncomplicated formulation, and only a few and easy accessible input parameters needed, it is therefore adequate to use the Equation (4.19) to calculate \( h_{\text{conv}} \) in the concrete temperature prediction model developed in the present study.

**Wind Correction Factor**

As indicated above, the wind speed is the most dominant factor influencing the convective heat transfer coefficients and it is quite common for current concrete temperature prediction models to use the recorded wind speed data from the standard weather stations. The wind speed data reported from the standard meteorological station are generally measured 10 m above the terrain according to the guidelines of WMO (World Meteorological Organization), EPA (United States Environmental Protection Agency), KNMI (Royal Netherlands Meteorological Institute), and RMI (Royal Meteorological Institute of Belgium) etc. As discussed previously, empirical \( h_{\text{conv}} \) prediction models derived from field tests (Vehrencamp, 1953; Test et al., 1981; Kroger, 2001 and 2002) are correlated with wind speed measured at the height of 1~2 m above the ground, while the \( h_{\text{conv}} \) models obtained from laboratory tests are generally expressed with the uniform free stream wind flow. Moreover, the data of air temperature and relative humidity obtained from the standard weather stations are generally measured at a height of 1.5 to 2.0 m above the terrain, for instance, 1.5 m for KNMI and 2.0 m for RMI. It is therefore necessary to convert the wind speed measured at 10 m height above the ground to that of 2.0 m height where also the air temperature is measured. Several wind correction factors have been used in a number of building energy simulation programs to calculate the heat convection coefficients (Mirsadeghi et al. 2013). However, to the best of my knowledge, this has not yet been done for the current available pavement temperature prediction models. The wind speed correction model implemented in the Thermal Analysis Research Program is adopted in the present study, and is expressed as follows (Walton 1983):
\[ v_{\text{wind}} = V_{10} \cdot \beta_{\text{wind}} \cdot (z_{\text{wind}}/10)^{\alpha_{\text{wind}}} \]  

(4.20)

Where,

- \( v_{10} \) = wind speed measured at 10 m height above the ground, [m/s];
- \( z_{\text{wind}} \) = the interested height above the surface, here chosen as 2 m;
- \( \alpha_{\text{wind}}, \beta_{\text{wind}} \) = the terrain roughness coefficients that are given in Table 4.4.

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
<th>( \alpha_{\text{wind}} )</th>
<th>( \beta_{\text{wind}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ocean or water at least 5 km of unrestricted extension</td>
<td>0.10</td>
<td>1.30</td>
</tr>
<tr>
<td>2</td>
<td>Flat terrain with some isolated obstacles</td>
<td>0.15</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>Rural areas with low buildings, trees, etc.</td>
<td>0.20</td>
<td>0.85</td>
</tr>
<tr>
<td>4</td>
<td>Urban, industrial, or forest area</td>
<td>0.25</td>
<td>0.67</td>
</tr>
<tr>
<td>5</td>
<td>Centre of large city</td>
<td>0.35</td>
<td>0.47</td>
</tr>
</tbody>
</table>

Table 4.4 Terrain roughness coefficients (after Walton, 1983).

Figure 4.11 illustrates the correction factor for the wind speed measured at 10 m height of the standard weather stations, as a function of different terrain roughness. When the pavement is located in a rural area with low buildings and trees, the wind speed at 2.0 m above the pavement is 62% of that reported from the weather stations.

Figure 4.11 Correction factors for the wind speed to the data measured at 10 m height with different terrain roughness conditions.

### 4.1.5 Heat Irradiation

Thermal irradiation is a longwave heat transfer between the ground and the sky. The Stefan-Boltzmann law is commonly used for this type of heat transfer, which is defined as follows (McAdams 1954):
\[
q_{ir} = \varepsilon_p \cdot \sigma \cdot (T_{sky}^4 - T_s^4)
\] (4.21)

Where,
\( q_{ir} \) = thermal irradiation from the surface, [W/m²];
\( \varepsilon_p \) = surface emissivity of concrete, [-];
\( \sigma \) = Stefan-Boltzmann constant, \(5.669 \times 10^{-8}\) Wm⁻²K⁻⁴;
\( T_{sky} \) = effective sky temperature, [K].

The surface emissivity \( \varepsilon_p \) is a function of the concrete’s surface color. An ‘idealized’ black surface would have a value of 1.0. A value ranging from 0.88 to 0.90 is recommended when evaluating concrete pavement temperatures (Kapila et al. 1997; Ruiz et al. 2006). The effective sky temperature \( T_{sky} \) is the temperature of the surrounding environment, and is not equal to the ambient air temperature, and it is influenced by the dew point, water vapour pressure, sky cloud cover, etc. There are also several methods to calculate the effective sky temperature (Swinbank 1963; Berdahl and Fromberg 1982; Centeno 1982; Walton 1983; Ruiz et al. 2006; Qin 2011). Among those, the effective sky temperature in HIPERPAV II is evaluated by a series of complex formulas with several uncertain empirical parameters. For the sake of simplicity and clearance, the formula in HIPERPAV to compute the thermal irradiation at a pavement surface is not recommended. A simplified model proposed by Walton (1983) to calculate the effective sky temperature based on the dew point temperature, ambient temperature and the cloud cover is chosen in this study because the required inputs for the calculation of the effective temperature are available from meteorological stations. Moreover, Walton’s model to calculate the effective sky temperature has been successfully used in a variety of predictive models relevant for the construction community (Walton 1983; Zarr 1998; Bentz 2000). The effective sky temperature model proposed by Walton is expressed as:

\[
T_{sky} = \varepsilon_{sky}^{0.25} \cdot T_a
\] (4.22)

\[
\varepsilon_{sky} = 0.787 + 0.764 \cdot \ln\left(\frac{T_{dp} + 273}{273}\right) \cdot F_{cloud}
\] (4.23)

The cloud cover factor \( F_{cloud} \) is defined as follows:

\[
F_{cloud} = 1.0 + 0.024N - 0.0035N^2 + 0.00028N^3
\] (4.24)

Where, \( N \) is the cloud cover, taking values between 0.0 and 1.0. However, the cloud cover factor \( N \) is a highly uncertain parameter and normally not reported in meteorological sites. Moreover, as shown in Equation (4.23), the effect of the cloud cover on the thermal irradiation can be neglected due to its minimal effect on the effective sky emissivity \( \varepsilon_{sky} \), as \( F_{cloud} \) varies between 1.00 and 1.02. Thus, the cloud cover factor is not considered in the present study. Figure 4.12 presents the results of the calculated thermal irradiation for pavements constructed under Belgium
winter and summer conditions. The ‘negative’ sign indicates the net heat flux through the thermal irradiation between the pavement surface and its surroundings is outgoing for the pavement surface. The maximum temperature difference between pavement surface and surrounding environment can be up to 20 °C during daytime in the curing period, which leads to a considerable amount of heat flux through irradiation.

![Thermal Irradiation versus Temperature Difference](image)

Figure 4.12 Thermal irradiation versus temperature difference between pavement surface and air for both winter and summer pavement construction under Belgium condition.

### 4.1.6 Solar Absorption

Next to the heat of hydration, the solar absorption is the most important heat source to the pavement structure and should therefore be accurately described (Ruiz et al, 2006). McCullough and Rasmussen (1999) proposed the following formula to calculate the acquired shortwave radiation on the pavement surface $q_{sol}$:

\[ q_{sol} = \gamma_{abs} \cdot q_{ins} \]  

(4.25)

Where,

- $q_{sol}$ = solar absorption heat flux, [W/m²];
- $\gamma_{abs}$ = solar surface absorptivity, [-];
- $q_{ins}$ = instantaneous solar radiation, [W/m²].

During the verification phase of the pavement temperature prediction model, the solar radiation was taken directly from the weather data files. The solar absorptivity $\gamma_{abs}$ of Portland cement concrete is a function of the colour of the surface. An ideal white body would have a value of 0.0, and an ideal black body would have a value of 1.0. The solar absorptivity of a concrete pavement is generally determined by measuring the solar reflectance of the pavement surface, so-called albedo. There are two ASTM standard test methods for determining the solar reflectance of a surface: ASTM C1459 (2014) ‘Standard test method for
determination of solar reflectance near ambient temperature using portable solar reflectometer’ and ASTM E1918 (2015) ‘Standard test method for measuring solar reflectance of horizontal and low sloped surface in the field’. Levinson and Akbari (2002) investigated the effects of composition and exposure conditions (soiling, wetting, and abrasion) on the solar reflectance of 32 Portland cement concrete mixtures. They found the solar reflectance generally became more reflective in the early stage of curing, stabilizing after several weeks. Wetting the surface of concrete made it less reflective. The albedo of mature concrete ranging from 0.41 and 0.77 with a mean value of 0.59 was reported by Levinson and Akbari (2002) for the smooth surface. McCullough and Rasmussen (1999) recommended typical values ranging from 0.5 to 0.6 for new and older concrete, respectively. In addition, the application of white curing compound reduces the solar absorptivity by 0.1 to 0.35. A solar absorptivity of 0.50 was found most appropriate for concrete pavements cured with white curing compound according to several field temperature measurements (Schindler et al. 2002).

4.1.7 Initial Conditions and Boundary Conditions

As shown in Equation (4.1), the early age concrete pavement temperature prediction is a non-stationary heat conduction problem. The temperature profile at the initial time and the heat flux balance at the boundary nodes are needed to solve the above-mentioned equations.

**Pavement Surface**

The thermal balance equation at the pavement surface consists of the foregoing heat fluxes through the convection $q_{conv}$, radiation $q_{ir}$, absorption $q_{sol}$, the heat conduction within the pavement surface and the pavement layers beneath, and the energy stored at the pavement surface. In the case of the hardening concrete, the heat generation rate of the cement hydration at the top surface layer $q_{c,s}$ should be also included. Therefore, the thermal balance equation used in the present study is expressed by:

\[
q_{conv} + q_{ir} + q_{sol} + q_{c,s} - \lambda_c \frac{\partial T}{\partial x}_{surface} = \delta_{surface} \cdot \rho_c \cdot c_c \cdot \frac{\partial T}{\partial t}
\]  

(4.26)

Where $\lambda_c$, $\rho_c$, and $c_c$ are the thermal conductivity, density, and heat capacity of the top concrete layer, respectively. $\delta_{surface}$ is the assumed thickness of the pavement surface. Such a thermal balance equation at the pavement surface has been used successfully by other researchers (Gui et al. 2007; Han et al. 2011; Alavi et al. 2014).

**Pavement Bottom**

With increasing depth within the pavement, the temperature variation decreases and becomes less influenced by the fluctuation of the temperature at the surface. In the previous concrete pavement temperature prediction models, it is traditionally assumed that the pavement temperature reaches to a constant value at a certain depth (Hermansson 2001). The depth of zero annual temperature change was
reported approximately from 9 to 18 m below the ground surface by Lytton et al. (1993) or only 5 m used by Hermansson (2001). Such a depth of constant temperature is not always known and it is specific to the location of the pavement. Recently, Han et al. (2011) suggested a constant minimal heat flux at the depth of 3 m based on the LTPP pavement temperature database. It is consistent with an experiment to measure temperature at various depth of the pavement by Gui et al. (2007). They found that the temperature at the depth of 3 m has a minimal effect on the prediction of temperatures at the near surface layers. This value of 3 m is suggested as the location of the constant annual temperature for the bottom boundary condition in this study.

**Initial Temperature Profile**

The initial temperature distribution within the layers underlying the future pavement slab is obtained as follows. Firstly, a linear temperature distribution is assumed, where the temperature of the upper boundary is chosen as the mean monthly air temperature and the temperature of the bottom layer is taken as the annual average air temperature. Then, the past 7 days climate data from the nearest weather station is used as input values to calculate the temperature profiles. Eventually, a temperature distribution within the substructure at the end of the 7 days is obtained, which will be adopted as initial temperature distribution in the subsequent concrete pavement temperature prediction.

Figure 4.13 illustrates the calculated daily variations of the base temperature profiles on the E17 section, constructed on August 19, 2011. Due to high solar radiation absorptivity of the upmost asphalt interlayer, a quite large temperature difference at the base layer surface, nearly 17 °C is observed. It indicates the importance of adequately defining the temperature of the base layers. Furthermore,
Figure 4.13 shows that the daily base temperature variation decreases as the depth increases, and it becomes relative constant when the depth is larger than about 0.6 m. It is attributed to the lower heat conductivity of the gravel sub-base and the sand subgrade.

4.1.8 Curing methods with Polyethylene Sheeting

The liquid curing compound, which is the most common curing method for concrete pavement construction, is used to protect the pavement concrete against drying out. Recently, the polyethylene sheeting together with the curing compound is commonly used in the Belgium concrete pavement construction practice. Polyethylene sheeting is very beneficial in retaining moisture of hardening concrete and thus minimizes the drying shrinkage. Polyethylene sheeting also acts as a thermal insulator as the use of insulation materials reduces the heat flux at the pavement surface. However, it can be detrimental if used improperly. For instance, the colour of polyethylene sheeting is considered as the most critical variable that significantly increases the maximum concrete temperature and zero-stress temperature in a concrete pavement (Schindler et al. 2002).

Existing models

CIMS (1988), computer interactive maturity system, included some regression equations for the convective heat transfer coefficients to represent the heat loss by various insulation methods according to the experiments results. These equations were derived by a best-fit curve in terms of wind velocity by using the least squares method, and those equations were implemented in the program CIMS. In case of the curing method by polyethylene sheeting, the following regression equation is proposed (adapted for the SI units):

\[ h_{\text{conv,ps},\text{CIMS}} = -0.0040v_{\text{wind}}^2 + 0.5156v_{\text{wind}} + 5.1461 \] (4.27)

Where, the range of \( v_{\text{wind}} \) is from 0 to 10 m/s.

An overall heat transfer coefficient proposed by McAdams (1954) to evaluate the convective heat transfer coefficients due to the presence of different insulation materials is adopted in HIPERPAV II.

\[ h_0 = \left( \frac{1}{h_{\text{conv}}} + \frac{d_1}{k_1} + \frac{d_2}{k_2} + \cdots + \frac{d_n}{k_n} \right)^{-1} \] (4.28)

Where,

- \( h_0 \) = the overall heat convection coefficient, [W/m\(^2\)/°C];
- \( d_1, d_2, \cdots, d_n \) = thickness of n successive insulation layers, [m];
- \( k_1, k_2, \cdots, k_n \) = thermal conductivity of n successive insulation layers, [W/m°C].
Figure 4.14 Comparison of heat convection models with plastic sheeting.

The thermal characteristics of various insulation materials were well summarized by Schindler et al. (2002). In both projects of E17 and E313, clear polyethylene sheeting with a thickness of 0.15 mm was used immediately after the concrete placement. The thermal conductivity of the polyethylene sheeting is chosen as 0.043 W/m°C according to ASHRAE 1993. Figure 4.14 shows the calculated convective heat transfer coefficients for cases with and without the polyethylene sheeting curing method. The method by HIPERPAV significantly overestimates the convective heat transfer coefficient with polyethylene sheeting as compared to that by the regression equation in CIMS. It indicates that the treatment of heat flux for a pavement cured with polyethylene sheeting through the principle of overall heat transfer coefficient is inadequate, which thus leads to underestimation of the pavement temperature. This problem has been noticed by the program developers of HIPERPAV when they verified the HIPERPAV model with field measurement temperature data (Ruiz et al. 2006). They found that using the real thickness of the plastic sheet would significantly underestimate the amount of heat retained by the pavement in case of the plastic sheet curing method. They interpreted that as follows: when a plastic sheet is used, it is not in full contact with the slab, and the air voids between concrete and sheet act as additional insulation. Finally, they recommended using an additional air void thickness of 5 mm in addition to the plastic sheet in those cases where a plastic sheet is used. The air thermal conductivity at 20 °C is about 0.0257 W/m/K. As shown in Figure 4.14, the estimated heat convection coefficients by the calibrated model in HIPERPAV is close to the CIMS model that is in agreement with field measurements.

However, the calibrated overall heat convection model in HIPERPAV and the regression model in CIMS only account for the effect of a plastic sheet on the convective heat transfer, while its effects on the radiation are neglected. However, the radiation heat flux on the pavement surface alters as well when plastic sheet is used, because the plastic sheet in general has different reflectance, transmittance, and absorbance for both shortwave radiation from the sun (0.2 to 1.2 µm) and the longwave radiation (2 to 50 µm) originating from the pavement surface and
surrounding environment. For instance, it is well known that transparent polyethylene sheeting is widely used to increase the soil temperature in the agricultural field. The first part is the so-called greenhouse effect of the cover of the polyethylene sheeting through the reduction of heat losses by long wave radiation, especially when intensive cooling takes place during the night. Another effect of polyethylene sheeting is to reduce the evaporation, and thereby reduce the heat flux through the pavement surface. Thus, in case a plastic sheet is applied, a more fundamental heat transfer model initially proposed by Mahrer (1979) is adopted in the present study and it is briefly described as follows.

**Mahrer Model**
Mahrer (1979); Mahrer (1980); Mahrer et al. (1984); Ham and Kluitenberg (1994); Wu et al. (1996) have proposed one dimensional soil temperature models to predict the temperature of bare and mulched soil. In case of the pavement covered by polyethylene sheeting, the net heat flux at a pavement surface is determined as the sum of the net radiation flux, heat convection, and transmitted heat by conduction to the lower layers of the pavement structure.

**Governing equations for pavement surface**
When a plastic sheet is applied, the net radiation fluxes \( R_{nP} \) at the pavement surface consist of the shortwave radiation (incoming shortwave solar radiation) and longwave sky irradiance transmitted through the transparent plastic sheet, longwave radiation emitted from the plastic and pavement surface. Thus, the net radiation for the pavement surface with plastic sheet is given by:

\[
R_{nP} = (1 - \alpha_p) \tau_s q_{s\text{ol}} \rho^* + \rho_l^* \varepsilon_P \left( \tau_l \varepsilon_{sky} \sigma T_{sky}^4 + \varepsilon_l \sigma T_p^4 + \rho_l \varepsilon_P \sigma T_S^4 \right) - \varepsilon_P \sigma T_S^4
\]

\[\text{(4.29)}\]

Where,
- \( R_{nP} \) = net radiation at the pavement surface with plastic sheet, [W/m\(^2\)];
- \( \alpha_p \) = shortwave reflectivity of the pavement surface, [-];
- \( \tau_s \) = shortwave transmissivity of the plastic sheet, [-];
- \( \tau_l \) = longwave transmissivity of the plastic sheet, [-];
- \( \rho_l \) = longwave reflectivity of the plastic sheet, [-];
- \( \varepsilon_l \) = longwave emissivity of the plastic sheet, [-];
- \( T_p \) = temperature of the plastic sheet, [-].

The variables \( \rho^* \) and \( \rho_l^* \) in Equation (4.29) represent the multiple reflections of the shortwave and longwave radiation between the plastic sheet and pavement surface, respectively. For instance, considering the infinite transfer processes of short wave radiation under a polyethylene sheet as shown in Figure 4.15, the absorbed solar short wave radiation at the pavement surface with a polyethylene sheet is given by:
\[
q_{sol,p}^* = \tau_s q_{sol} (1 - \alpha_p) + \tau_s q_{sol} \alpha_p \rho_s (1 - \alpha_p) + \tau_s q_{sol} \alpha_p^2 \rho_s^2 (1 - \alpha_p) + \tau_s q_{sol} \alpha_p^3 \rho_s^3 (1 - \alpha_p) + \cdots
\] (4.30)

\[
\tau_s = \text{plastic sheet transmissivity}
\]
\[
\rho_s = \text{plastic sheet reflectivity}
\]
\[
\alpha_p = \text{pavement reflectivity}
\]

Figure 4.15 Schematic of multiple reflections of the shortwave radiation between polyethylene sheet and pavement surface.

Where, \(\rho_s\) is the shortwave reflectivity of the plastic sheet. Rearranging the terms on the right side of Equation (4.30) results in the following equation:

\[
q_{sol,p}^* = \tau_s q_{sol} (1 - \alpha_p) (1 + \alpha_p \rho_s + \alpha_p^2 \rho_s^2 + \alpha_p^3 \rho_s^3 + \cdots)
\] (4.31)

Based on Taylor series’ approach, then

\[
\rho^* = (1 + \alpha_p \rho_s + \alpha_p^2 \rho_s^2 + \alpha_p^3 \rho_s^3 + \cdots) = \frac{1}{1 - \alpha_p \rho_s}
\] (4.32)

Lastly, the absorbed short wave solar radiation at the pavement surface with polyethylene sheet can be represented as follows:

\[
q_{sol,p}^* = \tau_s q_{sol} (1 - \alpha_p) \rho^* = \tau_s q_{sol} (1 - \alpha_p) \frac{1}{1 - \alpha_p \rho_s}
\] (4.33)

Similarly, the variables accounting for the multiple reflections of longwave radiation can be obtained as follows:

\[
\rho_{ir}^* = \frac{1}{1 - \tau_l (1 - \varepsilon_p)}
\] (4.34)

The air gap between the pavement surface and the polyethylene sheet is very thin, thus it is adequate to assume that the temperature of the trapped air is the same as the temperature of the polyethylene sheet \(T_{ps}\). Moreover, the airflow through the wind does not affect the pavement surface covered by polyethylene sheeting, therefore, only the free heat convection is considered for the covered
pavement surface. Finally, the heat convection $q_{conv,ps}$ at the pavement surface covered by a polyethylene sheet is calculated by:

$$q_{conv,ps} = h_{conv,pl}(T_{ps} - T_s) \tag{4.35}$$

Where, the heat convection coefficient $h_{conv,pl}$ is determined by Equation (4.19) through inputting a zero wind speed.

Polyethylene sheet normally does not transmit water. Therefore, the latent heat through evaporation is taken as zero for the pavement surface covered with a polyethylene sheet. Then, the heat balance equation for the covered pavement surface is given as follows:

$$Rn + q_{conv,pl} - \lambda_c \left. \frac{\partial T}{\partial x} \right|_{surface} = 0 \tag{4.36}$$

**Governing equations for pavement surface**

As shown in the Equations (4.29) and (4.36), the polyethylene sheet temperature $T_{ps}$ must be known for each time step. It can be obtained by the heat balance equation for the plastic sheet. Considering the rather thin thickness of a polyethylene sheet, the heat flux through conduction and evaporation is considered as zero, and the heat balance equation for the plastic sheet can thus be written as:

$$Rn_{ps} + h_{conv,ps}(T_s - T_{ps}) + h_{conv}(T_a - T_{ps}) = 0 \tag{4.37}$$

Where, $Rn_{ps}$ is the net heat radiation for the plastic sheet; the subsequent two terms in Equation (4.37) represent the convective heat transfer between the downward plastic sheet surface and underlying pavement surface, and between the upward plastic sheet surface and atmosphere, respectively. The convective heat transfer coefficient $h_{conv,ps}$ between downward plastic sheet surface and the pavement surface is calculated by Equation (4.35), while the convective heat transfer coefficient $h_{conv}$ between the upward plastic sheet surface and atmosphere is defined by Equation (4.19). Similar to the net radiation for the pavement surface, the net radiation $Rn_{ps}$ for the plastic sheet is given by:

$$Rn_{ps} = (1 - \alpha_s)q_{sol}(1 + \rho_s \tau_s \alpha_P) + \varepsilon_i \varepsilon_{sky} \sigma T_{sky}^4 (1 + \rho_{ir} \tau_i (1 - \varepsilon_P)) + \rho_{ir} \varepsilon_i \varepsilon_P \sigma T_s^4 + \rho_{ir} (\varepsilon_i^2 \sigma T_{ps}^4 (1 - \varepsilon_P) - 2 \varepsilon_i \sigma T_{ps}^4) \tag{4.38}$$

Where, $\varepsilon_i$ is the emissivity of the plastic sheet that is equal to the longwave absorptivity of the plastic sheet, and is calculated from the corresponding longwave reflectivity and transmissivity of the plastic sheet ($\varepsilon_i = 1 - \tau_i - \rho_i$). Putting Equation (4.38) into Equation (4.37), the heat balance equation for the plastic sheet can be written as follows:
(1 - α_s)q_{sol}(1 + \rho_s \tau_s \alpha_p) + \varepsilon_i \varepsilon_{sky} \sigma T_{sky}^4 \left(1 + \rho_{ir} \tau_l (1 - \varepsilon_p)\right) + \rho_{ir} \varepsilon_i \varepsilon_p \sigma T_s^4 + \rho_{ir} \left(\varepsilon_i \sigma T_{ps}^4 (1 - \varepsilon_p) - 2 \varepsilon_i \sigma T_{ps}^4 + h_{conv, ps} (T_s - T_{ps})\right) + h_{conv} (T_a - T_{ps}) = 0 \tag{4.39}

Lastly, the unique value of $T_{ps}$ that simultaneously satisfies the heat balance Equation (4.39) for the plastic sheet is obtained by solving this quartic equation using Matlab. The optical properties of the polyethylene have been recognised as its primary parameters affecting the soil temperature (Mahrer 1979; Ham et al. 1993; Ham and Kluitenberg 1994). However, no European standard is currently available for selecting the type of polyethylene sheeting for concrete curing. ASTM C171 (2007) is currently the only available standard that covers sheet materials for curing concrete. Two types of polyethylene films, clear and white opaque, are included in ASTM C171. However, ASTM C171 only specifies that the minimum thickness of the used polyethylene films shall be not less than 0.10 mm and the daylight reflectance of the white opaque polyethylene sheet shall be at least 70\%, and no more requirements are listed. The low-density clear polyethylene sheeting is commonly used for the initial curing of a concrete pavement in Belgium. Table 4.5 summarizes the reported optical properties for clear polyethylene sheets by the other authors (Mahrer 1979; Sui and Zeng 1992; Ham and Kluitenberg 1994; Wu et al. 1996; Castro and Rey 2011). The moderate values of optical properties reported by Ham and Kluitenberg (1994) are used in the present study: the transmissivity and reflectivity of shortwave radiation are 0.84 and 0.11, respectively, and the transmissivity and the reflectivity of longwave radiation are 0.78 and 0.17, respectively. According to the heat transfer theory, the emissivity of a body to the radiation of certain wavelength is equal to the relevant absorptivity, and the sum of the transmissivity, the reflectivity, and the emissivity equals a unity.

<table>
<thead>
<tr>
<th>Author</th>
<th>Short wave optical properties</th>
<th>Long wave optical properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>transmissivity</td>
<td>reflectivity</td>
</tr>
<tr>
<td>Mahrer, 1979</td>
<td>0.880</td>
<td>0.220</td>
</tr>
<tr>
<td>Sui et al. 1992</td>
<td>0.840</td>
<td>--</td>
</tr>
<tr>
<td>Ham et al. 1994</td>
<td>0.840</td>
<td>0.110</td>
</tr>
<tr>
<td>Wu et al. 1996</td>
<td>0.890</td>
<td>0.060</td>
</tr>
<tr>
<td>De Castro et al. 2011</td>
<td>0.733</td>
<td>0.265</td>
</tr>
</tbody>
</table>

Figure 4.16 shows the calculated convective heat transfer at the pavement surface for E17 by the new model considering the optical properties of the polyethylene sheet. It clearly indicates that the polyethylene sheet reduce the convective heat flux significantly at the pavement surface. It is also observed that the convective heat flux is not sensitive to the wind speed when the pavement is covered by the plastic sheet curing. Detailed discussions on the influencing of plastic sheeting on concrete pavement temperature are further addressed in Section 6.3.
4.2 NUMERICAL IMPLEMENTATION

The heat-diffusion problem of the hardening concrete can be solved numerically through the finite difference method, finite element method, etc. Among those, the explicit finite difference method is commonly adopted due to its advantages of easy implementation and sufficient accuracy (Incropera 2011). Besides, the finite difference method can easily remedy the issue of the inconsistency in thermal properties of the material of pavement layers (Alavi et al. 2014).

Figure 4.17 shows the finite differential discretization of the space-time domain of the heat diffusion process of hardening concrete. The space domain is subdivided finer in the concrete slab and coarser below the pavement slab. The central-difference scheme for the space derivative and the forward scheme for the time derivative are introduced in the expression of the finite differential equation. Thus, the following finite differential expressions are adopted to approximate the second order space derivative and the first time derivative of the temperature field $T(x,t)$ in the node of the space-time domain (Chapra and Canale 2010). A forward scheme for time derivative leads to an explicit numerical integration scheme.

$$\frac{\partial^2 T}{\partial x^2}_{n,k} \approx \frac{T_{n+1,k} - 2 \cdot T_{n,k} + T_{n-1,k}}{\Delta x^2} \quad (4.40)$$

$$\frac{\partial T}{\partial t}_{n,k} \approx \frac{T_{n,k+1} - T_{n,k}}{\Delta t} \quad (4.41)$$

Where, $n$ is the node and $k$ is the time step.
4.2.1 Surface Node

For the edge node \( n = 0 \) at the pavement surface, \( T_{0,k+1} \) can be derived through the heat energy balance of a control volume (Patankar 1980), using the temperature of surface node \( T_{0,k} \) and the first node below the surface \( T_{1,k} \) in the previous time step \( k \), along with the internal heat of cement hydration rate \( q_{0,k} \) and the net heat flux at the pavement surface \( q_{hf,k} \). \( T_{0,k+1} \) is expressed as follows:

\[
T_{0,k+1} = T_{0,k} + 2 \frac{\lambda_c}{\rho_c c_c \Delta x_c} \Delta t (T_{1,k} - T_{0,k}) + \frac{q_{0,k} \Delta t}{\rho_c c_c} + 2 \frac{q_{hf,k} \Delta t}{\rho_c c_c \Delta x_c} \tag{4.42}
\]

Where, \( \lambda_c, \rho_c, c_c \) are the thermal conductivity, density, and heat capacity of concrete, respectively. \( \Delta x_c \) is the space increment in the concrete slab layer. When the pavement is not covered with a plastic sheet, \( q_{hf,k} \) is calculated by summing \( q_{conv,k} \) (Equation 4.19), \( q_{ir,k} \) (Equation 4.22) and \( q_{sol,k} \) (Equation 4.28). When the pavement is cured with a plastic sheet, \( q_{hf,k} \) is calculated as the sum of \( Rn_{p,k} \) (Equation 4.32) and \( q_{conv,ps,k} \) (Equation 4.38).

4.2.2 Interior Nodes

For the interior nodes within the concrete pavement slab \( (n = 1, \cdots, n_c - 1, n_c \) is the bottom node of the concrete slab), considering the internal heat hydration \( q_{n,k} \), the following finite differential expression can be determined by introducing...
Equation (4.40) and (4.41) into Equation (4.1). $T_{n,k+1}$ of the interior nodes within the concrete pavement slab is explicitly calculated according to the known temperature data of the previous step:

$$T_{n,k+1} = T_{n,k} + \frac{\lambda_c}{\rho_c c_c} \frac{\Delta t}{\Delta x_c} (T_{n+1,k} - 2T_{n,k} + T_{n-1,k}) + \frac{q_{n,k}\Delta t}{\rho_c c_c} \tag{4.43}$$

Similar to the interior nodes in the concrete pavement slab, the other interior nodes in the underlying pavement layers (base, sub-base, and subgrade) without the internal heat source can be derived by the above mentioned explicit numerical integration scheme. For instance, $T_{n,k+1}$ of interior nodes ($n = n_c + 1, \ldots, n_a - 1$, $n_a$ is the bottom node of the asphalt interlayer) of the asphalt interlayer can be presented as bellows:

$$T_{n,k+1} = T_{n,k} + \frac{\lambda_a}{\rho_a c_a} \frac{\Delta t}{\Delta x_a} (T_{n+1,k} - 2T_{n,k} + T_{n-1,k}) \tag{4.44}$$

Where, $\lambda_a, \rho_a, c_a$ are the thermal conductivity, density, and heat capacity of the asphalt interlayer, respectively, and $\Delta x_a$ is the space increment in the asphalt interlayer.

### 4.2.3 Interface Nodes

The heat balance method of the control volume is also adopted to express $T_{n,k+1}$ of the interface nodes (such as, $n_c$ is the interface node between pavement slab and base layer, and $n_b$ is the interface node between base and subbase, etc.) between the pavement layers. Among those, the internal cement hydration has to be included during the derivation of the temperature $T_{n,k+1}$ of the interface node of pavement slab and underneath layer of the subsequent time step.

For the interface node $n_c$ (the bottom node of the concrete slab):

$$T_{n,k+1} = T_{n,k} + 2 \frac{\lambda_c}{\rho_c c_c \Delta x_c + \rho_a c_a \Delta x_a} \frac{\Delta t}{\Delta x_c} (T_{n-1,k} - T_{n,k}) + 2 \frac{\lambda_b}{\rho_c c_c \Delta x_c + \rho_a c_a \Delta x_a} \frac{\Delta t}{\Delta x_a} (T_{n+1,k} - T_{n,k}) + \frac{\lambda_b}{\rho_c c_c \Delta x_c + \rho_a c_a \Delta x_a} q_{n,c,k} \tag{4.45}$$

For the interface node $n_a$ (the bottom node of the asphalt interlayer):

$$T_{n,k+1} = T_{n,k} + 2 \frac{\lambda_a}{\rho_a c_a \Delta x_a + \rho_b c_b \Delta x_b} \frac{\Delta t}{\Delta x_a} (T_{n-1,k} - T_{n,k}) + 2 \frac{\lambda_b}{\rho_a c_a \Delta x_a + \rho_b c_b \Delta x_b} \frac{\Delta t}{\Delta x_b} (T_{n+1,k} - T_{n,k}) \tag{4.46}$$
Where $\lambda_b, \rho_b, c_b$ are the thermal conductivity, density, and heat capacity of the layer below the asphalt interlayer, respectively, and $\Delta x_b$ is the space increment in this base layer.

### 4.2.4 Bottom Node

As discussed in Section 4.1.8, the temperature of the node of the bottom of subgrade is assumed a constant value equal to the annual average air temperature at the worksite.

$$T_{n_s,k+1} = \text{constant ground temperature} \quad (4.47)$$

### 4.2.5 Heat Generation Rate

The degree of hydration of hardening concrete is defined as the fraction of cement that has already hydrated. According to the heat hydration model proposed by De Schutter (1996), it yields a heat production rate depending on the actual degree of hydration and the actual temperature. The procedure to calculate the heat generation rates for the ‘$k+1$’ step explicitly based on the temperature field and the accumulated amount of hydration at the previous step is described as follows.

$$q_{p,n,k} = q(r_{p,n,k}) \cdot \exp\left[\frac{E_p}{R} \left(\frac{1}{293} - \frac{1}{273 + T_{n,k}}\right)\right]$$

$$q_{s,n,k} = q(r_{s,n,k}) \cdot \exp\left[\frac{E_s}{R} \left(\frac{1}{293} - \frac{1}{273 + T_{n,k}}\right)\right] \quad (4.48)$$

$$Q_{p,n,k+1} = \sum_{i=1}^{k+1} \frac{q_{p,n,i} + q_{p,n,i-1}}{2} \cdot \Delta t \right\}$$

$$Q_{s,n,k+1} = \sum_{i=1}^{k+1} \frac{q_{s,n,i} + q_{s,n,i-1}}{2} \cdot \Delta t \right\} \quad (4.49)$$

$$r_{p,n,k+1} = \left\{ \frac{Q_{p,n,k+1}}{Q_{p,max}} \right\}$$

$$r_{s,n,k+1} = \left\{ \frac{Q_{s,n,k+1}}{Q_{s,max}} \right\} \quad (4.50)$$

### 4.2.6 Numerical Implementation Procedure

Initial conditions can be easily imposed as follows:

$$T_{n,0} = T_R, \text{ for } n = 0, \cdots, n_s \quad (4.51)$$

Equation (4.43) to (4.46) demonstrate that the outlined numerical procedure is explicit in space and time, as the value $T_{n,k+1}$ depends on values of the tempera-
ture field at time $t_k$ and it also involves the degree of reaction $r_{p,n,k+1}$ and $r_{S,n,k+1}$ which, in turn, depend on all previous analysis steps. However, for each time step $t_k$, the values of the temperature of $(n_s + 1)$ nodes should be determined. In fact, the $(n_s - 1)$ field through Equation (4.43) to (4.46) along with the boundary conditions Equations (4.43) and (4.51) balance the number of unknowns. In order to achieve a stable solution for the explicit numerical integration scheme considered in this approach, a convergence criterion has to be considered. For a steady state one dimensional schematization for time integration, the following criterion holds (Carnahan et al. 1969):

$$\frac{\lambda}{\rho c} \frac{\Delta t}{\Delta x^2} \leq \frac{1}{2} \quad (4.52)$$

The proposed numerical solution of the heat-diffusion problem can easily be implemented in either a spreadsheet or a high-level programming language. In the present study, the temperature prediction algorithm is conducted with Matlab, as illustrated in Appendix I.

4.3 MODEL VERIFICATION

4.3.1 Simulation Information

Field measured temperature data of in-situ CRCP pavement slab on E17 near Ghent in August 2011, and on E313 at Herentals in September 2012 are used to verify the proposed temperature prediction model at early age. Thermocouples were installed along various depth of the pavement slab and the temperatures are recorded at half hour intervals over a 72 hours period. The pavement structures of both worksites are designed under the current CRCP standard design concept 3 in Belgium. The pavement structure consists of a 250 mm CRC slab laid upon a 50 mm bituminous inter-layer and a 300 mm roller compacted concrete base or lean concrete base, and then is sand sub-base and subgrade. However, it should be mentioned that the CRCP slab of E313 consists of two-lift construction, with a 50 mm top layer and 200 mm bottom layer constructed wet by wet. The depth of the simulation model is selected as 6.0 m and at that depth a constant ground temperature of 11.0°C, the approximation of the annual average air temperature in Belgium, is used. The computational domain is discretized by 48 elements, with 10 elements for the pavement slab, 2 elements for the asphalt interlayer, 6 elements for the cement-stabilized base, 6 elements for the granular base, and 24 elements for the subgrade. The thickness and thermal parameters, tabulated in Table 4.6, of the pavement slab and the underlying layers are obtained from literatures (Thompson et al. 1987; Schindler et al. 2002). The time increment for each step is 180 seconds to ensure the convergence of the explicit numerical integration scheme considered in this approach.
The concrete mixture compositions, construction conditions, and hydration parameters for both case studies are listed in Table 4.2. The cement adopted in both cases is blast furnace slag cement, CEM III/A 42.5 N/LA, produced by Holcim. It contains 36% to 65% granulated blast furnace slag, with 410 m²/kg Blaine surface, the chemical composition and the finesse of the cement is given in Table 3.1 and Table 3.3. Climatic input parameters are obtained from the nearest weather stations (www.wunderground.com), as shown in Figure 4.18. The ambient environmental conditions at any specific time are calculated by cubic spline interpolation of the values from the weather station records.

Figure 4.18 Air temperature, wind speed and solar radiation during the construction period for the worksites on E313 (September 2012) and E17 (August 2011), respectively.

The required concrete hydration parameters of the De Schutter hydration model for both projects are obtained from the measured values through isothermal calorimetry conduction tests on paste samples. For the sake of simplicity, the two
layers of the concrete slab in E313 are considered as one layer due to the relative small thickness of the top layer and the applied same type of cement with similar concrete mixture compositions. The same concrete hydration parameters are used to simulate the temperature in E17 and E313. The used values of those parameters of CEM III/A 42.5 N/LA are tabulated in Table 4.2.

Table 4.6 Summary of thermal parameters of concrete slab and underlying layers.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Concrete slab</th>
<th>Asphalt interlayer</th>
<th>Cement treated base</th>
<th>Sub-base</th>
<th>Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (m)</td>
<td>0.25</td>
<td>0.05</td>
<td>0.30</td>
<td>0.60</td>
<td>4.80</td>
</tr>
<tr>
<td>Space increment (m)</td>
<td>0.025</td>
<td>0.025</td>
<td>0.05</td>
<td>0.10</td>
<td>0.20</td>
</tr>
<tr>
<td>Time increment (s)</td>
<td>180</td>
<td>180</td>
<td>180</td>
<td>180</td>
<td>180</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2350</td>
<td>2300</td>
<td>2350</td>
<td>1800</td>
<td>2000</td>
</tr>
<tr>
<td>Heat capacity (J/m³/°C)</td>
<td>1000</td>
<td>1050</td>
<td>1000</td>
<td>900</td>
<td>1200</td>
</tr>
<tr>
<td>Heat conductivity (W/m²/°C)</td>
<td>3.0</td>
<td>1.4</td>
<td>2.5</td>
<td>2.4</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table 4.7 Summary of input parameters for both case studies in Belgium.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>E17 Ghent</th>
<th>E313 Herentals</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top layer</td>
<td>Bottom layer</td>
</tr>
<tr>
<td><strong>Construction Conditions</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction day and time</td>
<td>22:00</td>
<td>00:00,</td>
</tr>
<tr>
<td>18/08/2011</td>
<td>12/09/2012</td>
<td>23:30,</td>
</tr>
<tr>
<td>Fresh concrete temperature</td>
<td>25.0 °C</td>
<td>22.5 °C</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22.5 °C</td>
</tr>
<tr>
<td><strong>Curing method (plastic sheet)¹</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plastic sheet placement time</td>
<td>22:00</td>
<td>00:00,</td>
</tr>
<tr>
<td>18/08/2011</td>
<td>12/09/2012</td>
<td>--</td>
</tr>
<tr>
<td>Plastic sheet removal time</td>
<td>15:00,</td>
<td>13:30,</td>
</tr>
<tr>
<td>19/08/2011</td>
<td>12/09/2012</td>
<td>--</td>
</tr>
<tr>
<td>Polyethylene sheet type</td>
<td>Clear low density polyethylene</td>
<td></td>
</tr>
<tr>
<td>Shortwave radiation transmissivity</td>
<td>0.84</td>
<td></td>
</tr>
<tr>
<td>Shortwave radiation reflectivity</td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td>Longwave radiation transmissivity</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>Longwave radiation reflectivity</td>
<td>0.17</td>
<td></td>
</tr>
<tr>
<td>Pavement surface emissivity</td>
<td>0.80</td>
<td></td>
</tr>
<tr>
<td>Pavement surface absorptivity</td>
<td>0.50</td>
<td></td>
</tr>
</tbody>
</table>

Note:
1. The pavement was covered with polyethylene sheeting immediately after the concrete placement and the application of the setting retarder applied against drying out. Subsequent to the removal of the polyethylene sheeting and the skin of the concrete mortar, the concrete is protected against drying out a second time by spraying a white curing compound.

### 4.3.2 Prediction Results

The estimated hourly concrete temperatures at various depths for both E313 and E17 are illustrated in Figure 4.19. The calculated results for both cases are quite
close to the observed values. Moreover, the patterns of the estimated concrete temperatures at various depths are also similar to that of the corresponding observed temperatures. As shown in the Figure 4.19, the highest concrete temperature occurs during the first 24 hours after the concrete placement, which is due to the combination of external effects such as air temperature and internal effects of the generated hydration heat during the concrete curing process. The errors between estimated and observed temperature in the first 72 hours after concrete placement for both cases are summarized in Figure 4.20. The deviations of the estimated and observed values are mostly within the ± 3.0 °C, and the largest deviations mainly occur at the first daytime when the internal heat of hydration generation rate is the highest.

To test the accuracy of the proposed temperature prediction model, field measured concrete pavement temperatures are compared to the predicted values over the first 72 hours. The goodness of fit is assessed in several ways (Reicosky et al. 1989). Firstly, the Absolute Mean Error (AME), defined as the sum of the absolute value of the difference between the estimated and field measured temperature for a 72 hours period, is given by:

\[
AME = \frac{\sum_{i=1}^{n} |T_{Pi} - T_{Mi}|}{n}
\]  

(4.53)

Where \(n\) is the number of observations, \(T_{Pi}\) is the estimated temperature, and \(T_{Mi}\) is the observed temperature at the corresponding time.

Root Mean Square Error (RMSE) reflects the overall accuracy of the estimated curves, and is defined as:

\[
RMSE = \left[ \frac{\sum_{i=1}^{n} (T_{Pi} - T_{Mi})^2}{n} \right]^{1/2}
\]  

(4.54)

The closer the estimated temperatures are to the observed temperature, the smaller the RMSE. The RMSE tends to penalize larger individual errors heavily and therefore as such may be the better criterion of performance.

Lastly, the sum of residuals (RES) and the sum of the absolute residuals (|RES|) can be used to evaluate the tendency for the model consistently to over-predict or under-predict the temperature over a period. RES and |RES| are given by the following equations:

\[
RES = \sum_{i=1}^{n} (T_{Pi} - T_{Mi})
\]  

(4.55)
\[ |RES| = \sum_{i=1}^{n} |T_{P_i} - T_{M_i}| \] (4.56)

By comparing \( RES \) and \( |RES| \), one can determine how errors will cancel over a period of time. A large positive \( RES \) that approaches \( |RES| \) suggests that the model consistently overestimates the actual value. By contrast, a large negative \( RES \) compared to \( |RES| \) indicates that the model consistently underestimates the actual value. A small value for \( RES \) in comparison to \( |RES| \) indicates that the errors of the model tend to cancel over the calculated period.

The error analyses for the first 72 hours after concrete placement for both E17 and E313 are summarized in Table 4.8. The relative lower value of \( AME \) and \( RMSE \) indicates a reasonable fit between the observed and estimated temperatures for every location. Within each project, higher values of \( AME \) and \( RMSE \) are observed for the upper part of the pavement slab. The observed negative values of \( RES \) in the project E17 as compared to \( |RES| \) for all locations suggest that the proposed model consistently underestimates the actual temperature value. The predicted temperature at various depths shows much smaller discrepancies between the measurement data in the project E313. The correlation coefficients \( R^2 \) for all the locations for both cases are above 0.915 as shown in Figure 4.21, suggesting a very accurate fit, based on the overall first 72 hour data.

Table 4.8 Summary of the statistics of hourly estimated and observed temperatures (°C) for the first 72 hours after concrete placement.

<table>
<thead>
<tr>
<th>Statistics</th>
<th>E17</th>
<th>E313</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50mm</td>
<td>100mm</td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.915</td>
<td>0.948</td>
</tr>
<tr>
<td>( AME )</td>
<td>1.31</td>
<td>1.16</td>
</tr>
<tr>
<td>( RMSE )</td>
<td>1.67</td>
<td>1.37</td>
</tr>
<tr>
<td>( RES )</td>
<td>-73.39</td>
<td>-63.15</td>
</tr>
<tr>
<td>(</td>
<td>RES</td>
<td>)</td>
</tr>
</tbody>
</table>
Figure 4.19 Estimated and observed temperature at various depth of concrete slab during the first 72 hours after concrete placement (a) on E313, Herentals, September 12 to 15, 2012; (b) on E17, Ghent, August 19 to 22, 2011 (no measurements done between 42 and 58 hours).
Figure 4.20 The hourly estimated and observed temperature error at various depth of concrete slab during the first 72 hours after concrete placement (a) on E313, Herentals, September 12 to 15, 2012; (b) on E17, Ghent, August 19 to 22, 2011.
Chapter 4 Early Age Concrete Pavement Temperature

Figure 4.21 The linear correlation between the hourly estimated and observed temperature at various depth of concrete slab during the first 72 hours after concrete placement (a) on E313, Herentals, September 12 to 15, 2012; (b) on E17, Ghent, August 19 to 22, 2011.
4.4 WEATHER FORECAST BASED TEMPERATURE PREDICTION

Figure 4.19 shows that the climate condition during the construction period plays a crucial role on the early age concrete pavement temperature distributions. The developed temperature model requires complete climate data sets as input parameters. Previous temperature prediction models normally use the historical record of monthly average maximum and average minimum temperatures of a given weather station. It could provide acceptable temperature prediction results in the design phase of a project. However, in order to prevent the uncontrolled transverse cracking due to unexpected climate conditions before sawcut operations during the construction phase, a weather forecast based concrete pavement temperature prediction model provides more accurate concrete pavement temperatures enabling the contractors to apply the relevant treatments at the right moment.

4.4.1 Approximated Air Temperature by Daily Minima and Maxima

Generally, the common weather forecast data consists of only the daily minimum and maximum air temperatures. In order to create complete weather data sets with the required short time intervals, it is therefore necessary to use appropriate and reliable data estimation methods. The shape of diurnal temperature curves have been modelled in numerous ways that vary from simple sine-curve fitting models to techniques that are more sophisticated. Reicosky et al. (1989) summarized several simple curve-fitting methods of hourly air temperatures from the daily minimum and maximum temperatures. His results have indicated that these curve fitting models worked well on clear days but with limited success on overcast days. The method presented by De Wit (1978) is selected because it is presumed to be not site specific and requires limited inputs, and the estimated air temperature shows reasonable accuracy. The day is divided into 3 segments: (a) midnight to sunrise + 2 hours (daily minimum temperature $T_{\text{min}}$); (b) daylight hours; (c) sunset to midnight. The method assumes a sinusoidal wave during the daylight period and the night temperature decreases exponentially to $T_{\text{min}}$ of the next day. In addition to the current day’s maximum and minimum temperature, this method also requires the maximum and minimum temperature of the previous day and the minimum of the following day. The intermediate air temperatures are calculated from the following equations:

(a) midnight time to sunrise $t_{\text{sr},n} + 2h$

$$T(t) = T_{ss,n-1} \exp[\ln(\frac{T_{\text{min},n}}{T_{ss,n-1}}) \cdot \frac{t - t_{ss,n-1}}{t_{\text{sr},n} + 26 - t_{ss,n-1}}] \quad (4.57)$$

(b) sunrise $t_{\text{sr},n} + 2h$ to sunset $t_{ss,n}$
\[
T(t) = T_{\text{min},n} + (T_{\text{max},n} - T_{\text{min},n}) \cdot \sin\left[\frac{t - t_{\text{sr},n} - 2}{t_{\text{ss},n} - t_{\text{sr},n}} \pi\right]
\] (4.58)

(c) sunset \(t_{\text{ss},n}\) to midnight

\[
T(t) = T_{\text{ss},n} \exp\left[\ln\left(\frac{T_{\text{min},n+1}}{T_{\text{ss},n}}\right) \cdot \frac{t - t_{\text{ss},n}}{t_{\text{sr},n+1} + 26 - t_{\text{ss},n}}\right]
\] (4.59)

Where,
- \(t\) = time in hours, \([0, 24]\);
- \(t_{\text{sr}}\) = time of sunrise, \([\text{hour}]\);
- \(t_{\text{ss}}\) = time of sunset, \([\text{hour}]\);
- \(T_{\text{max}}\) = maximum air temperature of a day, \([\degree C]\);
- \(T_{\text{ss}}\) = air temperature at sunset, \([\degree C]\);
- \(T_{\text{min}}\) = minimum air temperature of a day, \([\degree C]\).

The subscript ‘\(n\)’ in above Equations (4.57) to (4.59) represents the current simulated day, and the subscript ‘\(n-1\)’ and ‘\(n+1\)’ indicates the previous day and the following day, respectively. A simple method recommended by NOAA (2014) gives very accurate sunrise and sunset times for most parts of the earth for construction engineering purposes and is therefore adopted in the present study. The determination of \(t_{\text{sr}}\) and \(t_{\text{ss}}\) based on the NOAA algorithm is briefly presented as follows:

Firstly, the fractional of the year \(\gamma_{\text{DAY}}\) is calculated, in radians.

\[
\gamma_{\text{DAY}} = \frac{2\pi}{365} (\text{DAY} - 1)
\] (4.60)

Where, \(\text{DAY}\) is the number of days elapsed in each year, for instance, January 1st is the first day of a year. With the known \(\gamma_{\text{DAY}}\), the Equation of time \(t_{\text{et}}\) (the discrepancy between the apparent solar time which directly tracks the motion of the sun, and mean solar time which tracks a fictitious ‘mean’ sun with 24 hours apart, here is described in minutes) and the solar declination angle \(\delta\) (in radians) can be estimated:

\[
t_{\text{et}} = 229.18 \times [0.00075 + 0.001868\cos(\gamma_{\text{DAY}}) - 0.032077\sin(\gamma_{\text{DAY}}) - 0.014615\cos(2\gamma_{\text{DAY}}) - 0.040849\sin(2\gamma_{\text{DAY}})]
\] (4.61)

\[
\delta = 0.006918 - 0.399912\cos(\gamma_{\text{DAY}}) + 0.070257\sin(\gamma_{\text{DAY}}) - 0.006758\cos(2\gamma_{\text{DAY}}) + 0.000907\sin(2\gamma_{\text{DAY}}) - 0.002697\cos(3\gamma_{\text{DAY}}) + 0.00148\sin(3\gamma_{\text{DAY}})
\] (4.62)

Then, the solar zenith angle \(\varphi_{\text{sz}}\) as a function of time, day number, and latitude \(\varphi_{\text{lat}}\) is calculated by the following equation:
\[
\cos \varphi_{sz} = \sin \varphi_{lat} \sin \delta + \cos \varphi_{lat} \cos \varphi_{lat} \cos \omega
\] (4.63)

Where, \( \omega \) is hour angle. At solar noon, the hour angle is 0°, with the local time before solar noon expressed as negative degrees, and the time after solar noon expressed as positive degrees. For the time of sunrise and sunset, the solar zenith angle \( \varphi_{sz} \) is chosen as 90.83° considering the approximate correction for atmospheric refraction at sunrise and sunset. According to Equation (4.63), the hour angle \( \omega_0 \) for the sunrise/sunset then becomes:

\[
\cos \omega_0 = \frac{\sin(-0.83°) - \sin \varphi_{lat} \sin \delta}{\cos \varphi_{lat} \cos \delta}
\] (4.64)

Then, the solar noon \( t_{sn} \) (in hour) for a given location is found from the longitude \( \varphi_{longi} \) (in degrees, defined as positive east of Greenwich) and the Equation of time \( t_{et} \):

\[
t_{sn} = 12 + T_{zone} - \varphi_{longi}/4 - t_{et}/60
\] (4.65)

Where, \( T_{zone} \) is the time zone for a given location. Finally, the sunrise and sunset time, in hours, for a given longitude and latitude under the local UTC time is:

\[
\begin{align*}
&\{ t_{sr} = t_{sn} - \omega_0/15 \\
&t_{ss} = t_{sn} + \omega_0/15
\}
\] (4.66)

Approximated air temperature data from daily minima and maxima and the recorded data of the meteorological station for these two projects of E17 and E313 during the construction period are illustrated in Figure 4.22. It clearly shows that the proposed method produces high quality air temperature data every few minutes for the concrete pavement temperature prediction model, while only limited climate information is required. This approach can also be used to generate the relative humidity data for the concrete pavement temperature prediction model. It is assumed that the daily relative humidity variation is developing conversely to the air temperature.
Figure 4.22 Approximated air temperature data from daily minimum and maximum air temperature.

### 4.4.2 Solar Radiation

Solar radiation for short time periods is rarely reported by meteorological stations. Most meteorological stations record sunshine duration (hours) covering longer periods. In fact, only 0.2% of the meteorological stations record the short time period’s solar radiation on a global scale (Thornton and Running, 1999). The instantaneous solar radiation during one day depends on the latitude of the location, atmospheric conditions, and incident angle of the sun’s ray with the ground surface that varies with the time of a year. McCullough and Rasmussen (1999) proposed the following formula for the shortwave absorption of solar radiation.
\[ q_{ins} = I_f \cdot q_{solar} \] (4.67)

Where, \( I_f \) is the intensity factor accounting for the sun’s angle during a 24-hour/day, and \( q_{solar} \) is the peak value of the solar radiation during a day period. The solar radiation intensity factor \( I_f \) is assumed to follow a sinusoidal function (Bentz 2000; Ge 2005; Ruiz et al. 2006; Qin 2011). It ranges from zero at both sunrise and sunset to a peak value at solar noon. The solar radiation intensity factor \( I_f \) is presented as follows:

\[
I_f = \begin{cases} 
0 & t \in [0, t_{sr}) \\
\sin\left(\frac{\pi}{t_{ss}-t_{sr}}t + \frac{t_{sr}}{t_{sr}-t_{ss}}\pi\right) & t \in [t_{sr}, t_{ss}] \\
0 & t \in (t_{ss}, 24] 
\end{cases} \] (4.68)

The peak daily solar radiation \( q_{solar} \) including the direct solar radiation and the diffuse solar radiation is usually obtained from the historical records of a weather station. The value of \( q_{solar} \) used in HIPERPAV II is based on the 95-percentile value of solar radiation from historical records at the given weather stations. Qin (2011) used the monthly historical solar radiation records from the weather station together with a random normal distribution to generate probabilistic daily peak radiation data. Figure 4.23 illustrates the comparison of the calculated solar radiation intensity factor \( I_f \) to the normalized solar radiation for a very bright day of September 4, 2012 at Herentals (51.13°N, 4.83°E), Belgium. It indicates that the sinusoidal function consistently overestimates the global solar radiation for the whole day. This discrepancy is easy to understand because the solar radiation received by a horizontal surface on ground level is not only depending on the daily varying solar elevation angle, but also depending on the path of solar radiation through the atmosphere. Thus, the simple sinusoidal function cannot accurately describe the solar radiation. Moreover, this discrepancy can be considerable larger especially for the high latitude area in a summer period.

Slob and Moona (1991) derived an experience-based relation to describe the intensity of the direct solar radiation \( q_{solar, direct} \) on a horizontal surface in a bright day. This relation is adopted in the present study and it is formulated as follows:

\[
q_{solar, direct} = q_0 \sin \varphi_{se} \exp\left(-\frac{T_l}{0.949 + 9.8 \sin \varphi_{se}}\right) \quad \varphi_{se} > 10^\circ \] (4.69)

Where, \( q_0 \) is the solar constant, having an average value of 1367 W/m². \( T_l \) is the Linke turbidity that describes the dispersion and pollution of the atmosphere. Commonly, the Linke turbidity factor ranges between 4 and 7, and extreme values are 2.5 and 10. The solar elevation angle \( \varphi_{se} \), in degrees, is the complementary angle of solar zenith angle \( \varphi_{sz} \), thus,

\[
\varphi_{se} = \arcsin(\sin \varphi_{lat} \sin \delta + \cos \varphi_{lat} \cos \varphi_{lat} \cos \omega) \] (4.70)
The diffuse component of solar radiation is also evaluated with the solar elevation angle and the Linke turbidity factor (Slob and Moona, 1991).

\[ q_{\text{solar, diffuse}} = 40.3 + 41.3T_i \sin \varphi_{se} \quad \varphi_{se} > 5^\circ \]  

(4.71)

The global solar radiation for the bright day at Herentals in September 4, 2012 is calculated by Slob’s approach with a value of 4 for Linke turbidity factor, which is considered as the common condition for the Netherlands and Belgium. The calculated global solar radiation is then normalized with its daily peak value and is illustrated in Figure 4.23. The approach by Slob gives an accurate description of the solar intensity factor for a bright day. The global solar radiation for the cloudy and overcast conditions are also well documented by Slob and Moona (1991). Figure 4.24 presents the global solar radiation for a bright day at Ghent (51.05° N, 3.70° E) for different seasons.

![Figure 4.23 Solar intensity factors for a bright day in Herentals, in September 2012.](image)

![Figure 4.24 Global solar radiation versus daily variation in the four seasons, in Ghent.](image)
4.5 SUMMARY

This chapter describes the processes of developing an early age concrete pavement temperature prediction model. Firstly, the heat of hydration of concrete is determined by isothermal calorimetry conduction tests on paste sample, and a hydration model that more accurate simulates the heat generation rate for slag cement is adopted in this study. Subsequently, a critical review of the boundary conditions with respect to a concrete pavement is well documented. Then, a simple numerical calculation using the finite differential method is presented. After that, the proposed temperature model is verified with two field measurements. Lastly, this Chapter gives a reliable procedure to generate high quality climate input parameters for the concrete temperature prediction model.

- The hydration model by De Schutter for blended cement enables an accurate simulation of the heat production rate as a function of the actual temperature and degree of hydration. Considering the significant effect of the activation energy on the heat generation rate, more tests under different temperature conditions are recommended for further studies.
- The net heat flux on the pavement surface through convection, irradiation, and absorption dominates the temperature development in the pavement slab and should thus be described accurately. Based on the critical review of the commonly used convective heat transfer coefficients models, a wind correction factor is required when directly using the climatic data from the standard weather stations.
- Polyethylene sheeting curing is very beneficial in capturing moisture of fresh concrete that minimizes the plastic shrinkage damage and reduces the drying shrinkage as well. However, the polyethylene sheeting curing method can be detrimental if used improperly. It could result in too high concrete temperature in the summer construction conditions that thus causes damages following placement. The proposed model theoretically simulates the effect of plastic sheet on concrete temperature development.
- The proposed temperature prediction model is verified by E17 and E313 projects. The predicted temperature shows a satisfying match with field measured data.
- A reliable procedure to generate high quality climate input parameters is proposed. The proposed procedure has the advantages of using very limited inputs to generate high quality climatic inputs for the temperature prediction model.
- It should be mentioned that the moisture and the evaporation cooling are not included in this early age concrete temperature prediction model. The influences of these two terms have to be further verified by laboratory and field tests.
Concrete for pavements is mixed, placed, and cured at fluctuating temperatures under field conditions and will therefore exhibit different development rates of the early age mechanical properties, compared to those measured from laboratory-cured specimens. In order to more accurately evaluate the cracking potential in a concrete pavement due to thermal and shrinkage stresses that develop during hardening, the evolution of the early age mechanical properties is formulated as a function of the degree of hydration. In combination with the concrete pavement temperature prediction model described in the previous chapter, the degree of hydration enables the correlation between the concrete properties in field and laboratory conditions during hardening.

Because notches are commonly placed in the concrete pavement at a very early age to prevent randomly initiated cracks due to thermal and shrinkage-induced tensile stresses, in addition to the mechanical properties tensile strength and elastic modulus, also the evolution of fracture energy and fracture toughness has to be determined to evaluate the cracking behaviour of the notched concrete pavement. In contrast to the commonly used three-point bending test and wedge-splitting test, a deformation-controlled uniaxial tensile test on un-notched specimens (Hordijk 1991; van Mier 1996; Erkens 2002) is used in the present study. Through the measured complete stress-deformation relation, all the needed mechanical properties of tensile strength, elastic modulus, and fracture energy and fracture toughness are obtained.
5.1 **UNIAXIAL TENSILE TEST**

5.1.1 **Specimen Shape and Dimension**

In general, prismatic or cylindrical specimens with notches were considered as one of the most reliable method to perform deformation controlled uniaxial tensile tests (van Mier 1996). Although concrete is rather notch insensitive because, especially in coarser mixes, the heterogeneous structure can cause higher tensile stresses than the finer mixtures, sufficiently deep notches will cause the crack to initiate at a known location, namely the location of the notch. As a result, the locations for the control gauges are known, because if they are positioned away from the notches they will be outside the crack area. However, the adopting notch in the specimen of a uniaxial tensile test has a number of drawbacks. Firstly, stress concentrations around the notch are quite large and a significant deviation from the pure uniaxial tensile state of stress, that was intended in this test and which is of great interest to explain the failure processes (Li et al. 1993; van Mier 1996; van Vliet 2000), occurs. Secondly, notches force the major crack to form at a predetermined location, the notch may be situated in a stronger or weaker part of the specimen by chance, which will lead to the observed tensile strength of the concrete to be either higher or lower than the actual strength (van Mier 1996). Moreover, a practical problem that the early age concrete is still weak and easily damaged when trying to make a saw cut or removing it from a mould with an extension to cast a notch arises during the manufacturing of the notch in the early-age concrete (Tang et al. 1996; van Vliet 2000). Since in this part of the research the properties of concrete as a function of age will be determined, the decision was made to use un-notched in this study. Cylindrical un-notched specimens reintroduce the problem that it is unknown where the crack will initiate, which means elaborate instrumentation is needed to control the test and measure the response. Alternatives are so-called “dog-bone” or parabolic shaped specimens, which localize the crack in the area with the smallest diameter. In this project the parabolic shape as used by Erkens (2002) is used.

As shown in Figure 5.1, the specimen surface is a second-order or parabolic curve that does not have a curvature variation along the entire height of the specimen; therefore reducing the stress concentration and allowing a better strain assessment. The specimen has the largest diameter of 80 mm at the ends and it is gradually reduced to 50 mm at the centre of the specimen. The nearly 60% reduction of the cross-section area is large enough to initiate the crack at the centre to minimize the boundary effects due to the restrained contraction at the glued end caps. It should be noted that the choice of the selected dimensions of the specimen in this study is mainly based on the available test setups, moulds, and steel caps. For the typical mixtures applied in two-lift concrete pavements in Belgium, the maximum aggregate size $d_{max}$ of the top layer is 6.3 mm while it is 32 mm for the
bottom layer. Normally, the smallest dimension of the specimen should be taken at least 3 times \( d_{\text{max}} \) to minimize the boundary effects. Therefore, the aggregate size larger than 16 mm of the bottom layer concrete is removed from the mixture, while the water/cement ratio, cement content, and sand ratio remain the same as in the original mixture, and the gradation is adjusted according to the Fuller curve, as shown in Figure 3.5. The details of the concrete mixture can be found in Table 3.1. The radius of the cross section area of the parabolic specimen at any depth \( z \) can be calculated by means of the following equation:

\[
R(z) = \frac{1}{135} z^2 + 25
\]

(5.1)

Where,

\[
R(z) = \text{radius of the cross section area of the specimen (mm) at the depth } z \text{ (mm)};
\]

\[
z = \text{the depth of the specimen in the ordinate system as shown in Figure 5.1.}
\]

Figure 5.1 The concrete specimen shape and dimension, in mm.

### 5.1.2 Specimen Preparation

The parabolic shaped specimen is cast by an additional splitting mould inside the standard gyratory mould that is commonly used in asphalt concrete research, as shown in Figure 5.2(a). The diameter of the adopted gyratory mould is 100 mm. The custom designed inner mould consists of two halves, and its internal surface has the same parabolic shape as the specimen, as shown in Figure 5.2(b). The inner split mould is firstly assembled by connecting the two halves by four screws. It is then rigidly connected to the gyratory mould by means of four additional screws.

The specimen production is according to ASTM C192/C192M-07 ‘Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory’ and
ASTM C305-06 ‘Standard Practice for Mechanical Mixing of Hydraulic Cement Pastes and Mortars of Plastic Consistency’. The weight of each batch concrete mixture is 2.6 kg and it is mixed in a mixing bowl with a capacity of 20 Litre. Because of the varying cross section of the mould, sufficient compaction of the specimens was achieved by pre-compaction through manual rodding and subsequently placing the mould on an external vibrating table. The concrete is placed in three layers. Firstly, rod each layer throughout its depth and consolidate it through the vibrating table during 15 seconds. When the top layer of concrete was placed and rodded, a final consolidation was performed through 3 minutes external vibration. Because of the low slump (less than 20 mm) of the used concrete mixture, no segregation was found. After the compaction, the sample is cured at 20°C and 100% relative humidity.

Figure 5.2 Preparation procedures of the parabolic-shape concrete specimen.
The specimen is demoulded within 24 hours from the time of mixing. After the external screws that connected the gyratory and the inner split mould are removed, the concrete specimen together with the split mould is pressed out from the gyratory mould by means of a jack, as shown in Figure 5.2(c). After this, the split mould is detached from the concrete specimen. The specimens that are not tested at the day of the demoulding are immediately stored in a curing room at 20 °C and 100% relative humidity.

At the predesigned test day, the excess material outside the parabolic shape is cut off. To protect the specimen, especially the specimens tested at very early ages (1, 2, and 3 days) against being damaged during the cutting process, the specimen is placed in a cutting mould. Another benefit of using the cutting mould is to make both cutting surfaces parallel to each other. The cutting mould is a Perspex mould with the same shape as the split mould, as shown in Figure 5.2(e). The cutting mould is bolted to the sawing table during sawing and provides continuous support of the specimen.

After cutting the excess material, the specimen is glued to the end caps that are used to transfer the load to the specimen in the uniaxial tensile test. Despite the use of the cutting mould, the acquired cutting surface can still be somewhat non-parallel (up to 1.0 mm as appeared from measurements) due to the slight movements of the cutting blade and some freedom in positioning it, especially when cutting the last portion of the excess material. Zhou (1988) has concluded from numerical simulations that the load eccentricity has a significant effect on the softening behaviour of concrete. To prevent gluing the caps non-parallel, a gluing mould is therefore used. In this mould, the centre of the specimen rather than one of the cutting surfaces is used as a reference. The specimen is clamped at its centre, using a PVC ring of 40 mm height that has the same shape as the concrete specimens, as illustrated in Figure 5.2(f).

To prevent de-bonding of the specimen and the steel cap before concrete failure, a sufficient bond strength and shorter installation time is needed for the adhesive material. After comparing the pot life (pot life is the time between mixing component of the adhesive at 20°C until jellying) and bond strength of different adhesive materials, X60, a methacrylate based adhesive material, is used. It is a two component cold curing glue and suitable for use with deformation gauges on concrete. The pot life is about 3 minutes at room temperature that is long enough to allow complicated installations and short enough to avoid costly waiting times. The installation quality depends on the preparation of the adhesive surfaces or the measuring point. A slightly rough surface is an ideal anchorage for the adhesive and it can be obtained by sandblasting. All the dirt particles and dust have to be carefully removed. The specimen should be left dry at room temperature and gluing the specimen with the cap should not be done when the surface of the concrete is wet. In total, it takes about 2 hours to demould, cut, sand blast, dry, and
glue a specimen. To ensure that the glue can correct any deviation of the cutting surface, more glue than necessary is used. This allows the glue to fill any gap there might exist between cutting surface and cap. Excess glue spills out between cap and specimen. In order to prevent glue spilling on the topside of the specimen and the bottom cap, both are covered with tape. The glue spills onto the tape and is later removed together with the tape. Lastly, the used glue and gluing procedure has been proven very successful by the experimental results as none of the specimens was fractured at the specimen surface.

5.1.3 Test Set-up

The TU Delft tension set-up with 3 hinges developed in the Asphalt Concrete Response (ACRe) project is used in this study (Erkens 2002), as shown in Figure 5.3. The boundary rotations influence the stress-crack opening curve in uniaxial tensile experiments and thus influence the crack development (van Mier 1996). Three types of boundary rotation conditions are applied in uniaxial tensile experiments: fixed, free, and mixed (the specimen is in between a fixed and a rotating plate). The last type of boundary condition is not very favourable because the state of stress is rather complex. Under fixed boundary conditions, the experiment initiate a crack opening from the weakest side of the specimen, but because the specimen boundaries are forced to remain parallel during crack propagation, a closing bending moment develops. The effect of the bending stiffness of the loading frame is directly incorporated in the experimental results. The descending branch of the force-deformation curve under fixed boundary condition shows irregularities (Reinhardt 1984; van Mier 1986 and 1996; Hordijk 1987 and 1991). The free rotatable boundary condition (hinges) ensures a centric load application. The centric load leads to almost uniform deformations on all sides of the specimen in the pre-peak region, and with further increasing average deformation, the crack opens while the deformation on the opposite side decreases until compressive stresses act in that part.

Among the free rotatable boundary conditions, two hinges (one at either side of the specimen) are used by most investigators (Rusch and Hilsdorf 1963; Scheidler 1987; van Mier et al. 1994 and 1995). Ideally, this would be sufficient to ensure the force is applied along the specimen axis, but in reality, the system may not be flexible enough to accommodate imperfect specimens, such as not parallel surfaces. Two hinges give rotational freedom once a crack occurs, but they do not compensate slight misalignments. To compensate for misalignment, Erkens (2002) introduced a third hinge, which is able to move horizontally to accommodate possible small deviations.
The tension set-up consists of a rigid loading frame in which an MTS 50 kN hydraulic actuator is mounted. The actuator is connected rigidly to the bottom plate. The bottom plate is a sandwich structure of a 600×330×110 mm steel plate bolted onto a 600×600×50 mm steel plate. The top plate provides the connection to the vertical bars and stiffens the bottom plate to the extent necessary to support the actuator. Two vertical bars of 100 mm diameter stainless steel are connected to the bottom plate via bolts. A top transverse steel frame with dimensions 530×150×200 mm is clamped onto the vertical bars. The upper hinge is placed directly on the transverse frame. A 50 kN load cell is positioned underneath the upper hinge and immediately below the load cell is the middle hinge. The lower hinge is placed on the actuator. The specimen is connected between the middle and lower hinges (Erkens 2002).

5.1.4 Specimen Installation

In order to achieve a stable deformation-controlled uniaxial tensile test, the energy required for crack growth must remain larger than the energy released by the unloading of both the uncracked parts of the specimen and the loading frame. If that is not the case, the crack will grow explosively, resulting in a vertical unloading branch or even a snap back (Hordijk 1991). In addition to the stiffness of the loading frame, Hordijk also pointed out the influences of the measuring length on the stress-deformation relations in a uniaxial tensile test of concrete. A longer measuring length results in more uncracked material that unloads after the peak and this yields a snap back in the stress-deformation in the post peak region. In this
study, the loading frame (as seen in Figure 5.4) is much stiffer than the specimen to limit the energy stored in the frame.

In the concrete laboratory of the Stevin Laboratory of TU Delft, a control length of 35 mm (Hordijk 1991; Schlangen 1993) and 75 mm (van Vliet 2000) both have proven to give stable results in uniaxial tensile tests of concrete. In the ACRe project, there is no straight part of the parabolic surface specimen. Thus, the full specimen length of 90 mm is used as control length for the asphalt concrete specimen, while strain gauges on between the cap and the specimen centre are used to register the unloading after the peak (Erkens 2002). The same arrangement for the control length is used in these tests. Three LVDTs with a measurement length of 90 mm are installed at 120° intervals. One type of LVDT, Solartron AG 1, with a measuring capacity of ± 1 mm is used. The connection of the LVDTs to the caps is by means of a set of rings. One ring has three holes at 120° interval in which the LVDTs are clamped, while another is a massive ring on which the LVDTs are placed. After the specimen is placed between the lower two hinges and instrumented with LVDTs, a small pre-load (approximately 0.2 kN) is applied to ensure that there is no play in the setup components, more specifically, the hinges.

Figure 5.4 Specimen instrumented with LVDTs connected on the caps.

5.1.5 Test Conditions

In order to study the evolution of the concrete properties during hardening, the uniaxial tensile tests are performed at different concrete ages, 1, 2, 3, 5, 7, 14, 28, and 90 days for each mixture. At least three samples are prepared for each age. All the specimens are cured at 20°C and 100% relative humidity. The average axial deformations measured with the 3 LVDTs were used as feedback control signal for test control. The applied deformation rate was 0.1 μm/s for the specimens older than 3 days. However, specimens younger than 3 days suddenly broke just after the peak load under this deformation rate. Thus, for these specimens the deformation rate was reduced to 0.05 μm/s later. After the specimen is taken out of the
curing room, the sample is in laboratory conditions at approximately 50% relative humidity and room temperature, which was around 20°C during the periods of the preparation and installation works, and the subsequent test period. In the subsequent analysis, the concrete specimens with the fine aggregate (maximum size 6.3 mm) and the coarse aggregate (maximum size 16.0 mm) are called specimen type 6.3 and type 16, respectively.

5.2 UNIAXIAL TENSILE TEST RESULTS

5.2.1 Experimental Data

Figure 5.5(a) presents the raw experimental data of a specimen type 6.3 at an age of 5 days. In the pre-peak regime, the specimen behaves linearly elastic up to 50% of the peak load as shown by the measured deformation of all three LVDTs and the measured force. Further, the curves become non-linear due to permanent plastic deformation until the peak force. Just before the peak, LVDT number 1 and LVDT number 2 give a positive deformation and the largest deformation is measured with LVDT number 2. It indicates that the fracture initiates near LVDT number 2. At the other side of the specimen, near LVDT number 3, a compressive deformation (negative) is measured. When the total deformation increases, the crack grows rapidly into the direction of the LVDT number 3. Thus, the trend reverses: the tensile deformation develops between LVDT number 2 and 3, while the deformation near LVDT number 1 changes from tensile to compressive. The uniaxial tension tests were not continued until the complete separation of the two specimen halves. Because of the applied free rotation boundary condition, the crack opening is non-uniform along the perimeter of the specimen, with tensile deformation at one side while compressive deformation develops on the opposite side of the specimen. Besides, the average measured deformation of the three LVDTs is used as control signal. Due to the limited measuring range of the used type of LVDT, Solartron AG 1 with a measuring capacity of ±1 mm, one or even more LVDTs may reach to their range when the total average deformation exceeds 0.6 mm. Thus, continue to use this average measured deformation is not adequately anymore after one or more LVDT reach their range. As shown in Figure 5.5(b), the descending branch of the average measured deformation approaches the horizontal axis very gradually. Most of the experiments were stopped at an average deformation of 0.6 mm. Figure 5.6 shows the crack development of a specimen tested by the deformation controlled uniaxial tensile test in this study. It clearly indicates that the crack surfaces are not parallel when the crack is growing under the free boundary conditions.
The fracture surface is also changing significantly with the age of the concrete and the type of the aggregate. At an early age, for instance 24 hours, absolutely no aggregates are fracturing, which makes the fracture surface very rough, especially for the specimen type 16 with coarser aggregate. The fracture surface is much smoother at later ages, when the cement paste hardens becomes as strong as or stronger than the aggregates. As a result, the crack does not always go around the aggregates, but can also go through it. The resulting surface is smoother due to the pronounced fracture of the aggregates.

Figure 5.6 Crack development during deformation controlled uniaxial tensile test.
Figure 5.7 Fracture surface of specimens tested at increasing ages, (a) for specimen type 6.3; (b) for specimen type 16.

Figure 5.8 and Figure 5.9 present the measured force-deformation curves for the specimen type 6.3 and the specimen type 16 at each individual testing age, respectively. The peak load for the specimen type 6.3 with the finer maximum aggregate sizes is slightly larger than that for specimen type 16 at the same testing age.
Figure 5.8 Overview of the force-deformation curves for specimen type 6.3 that were fractured at the center of the specimen as a function of testing ages. ‘D1’ represents the specimen type 6.3 tested at an age of 1 day after casting; ‘-6’ represents the order number of the specimen for each specific specimen type and each specific age.

The curves indicate that the failure load and the slope of the elastic part increase with age. Increase in the elastic slope can be attributed to the evolution of the Young’s modulus with time. The rapid fall of the curve after the peak load shows that concrete is a brittle material and that it becomes more brittle when it ages.
Figure 5.9 Overview of the force-deformation curves for specimen type 16.0 that were fractured at the center of the specimen as a function of testing ages. ‘B1’ represents the specimen type 16.0 tested at an age of 1 day after casting; ‘-2’ represents the order number of the specimen for each specific specimen type and each specific age.
5.2.2 Correction of Experimental Data

Deformation data

In the pre-peak region, the average measured LVDTs deformation includes three components: specimen, caps, and glue layers. If a single stiffness is assumed for the concrete specimen, the deformation of the specimen in the pre-peak region can be calculated as follows:

\[
\delta_{\text{total}} = \delta_{\text{cap}} + \delta_{\text{specimen}} + \delta_{\text{glue}}
\]

\[
= \int_0^{h_{\text{cap}}} \frac{F}{E_s A_s} \, dh + \int_0^{h_{\text{specimen}}} \frac{F}{E_c A_z} \, dh + \int_0^{h_{\text{glue}}} \frac{F}{E_g A_g} \, dh
\]

(5.2)

Where,

- \(\delta_{\text{total}}\) = the total average deformation measured by LVDT, [mm];
- \(\delta_{\text{cap}}\) = the deformation of caps, [mm];
- \(\delta_{\text{glue}}\) = the deformation of glue layers, [mm];
- \(\delta_{\text{specimen}}\) = the deformation of concrete specimen, [mm];
- \(F\) = applied force, [N];
- \(E_c\) = Young’s modulus of concrete, [N/mm²];
- \(E_s\) = Young’s modulus of steel, 210000 [N/mm²];
- \(E_g\) = Young’s modulus of glue, 5000 [N/mm²];
- \(h_{\text{cap}}\) = distances from the centre of clamped ring to the edge of concrete specimen, normally, 20 [mm];
- \(h_{\text{specimen}}\) = the height of concrete specimen, 90 [mm];
- \(h_{\text{glue}}\) = the total thickness of glue layers, around 2 [mm];
- \(A_s\) = area of cross section of cap, 5026.548 [mm²];
- \(A_g\) = area of cross section of glue layer, 5026.548 [mm²].

The elastic modulus of the steel cap and the glue is 210000 MPa and 5000 MPa, respectively. The thickness of the two glue layers together ranged from 1.02 mm to 3.42 mm, with an average value of 2.21 mm. The measuring length of the LVDTs on two steel caps is approximately 20 mm. Figure 5.10 shows the percentage of the specimen deformation to the measured total deformation by the LVDTs. It shows the influence of the concrete modulus, the thickness of the glue layers, and the measuring distance on the steel caps on the deformation measurement, and it is very sensitive to the first two variables. The higher the concrete modulus and the thicker the glue layer, the smaller the percentage of the specimen deformation to the measured overall deformation. Under the most favourable conditions, the modulus of concrete is 20000 MPa for an age of 1 day and a thin glue layer of 1.0 mm is applied together with a short measuring distance of 10 mm on the steel caps. The deformations of the steel caps and the glue layer then account for 2.83% of the measured deformation by the LVDTs. Furthermore, those deformations due to caps and glue layers will rise up to 14.88% of the measured overall deformation for an extreme condition: a matured concrete having a modulus of 40000 MPa, 1.5 mm thick glue layers, and 30 mm of measuring distance on the steel caps. Thus, it
clearly shows that the influence of the caps and the glue on the measurement cannot be considered as negligible. The raw experimental deformation data therefore has to be corrected according to Equation (5.2).

![Graphs showing the influence of elastic modulus of concrete $E_c$, the thickness of glue layers $h_{\text{glue}}$, and the measuring distance on steel caps $h_{\text{cap}}$ on the measured concrete specimen deformation.]

**Figure 5.10** Influences of elastic modulus of concrete $E_c$, the thickness of glue layers $h_{\text{glue}}$, and the measuring distance on steel caps $h_{\text{cap}}$ on the measured concrete specimen deformation.

**Force Data**

The load is measured by a load cell that was placed underneath the top hinge and below the load cell is the middle hinge, as shown in Figure 5.3. Before the middle hinge and the specimen with loading caps were placed in the experimental set-up, the load was calibrated to zero. Thus, the weight of the middle hinge, the top loading cap, and the weight of the specimen above the final crack are still included in the external force recorded by the load cell. It has therefore to be subtracted. Based on the force equilibrium in the vertical direction as presented in Figure 5.11, the applied tensile force $F_{cr}$ on the fracture surface is determined by subtracting the
weight of the middle hinge, the top cap, and the upper half of the concrete specimen from the measured force $F_m$ by the load cell.

$$F_m = F_{cr} + F_{sw} \quad (5.3)$$

Where,

- $F_m$ = measured external force by load cell, [kN];
- $F_{cr}$ = applied external force on the fracture face of specimen, [kN];
- $F_{sw}$ = dead load of the middle hinge, the top cap, and the upper half of the concrete specimen. In this study, $F_{sw} = 0.15$ kN.

The self-weight term of 0.15 kN was found for almost all the tested specimens in the present study. It is equal to a stress of 0.08 MPa that is about 3% of the concrete tensile strength during the hardening phase. However, previous research has pointed out that the value of the fracture energy $G_f$ is very sensitive to an incorrect load measurement: an error of the zero load equal to 0.03 MPa leading to 10% error of the determined fracture energy was reported by Hordijk (1991). The effect of the self-weight term will be further outlined in the subsequent section on fracture energy determination.

Figure 5.11 Axial force equilibrium on the fracture surface.
Correction for the $F$-$\delta$ Relation

The average raw experimental $F$-$\delta$ relation of a specimen with an age of 5 days is illustrated in curve $a$ in Figure 5.12. Because of the flexibility of the hinges, the origin of the $F$-$\delta$ relation could not be determined by a compression test in which a deformation at zero load level is used to correct the deformation. In the present study, the origin of the $F$-$\delta$ relation is retrieved through back analysis. Firstly, the deformation of the specimen has been corrected though subtracting the deformation of the caps and the glue layers from the raw experimental data (curve $b$). Similarly, the external force applied on the fracture face has also been corrected by excluding the self-weight. The initial slope of the correction curve (curve $c$) was linear extrapolated until the intersection point with the axis of the deformation. Thus, this intercept deformation value is then used to define the corrected $F$-$\delta$ relation through origin (curve $d$).

Figure 5.12 Schematic view of corrections made on the experimental data.

Figure 5.13 Schematic view of deformation components for a complete deformation controlled uniaxial tensile test.
Concrete behaviour in a deformation controlled uniaxial tensile test

Initially, when increasing the deformation, the concrete material behaves elastically. In the pre-peak regime, a linear force-deformation relation almost up to the peak load is obtained. At the peak load, the strains start to localize within a narrow zone of micro cracks at the weakest section of the tensile specimen. After that a macro crack will develop. The total deformation of the specimen is composed of three components: namely instantaneous elastic, visco-elastic and permanent plastic deformation. Among those, the viscous elastic deformation for pavement concrete materials can be considered as negligible. In addition, the measured deformation includes the crack opening as well in the post crack regime (Figure 5.13).

In the pre-peak regime:

\[ \delta = \delta_p + \delta_e = \delta_p + \int_0^{h_{\text{specimen}}} \frac{F}{E_c A_z} \, dh \]  

(5.4)

At the peak load:

\[ \delta_u = \delta_{pu} + \delta_e = \delta_{pu} + \int_0^{h_{\text{specimen}}} \frac{F_u}{E_c A_z} \, dh \]  

(5.5)

In the post-peak regime:

\[ \delta = \delta_{pu} + \delta_e + w = \delta_{pu} + \int_0^{h_{\text{specimen}}} \frac{F}{E_c A_z} \, dh + w \]  

(5.6)

Where,

\[ \delta \] = the corrected average overall deformation of the specimen measured by three LVDTs, [mm];
\[ \delta_e \] = elastic deformation of the specimen under load level \( F \), is estimated by a simple stiffness along the specimen, [mm];
\[ \delta_p \] = plastic deformation of the specimen under load level \( F \) in the pre-peak regime, [mm];
\[ \delta_u \] = the corrected ultimate measured overall deformation of the specimen at the peak load, [mm];
\[ \delta_{pu} \] = plastic deformation of the specimen under peak load \( F_u \), [mm];
\[ w \] = crack width, [mm];
\[ F \] = the corrected load on the fracture surface, [N];
\[ F_u \] = the corrected peak load on the fracture surface, [N];
\[ E_c \] = the calculated elastic modulus of concrete by the measured force-deformation curves in the pre-peak regime, [MPa].

The Equations (5.4) and (5.5) are input into Equation (5.6), and the calculated integral \[ \int_0^{h_{\text{specimen}}} \frac{1}{A_z} \, dh = 0.03382 \text{ mm}^{-1} \] according to the radius equation of the specimen that is illustrated in Equation (5.1). Thus, the crack opening \( w \) is calculated as follows:
\[ w = \delta - \delta_{pu} - \delta_e \]
\[ = \delta - \left( \delta_u - \int_0^{h_{\text{specimen}}} \frac{F_u}{E_c A_z} dh \right) - \int_0^{h_{\text{specimen}}} \frac{F}{E_c A_z} dh \]
\[ = (\delta - \delta_u) + 0.03382 \frac{F_u - F}{E_c} \]

For each testing age and each specimen type, at least three successful tests (with a complete softening curve) were conducted. During the experimental program, some specimens were fractured during the demoulding or cutting phases, and a few of them were extracted due to the incorrect test operations by fault (forgetting to turn on the data acquisition system). Another few specimens fractured near the caps that had an extremely low peak load compared to the results of the same test series, as shown in Figure 5.14. The possible reasons of those specimens fractured near the cap were attributed to unnoticed pre-existing cracks during the specimen preparation procedures or during the compaction. The diameter of the fracture surface of the specimen was calculated by Equation (5.1) using the average measured height of the fractured specimen at 8 locations along the perimeter. The specimens with a diameter of the fracture surface larger than 60 mm, indicating the specimen fractured near the caps, were eliminated in the further analyses. Table 5.1 and Table 5.2 present for the remaining specimens the calculated diameter of the fracture surfaces, the measured peak load and the corresponding total deformation at the peak load for the specimens at each testing ages.

![Different location of the fracture plane along the specimen.](image)
Table 5.1 Experimental data for the tests on the concrete specimen type 6.3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$D_F$ mm</th>
<th>$F_u$ kN</th>
<th>$\delta_u$ μm</th>
<th>$E_c$ MPa</th>
<th>$f_c$ MPa</th>
<th>$\sigma_N$ MPa</th>
<th>$G_{160}$ N/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>UTT-D1-5</td>
<td>53</td>
<td>6.56</td>
<td>2.87</td>
<td>21500</td>
<td>1.46</td>
<td>1.30</td>
<td>0.054</td>
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<td>4.34</td>
<td>23000</td>
<td>2.21</td>
<td>2.21</td>
<td>0.078</td>
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<tr>
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<td>4.82</td>
<td>26000</td>
<td>2.45</td>
<td>2.27</td>
<td>0.078</td>
</tr>
<tr>
<td>UTT-D2-2</td>
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<td>7.71</td>
<td>5.87</td>
<td>30000</td>
<td>2.99</td>
<td>2.99</td>
<td>0.082</td>
</tr>
<tr>
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<td>56</td>
<td>8.61</td>
<td>6.04</td>
<td>30000</td>
<td>3.08</td>
<td>2.45</td>
<td>0.086</td>
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<td>3.83</td>
<td>0.087</td>
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Table 5.2 Experimental data for the tests on the concrete specimen type 16.

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<th>$δ_u$</th>
<th>$E_c$</th>
<th>$f_t$</th>
<th>$σ_N$</th>
<th>$G_{160}$</th>
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<td></td>
<td>mm</td>
<td>kN</td>
<td>μm</td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
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<td>3.34</td>
<td>0.082</td>
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<td>3.66</td>
<td>3.39</td>
<td>0.061</td>
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<td>0.080</td>
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<td>46000</td>
<td>4.44</td>
<td>4.44</td>
<td>0.072</td>
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5.2.3 Determination of Tensile Strength

Nominal failure stress and nominal tensile strength

In this uniaxial tension test of un-notched specimens, the nominal failure stress $σ_N$ can be calculated from the recorded maximum tensile load $F_u$ divided by the cross section area of the fracture surface $A_c$. It should be noted that the calculated $σ_N$ is a measure for the force level whereas it does not necessarily represent an actual stress. $A_c$ is the projected area of the fracture plane perpendicular to the stress direction. The diameter of the fracture plane is $D_p$ calculated by the second order parabolic function with the measured average height of the fracture surface, as shown in Figure 5.1. Moreover, the nominal tensile strength $f_t$ is defined as the recorded maximum tensile load $F_u$ divided by the smallest cross section area $A_{min}$. $A_{min} = 1962.5 \text{ mm}^2$ is used, as the smallest diameter of the specimen is 50 mm.

The nominal failure stress $σ_N$ and the nominal tensile strength $f_t$ are thus calculated as follows:

$$σ_N = \frac{F_u}{A_c}$$

(5.8)
\[ f_t = \frac{F_u}{A_{min}} \]  

(5.9)

The development of the tensile strength

Table 5.3 tabulates the mean values and the standard deviations of the nominal failure stress and the nominal tensile strength as a function of the age of both specimen type 6.3 and type 16. It can also be seen in Figure 5.15. For both types of specimens, the \( \sigma_N \) and \( f_t \) increase with increasing age of the specimen until a certain age of the specimen, about 28 days in the present study. After this age, the \( \sigma_N \) and \( f_t \) reach to a relative constant value. The nominal tensile strength \( f_t \) of type 6.3 specimen at an age of 1 day stored in the laboratory is 2.04 MPa, while this value is 4.79 MPa at the age of 90 days that is nearly 2.4 times the value at 1 day. A similar tendency is also observed for the specimen type 16. Table 5.3 also shows that the nominal failure stress is always smaller than the nominal tensile strength at the corresponding age for both types of specimen. It is easy to understand due to the existence of an inevitable weak point in the concrete specimen. Thus, the specimen is normally fractured before reaching its nominal tensile strength when it is under tension. Moreover, it is also shown that both the nominal failure stress and the nominal tensile strength of the specimen type 6.3 are larger than the values of the specimen type 16. It will be further outlined in the subsequent tensile strength scatter analysis.

Table 5.3 The nominal failure stress and nominal tensile strength, as a function of the specimen age.

<table>
<thead>
<tr>
<th>Age (day)</th>
<th>6.3</th>
<th>16</th>
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<tr>
<td>( \sigma_N ) Mean (MPa)</td>
<td>1.93</td>
<td>1.50</td>
</tr>
<tr>
<td>SDEV (MPa)</td>
<td>0.54</td>
<td>0.08</td>
</tr>
<tr>
<td>Number</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>( f_t ) Mean (MPa)</td>
<td>2.04</td>
<td>1.63</td>
</tr>
<tr>
<td>SDEV (MPa)</td>
<td>0.52</td>
<td>0.10</td>
</tr>
<tr>
<td>Number</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

Type 6.3 specimens show a smaller scatter of the measured nominal tensile strength as compared to that for type 16 specimens. This difference in test result scatter may be interpreted by the large homogeneity of the type 6.3 specimen having a higher aspect ratio of \( D/d_{max} = 8.3 \) for the type 16 specimens. Therefore, type 6.3 specimens will be less sensitive to the presence of local inhomogeneity, such as weak spots at the interface of the aggregate and the mortar. For instance, a 16 mm aggregate is occasionally present in the middle of the specimen that has a
width of only 50 mm. Such an aggregate distribution will have a distinct wall effect on both the stiffness and the tensile strength of the specimen.

Figure 5.15 Mean values and standard deviations of nominal failure stress and nominal tensile strength vs. the age of the specimens of both specimen type 6.3 and type 16.

5.2.4 Determination of Modulus of Elasticity

As discussed in Section 5.2.2, the relation between the measured deformation rate and the strain rate in the specimen is not straightforward due to the specimen shape. However, if a single stiffness is assumed for the whole specimen, the stiffness can be found on basis of the stress distribution. The modulus of elasticity of concrete $E$ under tension is calculated according to CRD-C 166 (1992), ‘Standard Test Method for Static Modulus of Elasticity of Concrete in Tension’. The chord modulus of elasticity is used as the modulus of elasticity, that is the slope of the chord drawn between 10% and 50% of the ultimate load on the pre-peak $F\sim\delta$ curve, and the results are rounded to 500 MPa. Limited data suggests that the modulus of elasticity in tension may not be significantly different from that in compression (CRD-C 166 1992).
\[ E_c = \frac{F_{0.5} - F_{0.1}}{u_{0.5} - u_{0.1}} \int_{0}^{\frac{h_{\text{specimen}}}{A}} \frac{1}{A_z} \, dh \]  

(5.10)

Where,
\[ F_{0.5} = 50\% \text{ of the ultimate load after correction for self-weight, [N]}; \]
\[ F_{0.1} = 10\% \text{ of the ultimate load after correction for self-weight, [N]}; \]
\[ u_{0.5} = \text{overall deformation of specimen at load level } F_{0.5}, [\text{mm}]; \]
\[ u_{0.1} = \text{overall deformation of specimen at load level } F_{0.1}, [\text{mm}]. \]

However, in practice, it was found that the calculated elastic modulus according to Equation (5.10) is strongly influenced by the fluctuation of the recorded force and deformation data because of the applied very low deformation rate, which is approaching the resolution of the used LVDTs. Therefore, a much more reliable method through the linear curve fitting technique using the force and deformation data between 10\% and 50\% of the ultimate load rather than two individual data points was used to calculate the elastic modulus of the concrete, and an example is shown in Figure 5.16.

![Figure 5.16](image)

Figure 5.16 Elastic modulus determination by curve fitting using the data between 10\% and 50\% of the ultimate load on the pre-peak \( F \sim \delta \) curve.

Table 5.4 tabulates the mean values and the standard deviations of the elastic modulus of concrete as a function of the age of both specimen type 6.3 and type 16. Similar to the development of the tensile strength, the elastic modulus of concrete under tension was also observed to first increase with the age for both types of specimens. After about 7 days, it became relatively constant. For specimen type 6.3, the calculated elastic modulus of concrete of an age of 1 day stored in the laboratory is 23500 MPa, while this value is 43500 MPa at the age of 90 days that is about 1.9 times of that for 1 day. With respect to the influence of the aggregate size on the elastic modulus of concrete under tension, there is no significant difference
between these two types of specimens, as shown in Figure 5.17. It is also observed that the scatter of the calculated elastic modulus under tension is smaller than that of the tensile strength. For instance, the coefficient of variations of the measured $E_c$ of the specimen type 6.3 varies between about 0.7 to 9.7%, and about 3.6 to 25.5% for $f_t$.

Table 5.4 Concrete elastic modulus under tension vs. the age of specimen as a function of the specimen age.

<table>
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<th>Age (day)</th>
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<th>2</th>
<th>3</th>
<th>5</th>
<th>7</th>
<th>14</th>
<th>28</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean (MPa)</td>
<td>23500</td>
<td>31200</td>
<td>32750</td>
<td>37300</td>
<td>37800</td>
<td>38833</td>
<td>38100</td>
<td>43500</td>
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<td>1956</td>
<td>645</td>
<td>2842</td>
<td>2361</td>
<td>289</td>
<td>3560</td>
<td>2000</td>
</tr>
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<td>4</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
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<td>34500</td>
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<td>38833</td>
<td>39167</td>
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<td>2</td>
</tr>
</tbody>
</table>

Figure 5.17 Mean values and standard deviations of concrete elastic modulus under tension vs. the age of the specimens of both specimen type 6.3 and type 16.

### 5.2.5 Determination of Fracture Energy

In a deformation controlled uniaxial tensile test on concrete, the fracture energy $G_f$ is calculated as the applied work of energy per unit fracture area, and it can be illustrated as:

$$G_f = \frac{W_F}{A_c} = \frac{\int_0^{w_c} F_{cr} dw}{A_c} \quad (5.11)$$

Where, $W_F$ [N/mm] is the work of the fracture that is defined as the area under the post peak of force-crack opening diagram according to the procedure of Figure 5.13. $F_{cr}$ [N] is the applied external force on the fracture surface of the concrete specimen, and it is determined as explained in the Section 5.2.3. $A_c$ [mm$^2$] is the area of the fracture surface that is estimated as illustrated in Section 5.2.4 consider-
ing the tortuosity of the fracture surface. Lastly, \( w \) and \( w_c \) [mm] are the crack opening of the fracture surface and the critical crack opening at which stress is no longer transferred, respectively. Problems that arise in the determination of the fracture energy according to the RILEM recommended three-point bend notched beam test were already addressed by Peterson (1980); Elices et al. (1992), which will be further outlined in Section 5.3.4. Here the procedures to calculate the fracture energy from the uniaxial tensile tests in the present study will be explained.

![Stress and crack width curve](image)

Figure 5.18 Stress and crack width curve for one specimen of type 6.3 measured at the age of 5 days.

The first problem is to determine the critical crack opening \( w_c \) at which stress is no longer transferred. Generally, the uniaxial tensile tests were not continued until complete separation of the two specimen halves due to reasons discussed in Section 5.2. A long and gradual tail is observed at the end of each curve, as shown in Figure 5.18. Except a few specimens that suddenly fractured just beyond the peak load, the average deformation of three LVDTs was about 0.6 mm at zero stress for nearly all specimens under both deformation rate conditions in the present study. Hordijk (1991) concluded that the determination of the critical crack opening strongly affects the absolute value of the fracture energy \( G_f \), where the calculated energy \( G_f \) up to \( w_c = 0.6 \) mm may be about 40% more than that for \( w_c = 0.15 \) mm. One possible reason is the incorrect force measurement at the gradually increasing deformation branch. The nearly unchanging stress at large deformation, as shown in Figure 5.18, indicates that the \( w_c \) has already been passed. Another possible explanation of the long tail of the force-deformation curve might be the ‘hinge mechanism’ due to aggregates interlocking, which does not contribute to the fracture energy (Wittmann et al. 1990). In the literature, a value for \( w_c \) of about 0.5 mm was reported (Hordijk 1991). However, Hordijk further
remarked that the active friction forces in the process zone up to deformations of about 0.5 mm is not remarkable from a physical point view. In present study, a unique value $w_c = 0.16$ mm for each testing age for both types of specimens is finally chosen in the present study according to the equation in the CEB-FIP code (Hilsdorf and Brameshuber 1991; fib 1999), which correlates the critical crack opening displacement with the maximum aggregate size, concrete compressive strength. This value of $w_c = 0.16$ mm also agrees with the unbound value of the regressed expression of the concrete softening relations proposed by Hordijk, thus the corresponding calculated fracture energy is defined as $G_{f160}$. A similar value was also used for the determination of the fracture energy of concrete in uniaxial tensile tests of un-notched specimens in the study by van Vliet (2000). Apart from the above chosen critical crack opening, the fracture energy until a crack width of about 0.4 mm and 0.6 mm are also calculated, called $G_{f400}$ and $G_{f600}$, respectively (Table 5.5 and Figure 5.19).

Another problem in the determination of the fracture energy is the real area of the fracture surface. The area of the projected fracture plane is considered as underestimating the true area of the fracture surface (Nallathambi and Karihaloo 1986). It is because of the tortuosity of the crack propagation path. More specifically, the more tortuosity fracture surfaces were found for the younger age concrete specimens and the specimens with coarser aggregate, as shown in Figure 5.7. The underestimated fracture surface area will lead to overestimation of the corresponding concrete tensile strength and fracture energy. Therefore, a fracture surface correction coefficient $\alpha_{afs}$ for the area of the fracture surface introduced by Nallathambi and Karihaloo (1986) is used in the present study. A $\alpha_{afs} = 1.1$ was recommended based on their experimental data of the three-point bending tests on notched specimens.

Table 5.5 Fracture energy (N/mm) vs. the age of the specimens.

<table>
<thead>
<tr>
<th>Age (day)</th>
<th>$G_{f160}$</th>
<th>$G_{f400}$</th>
<th>$G_{f600}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3</td>
<td>Number</td>
<td>Number</td>
<td>Number</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>Mean</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SDEV</td>
<td>SDEV</td>
</tr>
<tr>
<td>16</td>
<td>$G_{f160}$</td>
<td>0.070</td>
<td>0.099</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>0.014</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>SDEV</td>
<td>0.006</td>
<td>0.007</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.016</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.014</td>
<td>0.026</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.009</td>
<td>0.017</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.011</td>
<td>0.007</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.016</td>
<td>0.012</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.007</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.004</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.009</td>
<td>0.014</td>
</tr>
<tr>
<td>1</td>
<td>$G_{f160}$</td>
<td>0.045</td>
<td>0.063</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td>0.011</td>
<td>0.017</td>
</tr>
<tr>
<td></td>
<td>SDEV</td>
<td>0.006</td>
<td>0.003</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.004</td>
<td>0.007</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.006</td>
<td>0.020</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.006</td>
<td>0.097</td>
</tr>
</tbody>
</table>
Figure 5.19 Mean values and standard deviations of fracture energy vs. the age of the specimens of both specimen type 6.3 and type 16.
5.3 EARLY AGE CONCRETE PROPERTIES BASED ON DEGREE OF HYDRATION

In order to evaluate the risk of thermal cracking in a hardening concrete pavement, adequate descriptions of the time dependent concrete properties are extremely important. However, the commonly obtained early age concrete mechanical properties through laboratory-cured specimens under relative constant curing conditions do not accurately represent the properties of field placed concrete that experiences varying temperatures. The degree of hydration is considered to be a very fundamental parameter, therefore the degree of hydration-based formulations of early age concrete mechanical properties can be considered as more applicable to describe the evolution of hardening concrete under field conditions then time-based formulations (van Breugel et al. 1998). In this way, the concrete properties of a specimen continuously stored at 20 °C and those of the pavement that has experienced a varying temperature history can be transformed into the corresponding degree of hydration for each point in time. De Schutter and Taerwe (1996) proposed general degree of hydration based formulations to describe early age concrete mechanical properties.

\[
\frac{f_t(\alpha_h(t))}{f_t(\alpha_{h_{\text{max}}})} = \left(\frac{\alpha_h(t) - \alpha_0}{\alpha_{h_{\text{max}}} - \alpha_0}\right)^a \quad (5.12)
\]

\[
\frac{E_c(\alpha_h(t))}{E_c(\alpha_{h_{\text{max}}})} = \left(\frac{\alpha_h(t) - \alpha_0}{\alpha_{h_{\text{max}}} - \alpha_0}\right)^b \quad (5.13)
\]

\[
\frac{G_f(\alpha_h(t))}{G_f(\alpha_{h_{\text{max}}})} = \left(\frac{\alpha_h(t) - \alpha_0}{\alpha_{h_{\text{max}}} - \alpha_0}\right)^c \quad (5.14)
\]

Where, \(f_t(\alpha_h(t))\), \(E_c(\alpha_h(t))\), and \(G_f(\alpha_h(t))\) are the degree of hydration based uniaxial tensile strength, the elasticity modulus and the fracture energy at the age \(t\), respectively. \(\alpha_h(t)\) is the degree of hydration of cement and \(\alpha_{h_{\text{max}}}\) is the ultimate degree of hydration depending on the water cement ratio. \(\alpha_0\) is the critical degree of hydration that will be further discussed in Section 5.3.2. The coefficients \(a\), \(b\) and \(c\) are obtained by fitting the test results of deformation controlled uniaxial tensile tests.

5.3.1 Degree of Hydration of Laboratory Cured Specimens

Based on the developed temperature prediction model in Chapter 4, the degree of hydration \(\alpha_h(t)\) of the laboratory-cured concrete specimens for each tested age \(t\) was calculated as a fraction of heat released. As mentioned in Section 5.1.2, after the concrete was mixed, the specimens were all cured at 20 °C and 100% relative humidity until the designated test age. Considering the relatively small dimension of the casted specimens, it is reasonable to assume that the hydration process of
these concrete specimens is under isothermal temperature conditions at 20 °C. The calculated degrees of hydration for both types of specimen as a function of the age are presented in Figure 5.20.

Figure 5.20 The calculated degree of hydration of the laboratory cured specimens for both types of specimens, at 20°C and 100% relative humidity.

5.3.2 Critical Degree of Hydration

The setting of concrete is a percolation process in which isolated or weakly bound particles are connected together by the formation of hydration products (Jiang et al. 1995; Bentz 2008). At initial set concrete has reached the point where it has stiffened sufficiently so that it can no longer be vibrated without damaging the concrete. The final set of concrete relates to the point where the strength and stiffness start to develop. The final set is the term of interest in the present study to calculate the early age stress due to environmental loadings. The setting time of concrete can be determined by the penetration resistance method according to ASTM C403 (2008). However, it should be noted that the setting of concrete is the gradual transition from liquid to solid. Thus, the definition of any point at which it is considered to have set is somewhat arbitrary (Neville 1996; Schindler et al. 2002).

As mentioned above, the concrete setting processes are generally related to the cement hydration process and it is therefore rational to determine the concrete set time in the field by the maturity method that takes into account the concrete temperature in the field and adjusts the set time accordingly. Initially, Byfors (1980) defined the critical degree of hydration $\alpha_{cr}$ as the amount of the hydration that has to be reached before strength and stiffness start to develop, which is similar to the definition of the final set. A correction factor ranging from 0.4 to 0.46 with the adopted water to cement ratio is recommended to calculate $\alpha_{cr}$. Pinto and Hover (1999) found a uniform equivalent age of concrete setting when concrete mixtures were tested at different temperatures according to ASTM C403, which confirms
that the concrete setting can be determined by the degree of hydration. Recently, Schindler et al. (2002) recommended the following formulation to define the degree of hydration $\alpha_f$ at the final set based on the results according to ASTM C403.

$$\alpha_f = 0.26 \frac{w}{c}$$  \hspace{1cm} (5.15)

With $w/c$ = the water to cement ratio of the concrete.

It is clearly shown that the estimated degree of hydration of concrete at final set by Schindler’s equation is lower than the critical degree of hydration calculated by Byfors’s formulation, which recommend a correction factor between 0.4 to 0.46. This difference had been well interpreted by Schindler. As shown in Figure 5.21, the critical degree of hydration $\alpha_{cr}$ is determined by extrapolating a linear line for the later age measured strength, thus the gradual initial gain in concrete strength is not considered. The exponential gain in strength can however be captured by the penetration resistance test by the ASTM C403. Therefore, the approach according to ASTM C403 and the corresponding Equation (5.15) is adopted in the present study to determine the degree of hydration of concrete at the final set.

![Diagram](image)

**Figure 5.21** Hypothesis of difference in setting degree of hydration (after Schindler et al. 2002).

The same value of $\alpha_0$ for Equation (5.12), (5.13), and (5.14) is calculated according to Equation (5.16), that is the degree of hydration at the final set of concrete, is chosen in present study. Equation (5.15) is derived based on measurements on early age concrete. Thus, the values of $\alpha_0$, for Equation (5.12), (5.13), and (5.14), of 0.107 and 0.112 are calculated for the specimen type 6.3 and type 16, respectively, in the present study according to the used water to cement ratio.
5.3.3 Degree of Hydration Described Concrete Mechanical Properties

Table 5.6 lists the mean values of the measured early age concrete mechanical properties as a function of the degree of hydration for both mixtures. The average values of the test results at 28 days are assumed as the ultimate concrete properties at the end of the cement hydration that can be achieved in practice. Because the objective of this study is to use the measured early age concrete properties to evaluate the cracking tendency at the very early age, the crossover effect of temperature on long-term strength development is not considered. The relevant value of the coefficients $a$, $b$, $c$ (Equations 5.12 to 5.14) was thus obtained by fitting the test results by mean values at each test age (as illustrated in Table 5.6), and the values of all tested specimens as shown in Table 5.1 and 5.2 for both types of specimens. The regressed curves are shown in Figure 5.22. The relevant coefficients $a$, $b$, $c$, and the ultimate values at the end of the reaction are summarized in Table 5.7. It is seen in Figure 5.22 that the variations of most of the results are quite large. The consequence is that the obtained results should be applied with care.

Table 5.6 Mean values of early age concrete properties vs. degree of hydration.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Degree of Hydration (+)</th>
<th>$f_t$ (MPa)</th>
<th>$E_c$ (MPa)</th>
<th>$G_{f160}$ (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.324</td>
<td>2.04</td>
<td>1.63</td>
<td>23500 21167 0.070 0.045</td>
</tr>
<tr>
<td>2</td>
<td>0.548</td>
<td>2.95</td>
<td>2.66</td>
<td>31200 30833 0.085 0.057</td>
</tr>
<tr>
<td>3</td>
<td>0.638</td>
<td>3.18</td>
<td>3.04</td>
<td>32750 34500 0.084 0.061</td>
</tr>
<tr>
<td>5</td>
<td>0.727</td>
<td>3.73</td>
<td>3.06</td>
<td>37300 33667 0.097 0.059</td>
</tr>
<tr>
<td>7</td>
<td>0.773</td>
<td>3.80</td>
<td>3.05</td>
<td>37800 38667 0.094 0.079</td>
</tr>
<tr>
<td>14</td>
<td>0.834</td>
<td>4.15</td>
<td>3.69</td>
<td>38833 38833 0.097 0.062</td>
</tr>
<tr>
<td>28</td>
<td>0.858</td>
<td>4.77</td>
<td>3.63</td>
<td>38100 39167 0.090 0.070</td>
</tr>
</tbody>
</table>

Table 5.7 Regression coefficients of degree of hydration based early age concrete fracture properties.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Specimen type</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3</td>
<td>16</td>
</tr>
<tr>
<td>$f_t(\alpha_{h_{\text{max}}})$ (MPa)</td>
<td>4.77</td>
</tr>
<tr>
<td>$a$ (-)</td>
<td>0.93</td>
</tr>
<tr>
<td>$E_c(\alpha_{h_{\text{max}}})$ (MPa)</td>
<td>39000</td>
</tr>
<tr>
<td>$b$ (-)</td>
<td>0.40</td>
</tr>
<tr>
<td>$G_f(\alpha_{h_{\text{max}}})$ (N/mm)</td>
<td>0.100</td>
</tr>
<tr>
<td>$c$ (-)</td>
<td>0.31</td>
</tr>
</tbody>
</table>

The left side of the Equation (5.12), (5.13), and (5.14) represents the normalized early age concrete mechanical properties as a function of the degree of hydration. In fact, $\left(\frac{a_{h(t)}-a_0}{a_{h_{\text{max}}}-a_0}\right)$ at the right side of the above equations ranges from 0 to 1, thus, a smaller regression coefficient $a$, $b$, and $c$ of those equations indicates a faster development of the relevant concrete properties at early age. As shown in the
Table 5.7, in the hardening phase the elastic modulus of both mixtures is developing in a faster way than the tensile strength. The similar tendency is reported by Weigler and Karl (1974), and has been become a well-known phenomenon (De Schutter 1996).

![Graphs showing the development of early age concrete properties](image)

Figure 5.22 Development of early age concrete properties described by degree of hydration for specimen type 6.3 and type 16 based on the values of all specimens (a) Uniaxial tensile strength; (b) elastic modulus; (c) fracture energy
However, very few previous researches focused on the evolution of fracture energy or fracture toughness during the hardening phase of concrete. According to the regression analysis of the calculated fracture energy \( G_{f160} \) in the present study, the value of coefficient \( c \) is smallest compared to that for tensile strength and elastic modulus, which indicates that the fracture energy develops much faster than tensile strength and the elastic modulus of concrete. However, it should be mentioned that the obtained fracture energy development results should be applied with care because of the arbitrary determination of the critical crack openings and the lack of test results of the specimens with the age less than 24 hours. Besides, some contrary conclusions were reported in the limited available literature about the evolution of concrete fracture energy. De Schutter (1996) found a linear increment of fracture energy as a function of the degree of hydration for concrete made of blast furnace slag cement according to three-point bending tests of un-notched beams. Zollinger et al. (1993) gave a relation about the development of fracture toughness of concrete as a function of the age of the specimen, \( K_{328} / K_{328}^0 = (t/28)^{0.25} \), based on the size effect law through the three point bending tests of various size notched beams with similar geometry. Gaedicke et al. (2007) observed that the evolution of fracture toughness, through wedge splitting tests of two mixtures for concrete pavements, was faster than that of the elastic modulus from 6 hours to 24 hours. Unfortunately, no test results of later ages were reported in his study.

### 5.3.4 Size Effect of Fracture Energy

It should be noted that the aim of this experimental program is to study the evolution of the fracture energy during hardening, not to verify the size effect of fracture energy. However, when intending to use the obtained fracture energy from this relative small size specimens to evaluate the crack development of concrete pavement slabs, the consequence of the size-dependence of \( G_f \) should be considered.

The most direct way of determining the size-independent fracture energy \( G_F \) is by means of a stable uniaxial tensile test (Hillerborg et al. 1976), as understood at that time. Unfortunately, in the past it was difficult to perform stable and representative tensile tests (Hordijk 1991). Consequently, several simpler procedures were proposed based on determination of the total work done of fracture of three-point bend notched beams (RILEM 1985), wedge splitting tests, and compact tension tests (Wittmann et al. 1990). In the past few decades many experiments for the determination of the fracture energy of concrete specimens have been carried out by the above mentioned methods and most of those results show that the fracture energy is more or less size dependent (Trunk and Wittmann 2001; Duan et al. 2003; Hu and Duan 2004; Xu et al. 2006). The fracture energy
increases with increasing specimen size with a tendency towards a constant value for large specimen size. To avoid confusion, in the following analysis $G_f$ is used exclusively for the experimentally determined size-dependent fracture energy, and $G_F$ for the size-independent fracture energy, which is the asymptotic value of $G_f$ when the specimen size is very large. In compact tension tests, the size-independent fracture energy $G_F$ was found when increasing the ligament length beyond 300 mm that is about 20 times greater than the maximum aggregate size. Moreover, an increase of $G_f$ by about 25% has been observed when the ligament length of the compact tension specimen increased from 100 mm to 300 mm (Wittmann et al. 1990). With regard to the fracture energy $G_F$ determined by three point bending tests of notched beams, Brameshuber and Hilsdorf (1990) have observed an increase of the fracture energy $G_f$ of about 20% when increasing the beam depth from 100 mm to 800 mm.

Figure 5.23 The concept of local fracture energy $G_f$, the boundary effect (the ligament transition length $a_l^*$, and the distribution of fracture energy $G_f$ (after Wittmann, 1992 and 2003).

Several interpretations of the size effect of the fracture energy determined by the RILEM fracture energy method have been proposed by Elices et al. (1992), Planas et al. (1992) and Guinea et al. (1992), which include: the experimental procedures, the bulk energy dissipation, and the cutting of the force-displacement curve tail. However, those interpretations are phenomenological and it has proven not to be sufficient to explain the size effect (Guo and Gilbert 2000). Even more important, they do not attempt to explain the mechanics of the problem of size dependency of the concrete fracture energy. Hu and Wittmann (1992) have
advanced to link this size dependency to the development of the fracture process zone, which is based on the mechanics of the fracture. They introduced the term ‘local fracture energy’ and the effect of back boundary conditions. It was proposed that the fracture energy dissipation was not constant over the whole cracked area, and a fracture energy distribution exists. It assumes that the energy is higher and remains constant in areas where a Fracture Process Zone (FPZ) can fully develop when the ligament length is large enough. On the other hand, the development of FPZ is restricted by the limited ligament length leading to a reduced FPZ size, both for the length and the height, when a crack is close to the back surface of a specimen. If the region where a full FPZ can develop is dominant in a specimen, then the average fracture energy \( G_f \) is close to the size-independent \( G_F \) and hence there is no size effect. However, if the specimen size where only a limited FPZ can develop is dominant, the average fracture energy \( G_f \) is specimen size dependent and is less than \( G_F \) (Hu and Wittmann 2000). This proposed method has been successfully used to explain the size dependency of fracture energy and been verified by numerous experimental results (Trunk and Wittmann 2001; Duan et al. 2003; Hu and Duan 2004; Xu et al. 2006). The concept of a boundary zone where the fracture energy varies allows the determination of the size-independent fracture energy and, probably more important, the prediction of the fracture behavior of differently sized structures on the basis of the test results obtained on standard laboratory specimens (Duan et al. 2007).

Figure 5.24 Fracture energy measurements as a function of the ratio of minimum cross section dimension to maximum aggregate size \( D/d_{\text{max}} \), the solid regression lines are calculated according to the equation \( \frac{G_f}{G_F} = \frac{\beta_1 \cdot D}{1 + \beta_1 \cdot D} \) proposed by Hu and Wittmann (2000), where \( G_F \) and \( \beta_1 \) are determined by curve fitting of their experimental data.
Regarding the uniaxial tensile test, a similar size-dependency of the concrete fracture energy was also observed for unnotched dog bone-shape specimens of various size and with similar specimen geometry (Carpinteri and Ferro 1994; van Vliet 2000). The fracture energy was found to increase with the specimen size towards a horizontal asymptote for large sizes, as shown in Figure 5.24. An increase of about 30% fracture energy calculated from the average measured deformation along the specimen was found when the specimen aspect ratio \(D/d_{max}\) increases from 3.75 to 100, whereas no significant difference of the calculated fracture energy was observed when \(D/d_{max}\) increases from 7.5 to 100.

Therefore, it appeared from experimental results as well as from numerical simulations that the adopted smallest aspect ratio \(D/d_{max}=3.75\) was too small. In addition, the difference between the latitude of the size-independent fracture energy \(G_F\) between Carpinteri and van Vliet may be partly due to the larger maximum aggregate size of 16 mm used in the first author’s concrete mixture compared to a maximum grain size of 8 mm of the second author’s mixture. Moreover, the compressive strength is higher in Carpinteri’s project as well. As mentioned previously, a clear tendency of \(G_F\) increases with increasing compressive strength and maximum aggregate size was observed (CEB-FIP, 1990).

In the present study, the aspect ratio \(D/d_{max}\) is 8.0 and 3.125 for specimen type 6.3 and 16, respectively. Thus, the calculated fracture energy \(G_F\) of type 16 mixture may be much lower than the corresponding material property. Hillerborg (1985) concluded that the acceptance error and standard deviation of a fracture energy \(G_F\) test is about 3 times larger as in most strength tests. Therefore, the fracture energy determined from the relatively small specimens adopted in this study is still useful when analysing pavement structures subjected to tensile stresses.

5.3.5 Fracture Toughness

In linear elastic fracture mechanics (LEFM), the critical stress intensity factor \(K_{IC}\) for an ellipsoidal crack in an infinitely large specimen is related to \(E_cG_{IC}\) according to (Peterson 1980; Zollinger et al. 1993; De Schutter and Taerwe 1997):

\[
K_{IC}^2 = E_cG_{IC} \tag{5.16}
\]

Where, \(G_{IC}\) is the critical energy release rate that is defined as the energy required per unit crack extension in a brittle material in which there is no process zone. In other words, all the energy needed to create or increase a crack is the surface energy, and no energy is dissipated at the crack tip (Bažant 1992). However, numerous investigations have indicated that for a quasi-brittle material such as concrete there is a sizeable zone ahead of the crack tip where micro-cracking and other inelastic phenomena occur that cause crack initiation and slow crack growth behaviour before reaching the instable condition where the crack tip
propagates (Kumar and Barai 2011). Therefore, the relation such as \( K_{IC} = \sqrt{E_c G_{IC}} \) cannot be applied for concrete material. However, the fracture process zone that is the nonlinear characteristic of concrete fracture may be changed into a linear elastic part through an equivalent elastic approach. Then, LEFM could again be used to evaluate the concrete fracture process by modified elastic concrete fracture models, such as the two parameters fracture model (TPFM), the double K fracture model (DKFM), and the double G fracture model (DGFM). In other words, \( K_{IC} = \sqrt{E_c G_{IC}} \) is applicable (Xu and Zhang 2008) when those models are used.

**Further Remarks**

Numerous researchers have attempted to quantify the fracture toughness \( K_{IC} \), or the critical energy release rate \( G_{IC} \) in terms of the fracture energy \( G_F \) (Peterson 1980; Hilsdorf and Brameshuber 1985; Nallathambi and Karihaloo 1986; Xu and Zhang 2008). According to the Fictitious Crack Model (FCM) proposed by Hillerborg, the fracture energy \( G_F \) is defined as a material constant to describe the resistance of a concrete fracture subjected to tensile stress. It is commonly determined by the total applied energy with the projected fracture area, which is the area under the load-deflection curve per unit fracture surface area, the so-called RILEM fracture energy \( G_f \). In fact, the RILEM fracture energy \( G_f \) is a specific fracture energy that involves the entire crack propagation process from the crack initiation to propagation through the entire ligament. As discussed previously, the RILEM fracture energy \( G_f \) is dependent on the specimen size, in the sense that it increases with increasing specimen size, and has a tendency towards a constant value for large specimen sizes. Based on the concept of local fracture energy and the boundary effect proposed by Hu and Wittmann (1992), the RILEM fracture energy \( G_f \) obtained from the normal laboratory specimen size is obviously underestimating the size-independent fracture energy \( G_F \).

### 5.4 FINDINGS AND CONCLUSIONS

The tension set-up developed in the ACRe project worked very well to obtain the fracture energy, uniaxial tensile strength and the elastic modulus of the concrete at early age. Due to the adopted small specimen size, very fast servo system, and stiff testing frame, a stable experiment and a complete softening curve was obtained, even for concrete specimens with an age as early as 24 hours. Based on an extensive deformation controlled uniaxial tensile test programs on two typical pavements concrete mixtures used in Belgium, the following conclusions can be drawn:

- The applied parabolic shape of the concrete specimens ensured the crack occurring near the center of the specimen where it was in the intended state of more or less uniform stress. Besides, this parabolic shape specimen
with a considerable reduction of cross section area (nearly 60%) at mid-height let the cracks initiate far enough from the end caps to prevent influence of the confinement there.

- The loading rate of 0.1 \( \mu \text{m/s} \) corresponding to a measuring length of 90 mm has been proved successful in preventing the crack from growing explosively for the concrete with an age higher than 48 hours, as no snap-back was observed in the post-peak region of all the measured force-deformation curves. A lower loading rate of 0.05 \( \mu \text{m/s} \) was subsequently found to provide a stable complete uniaxial tension test for the concrete with an age of 24 hours.

- Higher fracture energy, uniaxial tensile strength, and elasticity modulus are observed for the specimen type 6 (smaller aggregates), which does not coincide with the tendency from the literature. Besides, a larger scatter of the measured results is observed for specimen type 16, which has an aspect ratio of 3.125. It may indicate that this aspect ratio is too small, which might be not give a sufficiently representative volume element to determine the fracture energy through this parabolic shape specimen. Due to the shape of the split mould, decreased compaction at the center of the specimen as found in the ACRe project may result in weak locations in the center of the specimens, especially for the specimen type 16, and it would obscure the actual tensile strength and fracture energy of concrete.

- The fracture energy, uniaxial tensile strength, and elasticity modulus were all found to increase with age going towards a horizontal asymptote as concrete hardened in a tested age range of 1 day to 90 days. The development rate of the fracture energy was found to be higher as compared to the tensile strength and the stiffness. Based on the measurement results on hardening concrete specimens, a degree of hydration-based description for the uniaxial tensile strength, elasticity modulus, and the fracture energy has been given.

However, very few test data of the fracture energy for the concrete age less than 24 hours are available in the literature, which is required in the case of crack induction at a very early age of concrete. For instance, in the early entry method a shallow saw cut is made just after the final set of concrete to let the crack initiate at predesignated locations. The following recommendations are given to obtain the concrete fracture energy at a very early age:

- The hydration process of hardening concrete is significantly influenced by the curing conditions (temperature and moisture). It is generally accepted that the measured fracture energy of concrete is influenced by the temperature and moisture gradients inside the measured concrete specimen. However, the current uniaxial tension test for early age concrete is con-
ducted in the room condition where the temperature and the moisture are uncontrolled. In order to accurately capture the concrete properties at the very early age of concrete, it is recommended that these tests are conducted in a climate chamber are taken during the specimen preparation and installation phase.

- Larger specimen sizes should be used for the typical paved concrete mixtures, which generally have a maximum aggregate size of 32 mm. The aspect ratio should be higher than 5. However, the measuring length of the control signal would increase as well and it has to be checked whether it is too long or not to achieve a stable softening curves.

- There are great difficulties to use the proposed uniaxial tension set-up and this parabolic shape specimen for early age concrete. It takes many efforts and too long time to prepare and install the specimen in the current test procedures for the proposed uniaxial tension test. It takes about 3 hours to demould, saw cut, glue, and install the specimen in the current test procedure. Besides, gluing the specimen with the loading platen may be impossible on the very early concrete specimen. Considering the above-mentioned difficulties and drawbacks, the proposed tension test set-up and test procedures in the present study might be not suited for the very early age concrete (less than 24 hours). The indirect method, such as wedge splitting test and horizontal tensile test, might be an alternative option to obtain the softening curves for very early age concrete.
Active Crack Control for CRCP

Many field investigations have shown that the majority of the transverse cracks in CRCP are caused by restrained volume changes due to the temperature and moisture variations in the early age, prior to the application of traffic loading to the pavement (Suh et al. 1992). In contrast to anxiety of cracking in most common concrete structures, these transverse cracks in CRCP are considered as not harmful to CRCP’s long term performance as long as the cracks are kept quite tight (ARA Inc. 2003a; Yeon et al. 2012). However, extensive field observations have shown a tendency that the majority of punchout do occur in CRCP that have a randomly distributed crack pattern, especially in clusters of closely spaced cracks (Selezneva et al. 2003). Thus, the attempts to initiate the cracks in CRCP in a desirable distribution are attractive measures to decrease the risk of severe distresses, such as punchout and spalling, in the long-term of CRCP.

6.1 ENVIRONMENTAL INDUCED STRESSES

The environmental induced stresses of the unnotched pavement slab need to be calculated first before the analysis of the active crack control method of CRCP. Environmental induced stresses consist of two parts, temperature and moisture induced stresses. In the present study, the effect of the thermal induced strain is thought to dominate the total stress that is caused by environmental loads in the very early age of the concrete pavement. In the present study, the moisture related concrete strain is not taken into account for the early age stress calculation because:

1. During the first few days, the drying shrinkage is considered to be minimal in case of an adequately applied curing practice. The curing treatment using a plastic sheet at first and subsequently a curing compound which is used for the Belgium CRCP projects is considered effective in preventing a large
amount of moisture evaporation from the slab, especially in the most important part of the slab that is just below the pavement surface. For instance, one in-situ relative humidity measurement has shown that the moisture loss was less than 3% at 5 mm depth and was negligible at 12.5 mm depth when the curing compound was applied according to TxDOT specifications (Yoen 2011).

(2) In addition, the autogenous shrinkage is neglected as well because of the applied moderate water to cement ratio of 0.43 that is commonly used for pavement concrete in Belgium.

### 6.1.1 Temperature Profile

As shown in Figure 6.1, the varying temperature profiles $T(z, t)$ along the depth of the pavement slab during the early age show high nonlinearity. A larger temperature fluctuation at the pavement surface is observed as compared to that for the bottom of the slab. The actual nonlinear temperature profile can be separated into three components, as shown in Figure 6.1.

\[
T(z, t) = T_{axial}(t) + T_{linear}(z, t) + T_{ses}(z, t)
\]  

Figure 6.1 Three components of the concrete pavement temperature distribution.

The component of the axial temperature $T_{axial}(t)$ is a uniform temperature that causes the pavement slab to expand or contract evenly through its depth. In concrete pavement thickness design, and the fatigue analysis of the concrete slab, the corresponding strain induced by this axial temperature change is generally ignored, because the restraint to this deformation is assumed minimal. However, this assumption is not valid at the early age of the concrete pavement, especially when calculating the environmental induced stress without any cracks in both JPCP and CRCP. In fact, it is rational to assume that this axial strain is completely restrained in the centre parts of an infinitely long uncracked pavement slab, such as the 24 hours continuously constructed projects in the present study, prior to the
occurrence of any transverse crack. Field investigations have indicated that all the early age cracks occurred in the night with a negative temperature profile, and none of them occurred with a positive temperature gradient in the pavement slab. The second component of the equivalent linear temperature difference $T_{\text{linear}}(z,t)$ is quantified as causing the same bending moment as the total nonlinear temperature profile $T(z,t)$ minus the axial temperature component $T_{\text{axial}}(t)$.

The last component is the nonlinear part that remains after the uniform and the linear temperature parts have been subtracted from the total temperature distribution. The last nonlinear component does not affect the deflection profile of the slab (Ioannides and Khazanovich 1998). Considering the continuity requirements of the body, each element will exert a restraining action on the movement of the surrounding elements that is called internal self-equilibrating strain.

Many investigators (Choubane and Tia 1992; Mohamed and Hansen 1997) have used either a quadratic or third order polynomial to represent the actual nonlinear temperature profile in a concrete pavement. Measured temperature data at three and four points along the depth are required to determine the relevant coefficients of the assumed quadratic or third order polynomial curves, respectively. In the present study, the three temperature components are more accurately quantified with the estimated temperature profiles along the concrete pavement depth, and are presented as follows:

$$T_{\text{axial}}(t) = \frac{1}{h} \int_0^h T(z,t)dz$$

$$T_{\text{linear}}(z,t) = \int_0^h [(T(z,t) - T_{\text{axial}}(z,t))]zd\zeta$$

$$T_{\text{SES}}(z,t) = T(z,t) - T_{\text{axial}}(t) - T_{\text{linear}}(z,t)$$

Where, $z$ is the depth along the concrete slab; $h$ is the thickness of the concrete slab; $I$ is the moment of inertia of the cross section and $I = h^3/12$ for a unit width cross section.

### 6.1.2 Zero-stress Temperature

In order to estimate the thermal stress development, it is essential to define a reference temperature from where the stress starts to develop. The definition of the zero-stress temperature in a concrete pavement is briefly described below. As shown Figure 6.2, shortly after concrete placement the concrete pavement slab typically experiences a volume expansion as well as a temperature increase due to the hydration of cement, but no stress is developing within this period because the concrete is still in a plastic state. At the concrete final set $t_{fs}$, both the strength and stiffness start to develop, that is the so-called ‘first zero-stress temperature’ $T_{fs}$.
(Eisenmann and Leykauf 1990). After the final set, a momentary compressive stress will develop as the expansion of the concrete is restrained until the peak temperature $T_{\text{max}}$. Subsequently, this compressive stress begins to relieve as the concrete temperature decreases due to the heat loss to the environment. At some point, the developed compressive stress is completely relieved and the stress condition changes from compression to tension. The temperature at this point is referred to as ‘second’ zero-stress temperature $T_{\text{zs}}$ (Suh et al. 1992; ARA Inc. 2003a). If it is assumed that there is no drying shrinkage or creep during this period, the zero-stress temperature will be the same as the final setting temperature. But, the rapid development of the elastic modulus of concrete during this period together with the above mentioned effects of shrinkage and creep, the second zero-stress point will occur at a higher temperature, as indicated in Figure 6.2.

![Figure 6.2 A conceptual determination of zero stress temperature (after Suh et al. 1992).](image)

Quite recently, the zero-stress temperature is receiving significant attention by researchers and practitioners being one of the various factors influencing the behaviour of CRCP (Yoen 2001). Firstly, $T_{\text{zs}}$ is considered as an important input
parameter in MEPDG, which affects the post-cracking behaviour of CRCP such as crack width and load transfer efficiency. A higher $T_{zs}$ tends to significantly increase the number of punchouts (Won 2009). On the other hand, $T_{zs}$ is also considered as substantially affecting the early age cracking risk of concrete structures since $T_{zs}$ is the reference point where the tensile stress starts to develop (Yeon et al. 2013). It should be noted that $T_{zs}$ actually is not a single temperature but varies through the depth of the slab, termed as the zero-stress gradient (ARA Inc. 2004).

While recognizing the importance of $T_{zs}$ in CRCP design, a reliable determination of $T_{zs}$ is not well documented. The currently available analytical $T_{zs}$ prediction model for concrete pavement, proposed in MEPDG, is a quite simple model with only two input parameters, cementitious content, and mean monthly temperature of the month of construction. Thus, the $T_{zs}$ prediction model in MEPDG cannot predict the influences of the paving time on a day, the paving temperature, and the other climate and construction parameters. For instance, field measurements have revealed a higher $T_{zs}$ for concrete placed in the morning compared to the section placed in the afternoon (Schindler et al. 2002; Hansen et al. 2006; Won 2009). On the other hand, the prediction model in MEPDG also tends to overestimate $T_{zs}$ due to the assumption of zero heat loss through the thermal boundaries made in it. There are also some attempts to correlate $T_{zs}$ and the peak temperature $T_{max}$ using a reduction factor based on experimental and numerical analyses. Reduction factors ranging from 6% to 8% have been reported (Schindler et al. 2002). Yeon et al. (2013) found reduction factors between 0.1% and 11.1% according to both stress dependent and stress independent measurements on four projects. They also concluded that it might be acceptable to use $T_{max}$ instead of $T_{zs}$ in the calculation of thermal stress from a practical standpoint, if there is no information on $T_{zs}$ for a given project.

As mentioned above, there are too many uncertainties in the above-mentioned methods to give a reliable value of the second zero-stress temperature $T_{zs}$ for a concrete pavement, and it is too difficult to obtain this temperature through field measurements (Suh et al. 1992). Thus, the ‘first’ zero-stress temperature $T_{fs}$ at the time of final set, which is easier to quantify through the assumed critical degree of hydration and with less uncertainties influencing it, is chosen in the present study as the reference temperature to evaluate the thermal stress development in the early age of a concrete pavement.

### 6.1.3 Built-in Curling

The concrete temperature can be considered as uniformly distributed along the pavement depth at the time of placement. Thereafter, a nonlinear temperature distribution of the pavement slab will develop under field conditions, as discussed in Chapter 4. The pavement at final set is regarded as a zero stress condition and the pavement is in a flat condition because the concrete is just leaving a plastic state.
It should be pointed out that the flat condition at the final set corresponds to a temperature gradient in the slab. When the actual temperature gradient through the slab becomes zero later, the slab will be in a concave condition analogous to a slab with a negative temperature gradient, so called ‘built-in curling’ in the slab, as shown in Figure 6.3. Rao and Roesler (2005) theoretically defined the effective built-in temperature gradient $\Delta T_{ebi}$, which includes the built-in temperature gradient, drying shrinkage gradient, moisture gradient, and the effect of the creep component, indicated as follows:

$$\Delta T_{ebi} = \Delta T_{bi} + \Delta T_{shr} + \Delta T_{mg} - \Delta T_{creep}$$  \hspace{0.5cm} (6.5)

Where,

- $\Delta T_{ebi}$ = the effective linear temperature difference between the top and bottom of the concrete slab of the equivalent built-in temperature gradient, [°C];
- $\Delta T_{bi}$ = temperature difference between top and bottom of a slab equivalent to nonlinear built-in construction temperature gradient, [°C];
- $\Delta T_{shr}$ = temperature difference between top and bottom of a slab equivalent to irreversible differential drying shrinkage between the top and bottom of the slab, [°C];
- $\Delta T_{mg}$ = temperature difference between top and bottom of a slab equivalent to reversible differential moisture gradient between the top and bottom of the slab, [°C];
- $\Delta T_{creep}$ = the portion of $\Delta T_{bi}$ and $\Delta T_{shr}$ recovered by creep, [°C].

In the present study, the built-in temperature gradient $\Delta T_{bi}$ is used as exclusive term denoting the effective built-in temperature gradient $\Delta T_{ebi}$ to calculate the restrained stress in the early age of CRCP to optimize the saw cutting time. Both the differential drying shrinkage component $\Delta T_{shr}$ and moisture gradient $\Delta T_{mg}$
are not taken into account because of the little moisture gradient expected in the first few days for the Belgium CRCP construction practice.

Eisenmann and Leykauf (1990) found that the built-in temperature gradient $\Delta T_{ebi}$ is of the same importance in its effects on upward curling as the daily temperature gradient. Similar tendencies were also found in several field and analytical studies (Yu et al. 1998; Rao and Roesler 2005; Hansen et al. 2006; Vandenbossche et al. 2010; Yeon et al. 2012). The magnitude of the built-in temperature gradient is mainly influenced by the weather conditions during setting. Generally, a higher built-in temperature gradient is developed in warmer summer construction compared to spring and fall construction. Moreover, not only the construction season of the year, but also the concrete placement time on the day significantly influences the magnitude of the built-in temperature gradient. A largest negative built-in curling is normally observed for concrete placed in the morning of sunny summer days, the slab then reaches it maximum curl because the intensive heat of hydration and the solar radiation coincide at about the same time resulting in a large positive temperature gradient when the slab hardens (Rhodes 1950). In case of fall construction or night paving on the day (a 24 hours/day construction is common practice in Belgium CRCP projects), a negative temperature gradient at the final set may develop, which indicates a positive built-in temperature difference for the pavement slab. For instance, field measured built-in temperature gradient are -11.0°C and +1.5°C for the summer and fall construction of two JPCP projects in Michigan, respectively (Hansen et al. 2006). Several approaches to determine the built-in temperature gradient are reported in the literature (Yu et al. 1998; Beckemeyer et al. 2002; Rao and Roesler 2005; Yeon et al. 2012). Lastly, Springenschmid and Hiller (1998) pointed out that the use of plastic sheeting or insulation blankets can lead to the development of a higher built-in temperature gradient.

Figure 3.7 illustrates the measured temperatures at various depth of the slab in the first few days after concrete placement on E17 and E313. Because both temperature measurement points were installed in sections constructed around midnight and the pavement slabs were covered with a plastic sheet, rather uniform temperature profiles were observed at the time of the final set of the concrete about 9 hours after concrete placement, as shown in Figure 6.4.
Figure 6.4 Measured temperature gradient as a function of time for both projects.
6.2 THERMAL STRESSES CALCULATION

6.2.1 Degree of Restraint and Thermal Strain

CRCP is considered as completely restrained in the longitudinal direction before transverse cracking (Schindler et al. 2002; Nam 2005). However, field measurements have indicated that the longitudinal volume changes in CRCP prior to transverse cracking were not minute, as a longitudinal tensile strain at the mid-depth of the slab up to $50 \times 10^{-6}$ mm/mm was measured (Yeon et al. 2013). This does not agree with the above assumption that CRCP is completely restrained in the longitudinal direction before transverse cracking. The magnitude of the external and internal restraint of an infinite long slab casted on a continuous base can be found in ACI Committee 207.1R (2005); ACI Committee 224.1R (2007). Nevertheless, this assumption of complete restraint in the longitudinal direction is still adopted in the present study, considering it as a compensation of the unaccounted volume changes due to drying shrinkage and moisture gradient that are not included in the present study. Therefore, the maximum axial stress component, occurring at the centre of the slab in the transverse direction, is calculated by applying Hooke’s law and considering the stress state in two dimensions:

$$\Delta \sigma_{axial} = \frac{E \alpha_T \Delta T_{axial}}{1 - \nu} \tag{6.6}$$

CRCP before transverse cracking can be considered as a semi-infinite slab. Therefore, the maximum curling stresses, $\sigma_{curling}$ induced by the equivalent linear temperature difference, can be calculated using the Westergaard equation (Westergaard 1926):

$$\Delta \sigma_{curling} = \frac{C E \alpha_T \Delta T_{linear}}{2(1 - \nu)} \tag{6.7}$$

Where,

- $E$ = the modulus of elasticity of the concrete, [MPa];
- $\alpha_T$ = the coefficient of thermal expansion of the concrete, [1/°C];
- $\Delta T_{linear}$ = the temperature difference through the slab, [°C];
- $\nu$ = the Poisson’s ratio of the concrete, [-].

The slab geometry factor $C$ proposed by Bradbury is defined as (Bradbury 1938):

$$C = 1 - \frac{2 \cos \lambda \cosh \lambda (\tan \lambda + \tanh \lambda)}{\sin 2\lambda + \sinh 2\lambda} \tag{6.8}$$

$$\lambda = \frac{L}{l\sqrt{8}} \tag{6.9}$$
\[ l = \frac{\sqrt{\frac{Eh_{PCC}^3}{12(1 - \nu^2)k}}}{6.10} \]

Where,

- \( l \) = the radius of relative stiffness of the pavement slab, [m];
- \( L \) = the length of the pavement slab, [m];
- \( h_{PCC} \) = the thickness of the pavement slab, [m];
- \( k \) = the modulus of substructure reaction, [MPa/m]; the substructure includes all the layers, including the subgrade, below the pavement slab.

As shown previously, the self-equilibrium stresses due to the nonlinear temperature profile are produced internally to satisfy the continuity requirements. Those stresses exist regardless of the external restraint of the pavement slab and its boundary conditions (Mohamed and Hansen 1997). Similar to the axial stress component, the thermal stress induced by the nonlinear temperature component can be calculated as:

\[ \Delta \sigma_{nonlinear} = \frac{E\alpha_T \Delta T_{SES}}{1 - \nu} \tag{6.11} \]

No laboratory tests were conducted to measure the Poisson’s ratio of the concrete mixture used in E313 and E17. The Poisson’s ratio was taken as 0.15 in the present study, unless reported differently.

### 6.2.2 Coefficient of Thermal Expansion of Young Concrete

The coefficient of thermal expansion of concrete (CTE) is a key factor that determines the thermal-induced stress in a concrete pavement. The value of CTE of concrete is reported as depending on a number of factors: the age of the concrete, the type of aggregate, water to cement ratio, the content and the type of cement, and the moisture condition (Emanuel and Hulsey 1977). Various methods for measuring CTE of cementitious material are well documented (Boulay 2003; Won 2005). With respect to the time dependence of CTE of concrete, many experimental results have shown that the CTE of concrete decreases sharply during the first few hours that generally coincides with the final setting, and then remains constant (Kada et al. 2002; Loser et al. 2010). Figure 6.5 shows the measured CTE of concrete during very early age. The observed higher value of CTE before the setting is mainly contributed to the presence of water that is not yet linked in the structure of the concrete, as water at 20°C has coefficient of thermal expansion up to 20 times greater than that of the other concrete constituents (Kada et al. 2002). According to the findings by Kada and Loser, it is thus applicable to use a constant value of CTE of the concrete to evaluate the thermal stress development in the
early age concrete pavement that is assumed to start to develop after the final set. Typical values of CTE for the commonly used paved concrete materials depending on the type of aggregate are well documented by Schindler et al. (2002). Limestone coarse aggregate is used in E313 and E17 as indicated in Table 3.1, and a value of $1.06 \times 10^{-5}$ mm/mm/°C is thus chosen for CTE in the present study.

![Figure 6.5 Evolution of the CTE for concrete having a water to cement ratio of 0.45; the used type of coarse aggregate is limestone (after Kada et al. 2002).](image)

### 6.2.3 Relaxation Based on Degree of Hydration

Restrained thermal deformation leads to stress development in a concrete pavement. An accurate estimation of the early age stress development is only achieved by means of a good knowledge of the early age creep and relaxation behaviour due to relatively high creep for early age concrete. Few models (Emborg 1989; Westman 1999) are available to account for stress relaxation effects for early age concrete based on the work on hardened concrete by Bažant and Chern (1985). However, numerous experiments have shown that creep is profoundly affected by not only age but also the temperature during the curing period (De Schutter 1999). In other words, the creep evolution of early age concrete must be more adequately described with a fundamental parameter, like the degree of hydration. van Breugel (1980) presented a relaxation model for hardening concrete based on the degree of hydration. This formula has been verified by numerous tests and is widely used for practical calculations in the Netherlands. Thus, Van Breugel’s relaxation model for young concrete based on the degree of hydration is adopted in the present study and is presented as follows:

$$
\psi(\tau, t) = e^{-\left[\left(\frac{\alpha_h(t)}{\alpha_h(\tau)} - 1\right) + 1.34 \cdot (wcr)^{1.65} \cdot t - \frac{\tau}{d} \cdot (t - \tau)^n \cdot \frac{\alpha_h(t)}{\alpha_h(\tau)}\right]} \quad (6.12)
$$
Where,

\[ \tau = \text{time of existence of the stress increment, [hour]}; \]
\[ t = \text{the age of concrete, [hour]}; \]
\[ wc_r = \text{water to cement ratio, [-]}; \]
\[ \alpha_h(\tau) = \text{degree of hydration at the loading time } \tau, [-]; \]
\[ \alpha_h(t) = \text{degree of hydration at time } t, [-]; \]
\[ d = \text{constant depending on the hydration rate of cement,} \]
\[ \text{moderate hardening cement: } d = 0.3, \]
\[ \text{rapid hardening cement: } d = 0.4, \]
\[ n = \text{constant factor, here is chosen 0.3}. \]

Figure 6.6 shows the calculated relaxation factor \( \psi(\tau, t) \) for the concrete mixture used in E17 during the first week. It is clearly shown that the stress increments that develop during the hardening phase of concrete rapidly reduce and almost completely vanish, and the earlier the initiation of the stress increments, the faster the stress reduction.

![Figure 6.6 Relaxation factors \( \psi(\tau, t) \) for hardening concrete of E17, calculated with Equation (6.12).](image)

### 6.2.4 Stress Calculation-Superposition Method

Several numerical approaches, such as effective modulus method, age adjusted effective modulus method, increment method, and step by step method, have been used to calculate the stress history of hardening concrete (Schindler et al. 2002). The step-by-step method is adopted in the present study to calculate the stress development in a concrete pavement due to its advantages of considering the varying strain history along with the time-dependent material properties. Based on the superposition principle, the step-by-step method assumes that the stress history consists of a series of discontinuous stress increments, and then accumulates all individual stress increments with the corresponding relaxation factors to
obtain the residual stresses. The time is subdivided into discrete, sufficiently short time intervals $\Delta t$ (60 seconds is used in the present study). The calculation procedure of this step-by-step method is presented as follows:

Figure 6.7 Schematic representation of superposition principle, after Van Breugel (1998).

Step 1: calculate the temperature profile $T(z, t)$ along the pavement depth $z$ at time $t$, and the corresponding degree of hydration $\alpha_h(z, t)$ of concrete under the field curing conditions based on the temperature model proposed in Chapter 4.

Step 2: determine the final set time $t_{fs}$ when the critical degree of hydration $\alpha_f$ is reached. The critical degree of hydration $\alpha_f$ at the final set is calculated according to Equation (5.15). It is assumed that the stiffness of the concrete starts to develop at the time of $t_{fs}$.

Step 3: the stress increment $\Delta \sigma(\tau_j)$ at the $j^{th}$ interval ($j=1, 2, \ldots, i$) due to the corresponding strain increment $\Delta \varepsilon(\tau_j)$ is calculated as follows:
\[
\Delta \sigma(\tau_j) = R_F \cdot \frac{\Delta \varepsilon(\tau_j) \cdot E_c(\tau_j)}{1 - v} \tag{6.13}
\]

\[
\Delta \varepsilon(\tau_j) = \alpha_c \cdot \Delta T(\tau_j) \tag{6.14}
\]

\[
\Delta T(\tau_j) = T(\tau_j) - T(\tau_{j-1}) \tag{6.15}
\]

Step 4: considering the influence of the relaxation, at the age \( t_i \) (i\(^{th}\) interval, \( j \leq i \)), the stress increment \( \Delta \sigma(\tau_j) \) imposed at the \( j^{th} \) interval reduces to \( \Delta \sigma(\tau_j, t_i) \). The corresponding relaxation factor \( \psi(\tau_j, t_i) \) is calculated based on Equation (6.12).

\[
\Delta \sigma(\tau_j, t_i) = \psi(\tau_j, t_i) \cdot \Delta \sigma(\tau_j) \tag{6.16}
\]

Step 5: once the stress increment at each step is computed by Equation (6.16), the overall stress \( \sigma(t_i) \) is calculated by summing up all the stress increments illustrated in Figure 6.7, as follows:

\[
\sigma(t_i) = \sum_{j=1}^{i} \Delta \sigma(\tau_j, t_i) \tag{6.17}
\]

This above mentioned step-by-step thermal stress calculation algorithm is programmed with MATLAB, as illustrated in Appendix I.

### 6.2.5 Verification through Field Investigations

To verify the proposed thermal stress analysis method, the simulated results are verified with the field measurements on E17. Just after the concrete placement, regularly crack pattern surveys were performed for three 100 m long test sections with different longitudinal reinforcement percentages in the project E17 at Ghent, in August 2011. The detailed construction and curing conditions for each section are summarized in Table 6.1. The development of the number of transverse cracks in those three sections on E17 during the first few days after placement is illustrated in Figure 6.8.

<table>
<thead>
<tr>
<th>Section</th>
<th>Location (km)</th>
<th>Longitudinal reinforcement (%)</th>
<th>Time and day of concrete placement</th>
<th>Time of plastic sheeting removal</th>
<th>Time of the first observed crack</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subsection 1</td>
<td>45.0-45.1</td>
<td>0.75%</td>
<td>7:00~9:00, 18-08-2011</td>
<td>10:00, 19-08-2011</td>
<td>0:00~3:00, 20-08-011</td>
</tr>
<tr>
<td>Subsection 2</td>
<td>46.0-46.1</td>
<td>0.70%</td>
<td>1:00~3:00, 19-08-2011</td>
<td>15:00, 19-08-2011</td>
<td>Before 4:00, 20-08-2011</td>
</tr>
<tr>
<td>Subsection 3</td>
<td>46.4-46.5</td>
<td>0.65%</td>
<td>11:00~13:00, 19-08-2011</td>
<td>20:00, 19-08-2011</td>
<td>Before 5:15, 20-08-2011</td>
</tr>
<tr>
<td></td>
<td></td>
<td>+20 kg/m(^3) steel fiber</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Pavement temperature profiles in the early age are calculated by the model developed in Chapter 4. The relevant input parameters including concrete mixture properties and thermal properties, construction conditions, are well documented in Table 4.7. The required climate input parameters are obtained from the nearest weather station of Vlaams GEWEST nearest to Ghent (station ID: IVLAAMSG17, www.wunderground.com) that is about 10 km away from the test sections on E17.

As shown in Section 4.3, the calculated temperature profiles agree quite well with the measured values for all measuring points. The measured elastic modulus and tensile strength of the specimen type 16 (as shown in Table 5.7) are used in the subsequent thermal stress calculation, which is a function of the degree of hydration.

Figure 6.9 illustrates the calculated restraint stress development along the pavement slab. The diurnal largest tensile stresses are located in the upper portion of the slab that is subjected to the biggest temperature changes. Furthermore, the diurnal largest tensile stress occurred at the night and early morning when the slab experienced the lowest temperature. The cracking tendency prediction and the zero-stress temperature, built-in temperature gradient are evaluated with the node at 25 mm below the pavement surface rather than the surface node, because the estimated temperature for the surface node is very sensitive to sudden climate changes, such as the instantaneous solar radiation and wind speed. Figure 6.10 demonstrates the estimated temperature and thermal stress development at the top of the pavement slab (25 mm below the pavement surface) for the test sections.
on E17 and E313 for the various conditions: temperature effect only, temperature and relaxation, temperature and shrinkage, and considering all those three effects.

Figure 6.9 Calculated thermal stress development in the early age CRCP (prior to any transverse cracks) along the slab depth, with and without relaxation effect.
Figure 6.10 The calculated stress and temperature development for three test sections on E17 and E313.
Figure 6.10 The calculated stress and temperature development for three test sections on E17 and E313 (continued).
Effect of Relaxation on Early Age Stress Development

Table 6.2 summarizes the estimated zero stress temperature for E17 and E313 with and without the effect of shrinkage and relaxation. With respect to the effect of relaxation in typical PCC pavement designs, researchers generally agree that the creep effect does not substantially influence the stress history of PCC pavements, particularly when only environmental loadings are considered, because the creep effect may be internally compensated by a diurnal variation cycle (Yoen 2011). However, Figure 6.10 and Table 6.2 clearly show that it would be non-conservative to neglect the relaxation effect, since the rapid relaxation of compressive stress after the final set causes the zero-stress temperature to increase. Consequently, neglecting the relaxation effect will overestimate the cracking time and that will lead to a higher risk of developing uncontrolled randomly cracks, more especially in JPCP, before the sawcut implementation.

<table>
<thead>
<tr>
<th>Section</th>
<th>E17, 45.0-45.1</th>
<th>E313, 23.1-23.0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T</td>
<td>T+Re</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
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<td>33.7</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>22.8</td>
<td>20.1</td>
</tr>
</tbody>
</table>

Effect of Shrinkage on Early Age Stress Development

A number of previous studies have calculated the early age stress in concrete structures taking into consideration the effect of the variations of both temperature and shrinkage (Schindler et al. 2002; Houben 2008b; Zhang et al. 2013a; Zhang et al. 2013b). Some of them have concluded that shrinkage is one of the primary factors leading to cracking in concrete pavements (Zhang et al. 2013b). In case of Belgium CRCP conditions, the concrete has to be covered with plastic sheet immediately after spraying the setting retarder in order to protect it against drying out. The plastic sheet is then removed about 6 to 24 hours after concrete placement. During the period that the pavement slab is cured with plastic sheet, it is reasonable to consider only the autogenous shrinkage in the early age stress calculation, as minimal moisture loss occurs. After the removal of the plastic sheet, subsequently the curing compound is applied. Many field measurements of the internal relative humidity in concrete slabs have indicated that an adequately applied curing remarkably reduces the early age moisture loss (Ye 2007; Yoen 2011). The relative humidity drop was found to be less than 3% at 5 mm below the surface after 3 days drying in an ambient relative humidity of 40% when curing compound was applied. Ye (2007) observed that the relative humidity measured 25 mm below the pavement surface remained rather constant, around 80%, for the field CRCP sections with curing compound. Because relative humidity data is not available for the present study, a constant ambient relative humidity is chosen in the simulation.
of drying shrinkage. As found in Chapter 3, the predicted shrinkage by Eurocode 2 agreed well with the experimental data, and thus it was chosen in the present study. To verify the effect of shrinkage on the stress development of the concrete pavement at the very early age, the shrinking strain is added and estimated by the shrinkage model in Eurocode 2. The following parameters are used: mean compressive strength of 50 MPa, constant relative humidity of 85%, normal type of cement, notional size of the cross section of the pavement slab is chosen as the pavement thickness (HIPERPAV II), and the age of the concrete at the beginning of drying shrinkage is chosen as the end of plastic sheet curing.

As shown in Figure 6.10 and Table 6.2, adding of the shrinkage strain results in the zero stress state to occur earlier and thus leads to a higher zero stress temperature, and it increases the tensile stress especially in the later age. It however also illustrates that the effect of shrinkage on the concrete stress development in the first few days is rather minimal. It should be emphasized that these findings do not seek to neglect the important role of shrinkage in early age cracking in concrete pavement. The contribution of drying shrinkage in the early age stress development is minimal only when the appropriate curing treatment is applied. Besides, the effect of autogenous shrinkage is negligible as well, since the water to cement ratio of commonly used paving concrete is moderate.

Besides, this analysis has shown that the Eurocode 2 shrinkage model, which provides the average shrinkage of the concrete elements, is unable to quantify the effect of nonlinear distributed shrinkage on the early age stress development in concrete pavements. Because the moisture diffusivity is rather low for early age concrete, the top portion of the pavement slab will dry quicker than the remaining parts, thus, a highly nonlinear moisture distribution will exist in the pavement slab. The shrinkage at the pavement surface cannot be known from Eurocode 2, and the real shrinkage at the pavement surface at a given early age is believed to be larger than the corresponding average shrinkage estimated by Eurocode 2, especially in case of concrete structures having a larger volume to surface ratio for drying. The larger surface to volume ratio for drying leads to a higher half shrinkage time, which indicates that the estimated shrinkage of a concrete slab at the very early age is minimal, no matter the ambient relative humidity. Therefore, shrinkage is not considered in subsequent calculation of the early age stress development in concrete pavements.

Figure 6.10(b) shows the temperature and thermal stress development at the top of the pavement slab of the section with 0.70% longitudinal reinforcement on E17, placed at 2 am. The large amount of internal hydration heat coincides with the intensive solar radiation in the afternoon of the same day, which leads to a rapid concrete temperature increase after the final set. As a result, this section has the largest peak concrete temperature and zero-stress temperature. The plastic sheet cover was removed at 3 pm of the same day. A rapid temperature drop developed
due to the large amount of convection heat loss from the pavement due to the wind, and it thus leads to larger tensile stress. The field investigations have shown that the first series of cracks occurred within 26 hours after concrete placement. The estimated maximum thermal stress during the first night is 2.2 MPa and the concrete tensile strength at the corresponding time is 2.6 MPa. It indicates the high cracking potential during the first night for this section, especially when the variability of the mechanical properties of the concrete and its shrinkage are considered. A more accurate estimation of the cracking potential for concrete pavement during hardening is further recommended to account for the factors of shrinkage and probability.

Similar calculations were also performed for the other two subsections on E17, with 0.75% and 0.65% longitudinal reinforcement as shown in Figure 6.10(a) and (c), and one section on E313 as shown Figure 6.10(d). The estimated cracking time for the 0.75% section on E17 is during the second night after placement, which agrees quite accurately with the field observations that no cracks were found at the age of 40 hours, and 9 cracks developed between the age of 40 and 43 hours. However, the predicted cracking time for the 0.65% section on E17 is 43 hours during the second night, but 7 cracks were observed during the first night with a pavement age of 17.3 hours. The reason for the observed earlier cracking than expected in this section is not clear. To be sure, the plastic sheet for the subsection of km 46.4 to km 46.5 on E17 was removed in the first night after concrete placement. However, whether the surface treatment and the application of curing compound that should be applied at the same time, was actually applied then is not known by the author. If the surface treatment and the curing compound were not applied during the first night, a substantial amount of moisture loss will have occurred, resulting in larger shrinkage at the pavement surface. This possible considerable shrinkage may be the reason of the cracking during the first night for this subsection while the estimated thermal induced stress is far lower than the corresponding tensile strength.

Figure 6.10(d) shows the temperature and thermal stress development of the section (km 23.0~23.1) on E313, Herentals. The estimated thermal stress at the top of the slab during the subsequent night after construction is 2.3 MPa that is quite close to the corresponding tensile strength of the concrete of 2.6 MPa. An active control method was applied for the section that will be further discussed later on. During the subsequent night, 8 cracks were observed and they all initiated at the applied surface partial saw cuts.

It can be concluded that the applicability of the proposed temperature and thermal stress prediction models is well validated with field measurements. With those models, the effect of several important parameters, such as the time of placement, the curing method, climate conditions during the construction etc., on the development of the temperature and the thermal stress can be evaluated.
6.3 SENSITIVITY ANALYSIS

The early age concrete temperature and the corresponding thermal stress development in a concrete pavement is affected by complex interactions of numerous variables, including the general pavement structure variables, materials proportion and its properties, environment conditions, and construction conditions. A number of previous studies on the parameters sensitivity analysis of the development of temperature and thermal stress in a concrete pavement based on each proposed model are well documented (Schindler et al. 2002; Ge 2005; Yoen 2011). The variables that were found to have the largest effect on the maximum temperature, and zero-stress temperature, include the curing method, the time of placement, the climate conditions (cloud cover, wind speed) and the type of cement and its thermal properties. The final set time is more affected by cement hydration parameters and the use of admixtures.

However, it should be noted that the above-mentioned models are developed and validated according to the American CRCP design, construction practice, and environmental conditions, which may be not applicable for Belgium and the Netherlands CRCP projects. Moreover, the objective of this study is not to verify the sensitivity of all the above-mentioned variables. The major interest of this study is to investigate the effect of the specific climate conditions, the time of concrete placement, the consequence of the plastic sheeting curing method on the development of temperature and thermal stress under Belgium and the Netherlands conditions. The pavement structure and the concrete mixtures used on E17 are fixed as the baseline, as seen in Table 4.6 and Table 4.7. The effects of summer (July 15) and autumn (October 15) weather conditions are studied. Winter conditions are not considered because little concrete pavement construction is conducted in winter. The historical monthly average climate conditions of Ghent in July and October are summarized in Table 6.3. Then, the required short-interval climate inputs are produced following the methods presented in Section 4.4. The generated climate inputs used in the numerical simulations are shown in Figure 6.11. The heat of hydration parameters of the baseline mixture of E17 that were determined by isothermal conduction calorimetry tests are shown in Table 4.2. Lastly, the measured early age concrete mechanical properties for the specimen type 16, including the modulus, uniaxial tensile strength, and the fracture energy, are used in the simulations. These mechanical properties are described with the degree of hydration and are illustrated in Table 5.7. Rainy weather conditions are not analysed here because those atmospheric conditions, almost without exception, will result in little variation in the slab temperature and less drying shrinkage that is beneficial to the concrete.
Table 6.3 Historical monthly average climatic values at Ghent.

<table>
<thead>
<tr>
<th>Season</th>
<th>Monthly Averages of Climate Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_{\text{max}}$ (°C)</td>
</tr>
<tr>
<td>Summer (July 15)</td>
<td>23</td>
</tr>
<tr>
<td>Autumn (October 15)</td>
<td>15</td>
</tr>
</tbody>
</table>

Table 6.4 Construction and environmental variables and their ranges

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Range</th>
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</thead>
<tbody>
<tr>
<td>Placement season</td>
<td>--</td>
<td>summer autumn</td>
</tr>
<tr>
<td>Placement time on a day</td>
<td>--</td>
<td>8</td>
</tr>
<tr>
<td>Plastic sheet curing duration</td>
<td>hour</td>
<td>0</td>
</tr>
<tr>
<td>Cloud cover</td>
<td>--</td>
<td>sunshine cloudy overcast</td>
</tr>
<tr>
<td>Wind speed</td>
<td>m/s</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 6.11 Air temperature, dew point temperature, and solar radiation inputs used for parametric study.
The sensitivity analysis is conducted to evaluate the effect of different parameters on the following output results: final set time, built-in temperature gradient, maximum temperature, zero-stress temperature, and the potential cracking time. Moreover, one special interest of this study is to evaluate the effect of plastic sheet curing on the development of temperature and the corresponding thermal stress at the early age for the Belgium CRCP practice. The final goal is to give guidelines of the saw-cutting window for the active crack control method in CRCP. The final set time gives the lower limit for the saw-cutting operation to avoid ravelling. The estimated cracking time gives the upper limit of the saw-cutting window before the initiation of random uncontrolled cracks.

6.3.1 Effect of Paving Time

Concrete paving 24 hours a day and 7 days a week is common practice in CRCP rehabilitation projects in Belgium, especially for heavily trafficked motorways, to keep the construction period as short as possible. In addition, this construction schedule avoids the daily transverse construction joints that are time-consuming, inconvenient, and sometimes fail.

Figure 6.12 and Figure 6.13 show the effect of the concrete placement time on a day on the concrete slab temperature at various depths in summer and autumn conditions, respectively. Sunny condition is used for both seasons. The other variables are 3.5 m/s of wind speed, the concrete placement temperature chosen equal to the air temperature at placement, and without plastic sheet curing. The calculated thermal stress evolution at the top of the slab (25 mm below the surface) during the first few days after concrete placement is illustrated in Figure 6.14 and Figure 6.15, for summer and autumn conditions, respectively. The corresponding obtained zero-stress temperature and the potential cracking are summarized in Table 6.5 and Table 6.6, respectively.

In summer condition, a much higher temperature develops within the pavement slab due to the high air temperature, the intensive solar radiation, and the high concrete placement temperature. As discussed in section 4.12, the ambient temperature condition dominates the heat generation of cement hydration, a higher amount of generated heat of concrete hydration will occur in the high ambient temperature conditions. Moreover, it is noted that the slab temperature of morning placement shows a much higher maximum temperature compared to afternoon placement. Since the primary heat generation of cement hydration begins several hours after the cement is mixed with the water, the primary heat generation cycle for the concrete placed in the morning will occur coincidentally with the day’s peak ambient temperature and the most intensive solar radiation. In case the concrete is placed during the afternoon, the primary cycle will occur at the relatively lower ambient temperature condition of the evening following the construction day.
Figure 6.12 Effect of time of concrete placement on the concrete temperature development, summer construction, Belgium.
Figure 6.13 Effect of time of concrete placement on the concrete temperature development, autumn construction, Belgium.
Figure 6.14 Effect of concrete placement time on concrete temperature and thermal stress development for summer construction in Belgium conditions.

Table 6.5 Effect of concrete placement time on concrete temperature and thermal stress development for summer construction in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Placement time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Final set time (hour)</td>
<td>10.5  9.2   7.1</td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>-3.5 -9.2 -10.5</td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>42.0 40.1 37.6</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>16.9 14.0 10.1</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>39.4 38.8 36.9</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>19.3 15.5 11.4</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>23.8 21.0 21.6</td>
</tr>
</tbody>
</table>
Figure 6.15 Effect of concrete placement time on concrete temperature and thermal stress development for autumn construction in Belgium conditions.

Table 6.6 Effect of concrete placement time on concrete temperature and thermal stress development for autumn construction in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Placement time</th>
</tr>
</thead>
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<td>00:00</td>
</tr>
<tr>
<td>Final set time (hour)</td>
<td>15.4</td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>-4.9</td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>21.3</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>16.5</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>21.2</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>17.2</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>--</td>
</tr>
</tbody>
</table>
For autumn construction, the slab temperature is much lower than that of summer construction, and the effect of placement time is negligible. It is attributed to the lower and less fluctuating air temperature in autumn and the less intensive solar radiation as well.

Cracking itself is not a problem in CRCP. However, previous field observations did show that the cracks initiated during summer construction were significantly more meandering than those initiated during winter construction (Suh et al. 1992). Among cracks observed during summer construction, cracks occurring during the first night showed a more meandering shape and were wider than those occurring later. It is believed that the tendency of crack meandering at early age is caused by the relatively heterogeneous state of the freshly hardened concrete. The aggregate in the concrete, much stronger than the mortar at early ages, can cause cracks to extend in different directions. In spite of the closer crack spacing exhibited for the summer construction, crack widths were greater than those for cool season construction. A larger zero stress temperature will lead to more punchouts in the long term.

6.3.2 Effect of Plastic Sheet Cover

Exposed aggregate concrete is systematically applied in Belgium as the surface treatment for motorways and other important roads to reduce the rolling noise (Hendrickx 2006). In order to protect against drying out, the concrete in most cases is covered with plastic sheet immediately after spraying the setting retarder. Depending on the type of setting retarder used and the ambient weather, the surface treatment of exposing the aggregate at the surface of the hardened concrete can occur 6 to 24 hours after concreting.

Polyethylene sheeting curing is very beneficial in retraining moisture of fresh concrete that minimizes the plastic shrinkage damage and reduces the drying shrinkage as well (Schindler et al. 2002). In addition, polyethylene sheeting also acts as a thermal insulator. It can be beneficial when the placed concrete slab is subjected to a rapid cooling following construction. In this case, randomly premature cracks may initiate before the saw cuts are applied. However, the polyethylene sheeting curing method can be detrimental if used improperly. It may result in too high concrete temperature in the summer construction conditions that thus cause damages following placement. Figure 6.16 shows the measured concrete temperature at the top of the slab with and without the polyethylene sheet cover for a CRCP section placed in July in Texas, United States (Nam 2005). The difference between the maximum concrete temperatures at the 2nd day after paving is as high as 25 °F (≈14 °C) at the top of the slab.
To investigate the effect of the cover with the polyethylene sheet on concrete temperature and stress development, two simulations have been conducted for various duration of the plastic sheet cover in summer and autumn conditions, respectively, as shown in Figure 6.17 and Figure 6.18. The simulated examples include a morning placement (8 am) and an afternoon placement (4 pm). The other variables are sunny day, 3.5 m/s of wind speed, the concrete placement temperature chosen equal to the air temperature at placement. The optical properties of the polyethylene sheet that is commonly used in Belgium concrete pavement practice are summarized in Table 4.5.

Figure 6.17 and Figure 6.18 show the effect of the plastic sheet cover duration on the slab temperature and thermal stress development for the morning placement and afternoon placement in summer, respectively. The estimated zero-stress temperature and the potential cracking time are summarized in Table 6.7 and Table 6.8. Almost without exception, the application of a plastic sheet cover significantly increases the concrete temperature in summer construction. In the morning paving example, the peak concrete temperature at the top of the slab (25 mm below the surface) increases from 37.6 °C without plastic sheet cover to 45.3 °C when the pavement is covered by a plastic sheet during 12 hours. The peak temperature further increases when the plastic sheet is applied longer and the peak temperature can even occur at the second day after placement. It is observed that a considerable large positive temperature difference between the surface and bottom of the pavement slab, larger than 10 °C, develops at final set, which indicates a larger negative built-in temperature gradient. Another interesting finding is the rapid temperature drop when the plastic sheet is removed before the first night. Together with the corresponding lower developed tensile strength at that moment.
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Figure 6.17 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for summer construction at 8 am in Belgium conditions.

Table 6.7 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for summer construction at 8 am in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Plastic sheet covering duration (hour)</th>
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<td>Final set time (hour)</td>
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<td>Built-in temperature gradient (°C)</td>
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<td>Maximum temperature time (hour)</td>
<td>10.1</td>
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<tr>
<td>Zero-stress temperature (°C)</td>
<td>36.9</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>11.4</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>21.6</td>
</tr>
</tbody>
</table>
Figure 6.18 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for summer construction at 4 pm in Belgium conditions.

Table 6.8 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for summer construction at 4 pm in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Plastic sheet covering duration (hour)</th>
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</thead>
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<tr>
<td>Final set time (hour)</td>
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</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
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<tr>
<td>Maximum temperature (°C)</td>
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</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>24.0</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>33.1</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>28.9</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>36.5</td>
</tr>
</tbody>
</table>
Figure 6.19 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for autumn construction at 8 am in Belgium conditions.

Table 6.9 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for autumn construction at 8 am in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
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<tr>
<td>Final set time (hour)</td>
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</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
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</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>23.6</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
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<tr>
<td>Zero-stress temperature (°C)</td>
<td>19.1</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>37.0</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>--</td>
</tr>
</tbody>
</table>
Figure 6.20 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for autumn construction at 4 pm in Belgium conditions.

Table 6.10 Effect of duration of plastic sheet cover on concrete temperature and thermal stress for autumn construction at 4 pm in Belgium conditions.

<table>
<thead>
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<th>Parameters</th>
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<td>Final set time (hour)</td>
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<td>Built-in temperature gradient (°C)</td>
<td>2.7</td>
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<tr>
<td>Maximum temperature (°C)</td>
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</tr>
<tr>
<td>Maximum temperature time (hour)</td>
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<tr>
<td>Zero-stress temperature (°C)</td>
<td>21.0</td>
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<tr>
<td>Zero-stress temperature time (hour)</td>
<td>27.6</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>--</td>
</tr>
</tbody>
</table>
it leads to the primary crack initiating during the first night. It may cause unwanted random cracks before the saw cuts are implemented. In case of afternoon placement on the summer day, the peak temperature at the top of the slab occurs at the second day following the concrete paving regardless of the plastic sheet cover duration. The peak temperature and the zero-stress temperature are slightly lower for the afternoon paved sections as compared to the section placed in the morning. Lastly, it shows that no cracks do occur during the first night.

Figure 6.19 and Figure 6.20 illustrate the effect of plastic sheet cover duration on the slab temperature and thermal stress development for the morning placement and afternoon placement in autumn construction, respectively. The estimated zero-stress temperature and the potential cracking time are summarized in Table 6.9 and Table 6.10. It is clearly shown that the application of the plastic sheet curing method does not increase the zero-stress temperature considerably. A much more uniform temperature distribution is also observed at the final set of concrete for the autumn construction cured by plastic sheet.

If the plastic sheet curing method has to be used in sunny summer condition, due to the requirement of exposed aggregate surface treatment, one possible solution to avoid increasing the zero-stress temperature considerably is to select the adequate type of polyethylene sheeting. The optical properties of the polyethylene sheeting dominate the heat flux at the pavement surface thereby influencing the pavement temperature development in the early age. The type of polyethylene sheeting to be chosen depends on the concrete curing temperature. For instance, reflective sheeting should be used when curing temperatures exceed 30 °C (ASTM C171 2007). On the contrary, dark coloured sheeting is recommended in cold season construction to help increasing the curing temperature.

6.3.3 Effect of Substructure Temperature

The substructure temperature also influences the early age concrete pavement temperature development. However, it should be noted that the substructure temperature is not an independent parameter, but is normally determined by the paving time, especially during a summer sunny day when an asphalt interlayer with high solar radiation absorptivity is applied. Figure 4.15 shows the initial temperature profiles at various temperatures of a summer sunny day construction on E17. With respect to the substructure temperature, the substructure surface temperature difference between 8 am and 4 pm is up to 18.6 °C in a summer sunny day construction. When the substructure temperature is higher than the initial concrete temperature, the heat will conduct from the substructure to the concrete, which will increase the concrete pavement temperature and rate of heat of hydration.
Figure 6.21 Effect of substructure temperature on concrete pavement temperature development for summer construction at 4 pm in Belgium conditions.

Table 6.11 Effect of substructure temperature on concrete pavement temperature development for summer construction at 4 pm in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
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<th>Low base temperature</th>
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<td></td>
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<td>middle</td>
</tr>
<tr>
<td>Final set time (hour)</td>
<td>5.4</td>
<td>5.6</td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>3.0</td>
<td>3.4</td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
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<td>38.5</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
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<td>25.6</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
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<td>30.2</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
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<td>30.2</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>36.5</td>
<td>62.0</td>
</tr>
</tbody>
</table>

Figure 6.21 indicates the effect of the substructure temperature. A sunny summer placement is used because of larger daily temperature variations in the base layers. Two predetermined temperature profiles of base layers at 12 am and 4 pm, as shown in Figure 4.15, are used as inputs for the calculated example of concrete placement at 4 pm. The temperature at the surface of asphalt interlayer is 28.0 °C and 37.8 °C, respectively. The other variables are 3.5 m/s of wind speed, 12 hours curing with plastic sheet, and 25 mm below the concrete surface. When the substructure temperature increases from 28.0 °C to 37.8 °C, the zero-stress temperature increases by 0.5 °C, 1.0 °C, and 1.2 °C for the surface, middle, and the bottom of the concrete layer, respectively. After about 24 hours, the concrete temperature is similar regardless of the initial substructure temperature profile. As mentioned in section 6.1.6, the top of the slab is the most critical location for the zero-stress temperature and the potential of cracking. The substructure temperature has a smaller effect on the concrete top layer than the bottom layer. However,
Chapter 6 Active Crack Control for CRCP

it should be mentioned that the change of substructure temperature could have a considerable effect on the built-in curling temperature gradient for the morning placement.

6.3.4 Effect of Concrete Placement Temperature

Similar to the ambient air temperature, the concrete placement temperature is also critical for the early age concrete temperature development, and a few previous studies on this issue are well-documented (ACI Committee 305 1999; Schindler et al. 2002). A 35 °C limit of concrete placement temperature is proposed for nationwide concrete pavement construction in United States. The temperature of fresh concrete is determined by the temperature of its ingredients (Mehta and Monteiro 2006). The diurnal fresh concrete temperature is however rarely reported in the previous studies. In the above-mentioned analysis of the effect of paving time on the day and the plastic sheet cover on the temperature and the stress development, the placement temperature is simply taken the same as the air temperature at placement. Although this treatment has a slight error, it is not far away from reality. Moreover, the climate of the Netherlands and Belgium is characterized by cool summers and mild winters. Therefore, the very high concrete placement temperature is not a concern in the Netherlands and Belgium conditions. However, in order to illustrate the effect of the concrete placement temperature, the calculation of this variable is still given here.

A summer sunny day construction is used in the simulation. Besides assuming the concrete placement temperature the same as the air temperature at placement, 13.5 °C for 8 am placement and 23.0 °C for 4 pm placement adopted in this example, another concrete placement temperature of 18 °C is also chosen for both morning and afternoon constructions in the analysis of the effect of concrete temperature at placement. The other variables are 3.5 m/s of wind speed, 12 hours curing with plastic sheet and without plastic sheet. Figure 6.22 and Table 6.12 illustrate the effect of the concrete placement temperature for the morning paving section. The peak concrete temperature and the zero-stress temperature increase significantly, as the concrete placement temperature increases from 13.5 °C to 18 °C, more especially when the plastic sheet curing is applied. The peak concrete temperature can be up to 48.8 °C that may cause problems of unwanted shapes of cracks. Moreover, the effect of concrete placement temperature on the temperature development vanishes after about 30 hours. Figure 6.23 and Table 6.13 indicate this effect for the afternoon placement. When the concrete placement temperature increases from 18 °C to 23 °C, the zero-stress temperature will increase from 33.1 °C to 37.6 °C when it is covered with plastic sheet. It is also observed that the morning placement section would crack during the first night regardless of covered by the plastic sheet or not. For the afternoon placement section, the first series of cracks are expected to occur during the second night after concrete paving.
Figure 6.22 Effect of concrete placement temperature on concrete temperature and thermal stress for summer construction at 8 am in Belgium conditions.

Table 6.12 Effect of concrete placement temperature on concrete temperature and thermal stress for summer construction at 8 am in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Without plastic sheet curing</th>
<th>With plastic sheet curing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>13.5°C  18°C</td>
<td>13.5°C  18°C</td>
</tr>
<tr>
<td>Final set time (hour)</td>
<td>7.1     6.5</td>
<td>6.8     6.1</td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>-10.5   -9.9</td>
<td>-13.3   -12.9</td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>37.6    39.5</td>
<td>45.3    48.8</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>10.1    9.9</td>
<td>11.4    10.9</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>36.9    38.7</td>
<td>43.3    46.4</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>11.4    11.6</td>
<td>12.3    12.2</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>21.6    18.9</td>
<td>15.4    14.2</td>
</tr>
</tbody>
</table>
Figure 6.23 Effect of concrete placement temperature on pavement temperature for summer construction at 4 pm in Belgium conditions.

Table 6.13 Effect of concrete placement temperature on pavement temperature for summer construction at 4 pm in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Without plastic sheet curing</th>
<th>With plastic sheet curing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>18°C</td>
<td>23°C</td>
</tr>
<tr>
<td>Final set time (hour)</td>
<td>6.6</td>
<td>5.4</td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>3.7</td>
<td>3.0</td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>40.1</td>
<td>39.9</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>24.0</td>
<td>24.0</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>33.1</td>
<td>33.1</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>28.9</td>
<td>28.9</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>36.4</td>
<td>36.5</td>
</tr>
</tbody>
</table>
As mentioned previously, continuous construction work is common practice, thus, if concrete placement is scheduled in hot weather, treatments should be applied to reduce the zero-stress temperature, especially for the sections constructed in the morning. Many techniques are recommended to reduce the initial concrete temperature under hot weather summer conditions (ACI 305, 1999), but it can be very expensive in concrete pavement practice. It is therefore not recommended. A cheaper and more practicable method used in Germany (Springenschmid and Fleischer 2001), spraying additional water onto the surface of the plastic sheeting in warm summer condition, is however highly recommended. The pavement is cooled by the considerable amount of latent heat taken away when evaporative cooling occurs.

6.3.5 Effect of Environmental Conditions

The effect of plastic sheet cover on the temperature and the thermal stress development under various wind speeds and cloud covers are simulated for Belgium conditions. As discussed in section 4.1, cloud cover affects the intensity of the solar radiation that reaches the pavement surface, while wind speed determines the convection heat transfer coefficient, which determines the rate of heat energy transfer, along with the temperature difference between the concrete slab and the environment.

Figure 6.24 shows the effect of wind speed for a summer sunny day construction. The following wind speeds are chosen: 0.0 m/s, 3.5 m/s, 5.5 m/s, and 8.5 m/s. The other variables are 12 hours curing with plastic sheet, morning placement at 8 am, and the concrete placement temperature chosen equal to the air temperature at placement. The peak concrete temperature for the clear and calm summer day can reach as high as 50.9 °C, when the pavement slab is covered by a plastic sheet. It is therefore recommended to take treatments to decrease this too high concrete temperature.

Figure 6.25 shows the effect of cloud cover for summer construction. Two extreme clouds cover conditions, clear and overcast, are simulated. The other variables are 12 hours curing with plastic sheet and for overcast condition also 24 and 28 hours, morning placement at 8 am, 3.5 m/s of wind speed, and the concrete placement temperature chosen equal to the air temperature at placement. Without exception, the concrete temperature is much lower for the pavement constructed under the overcast condition as compared to that constructed on a clear day. It is also observed that the plastic sheet curing method does not increase the concrete pavement temperature for the overcast condition for Belgium CRCP practice.
Figure 6.24 Effect of wind speed on concrete temperature development for summer construction at 8 am in Belgium conditions.

Table 6.14 Effect of wind speed on concrete temperature development for summer construction at 8 am in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Wind speed (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.5¹</td>
</tr>
<tr>
<td>Final set time (hour)</td>
<td>7.1</td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>-10.5</td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>37.6</td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>10.1</td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>36.9</td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>11.4</td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>21.6</td>
</tr>
</tbody>
</table>

Note: ¹: without plastic sheet.
Figure 6.25 Effect of cloud cover on concrete temperature development for summer construction at 8 am in Belgium conditions.

Table 6.15 Effect of cloud cover on concrete temperature development for summer construction at 8 am in Belgium conditions.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Sunny</th>
<th>Overcast</th>
<th>Overcast</th>
<th>Overcast</th>
<th>Overcast</th>
</tr>
</thead>
<tbody>
<tr>
<td>Final set time (hour)</td>
<td>12</td>
<td>12</td>
<td>24</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td>Built-in temperature gradient (°C)</td>
<td>-10.5</td>
<td>-3.6</td>
<td>-3.6</td>
<td>-3.6</td>
<td></td>
</tr>
<tr>
<td>Maximum temperature (°C)</td>
<td>37.6</td>
<td>28.1</td>
<td>29.2</td>
<td>30.8</td>
<td></td>
</tr>
<tr>
<td>Maximum temperature time (hour)</td>
<td>10.1</td>
<td>32.9</td>
<td>32.3</td>
<td>28.1</td>
<td></td>
</tr>
<tr>
<td>Zero-stress temperature (°C)</td>
<td>36.9</td>
<td>25.5</td>
<td>28.2</td>
<td>28.9</td>
<td></td>
</tr>
<tr>
<td>Zero-stress temperature time (hour)</td>
<td>11.4</td>
<td>37.8</td>
<td>35.3</td>
<td>35.2</td>
<td></td>
</tr>
<tr>
<td>Potential cracking time (hour)</td>
<td>21.6</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td></td>
</tr>
</tbody>
</table>
6.4 CRCP ACTIVE CRACK CONTROL: A REVIEW

In 1993 McCullough and colleagues adopted active crack control methods on a large scale in four CRCP projects in Texas (McCullough and Dossey 1999; McCullough et al. 2000). The active methods used in these projects included shallow sawcut and metallic bar insertion. Field surveys revealed that the transverse cracks occurred much sooner and straighter, with a reduced number of clusters of closely spaced cracks. McCullough and colleagues recommended active crack control if the CRCP is placed at an air temperature exceeding 32 °C and constructed with aggregates that have a coefficient of thermal expansion greater than 7.92×10^{-6} mm/mm/°C.

Kohler and Roesler (2004) constructed 10 full-scale CRCP test sections at the Advanced Transportation Research and Engineering Laboratory in Illinois. Two transverse crack induction methods, early entry sawcut and automated tape insertion, were applied in 5 of 10 sections. The early entry sawcut was made approximately 4 hours after concrete placement, and consisted of a shallow notch, 38 mm in depth on the top of the pavement over the full width. In the tape insertion method a 3 mm thick and 75 mm deep plastic tape was inserted in the fresh concrete. The crack inducer spacing was set at 0.6, 1.2, 1.8 m for both methods. Regular crack surveys indicated that active crack control could quite effectively improve the crack pattern, i.e. more uniform, while straighter and early age transverse cracks were observed. Besides, it was found that in the tape insertion method the cracks developed slightly earlier than in the sawcut method. A transverse crack occurred at nearly all the notches, while on the other hand hardly any cracks occurred in between the notches.

Lim (2009) studied the cracking process in Portland cement concrete pavement by the saw cutting method. Factors affecting the efficiency of cracking included the ambient temperature, depth of sawcut, timing of sawcut, location of the sawcut, and concrete mix and subgrade properties. Lim conducted tension slab tests in the laboratory to investigate the effects of the shape of the sawcut (“V” shape, square or circular shape, and rectangular shape with rounded edges), location of initiators, timing of saw cutting, and depth of sawcut on the crack pattern.

The effectiveness of the active crack control methods for a concrete pavement is mainly dependent on the crack induction method, the operation timing, and the layout of the crack inducers.

6.4.1 Early Entry Method

Early entry saws are lightweight devices, which allow the sawing operation to begin as soon as 1 to 4 hours after concrete placement, depending on the concrete properties and weather conditions (CPTP 2007). In addition, most early entry saws use a dry-cutting operation with specially designed blades that do not require
water for cooling. Early sawing is believed to increase the probability that the cracks will be induced at the sawcut location. Besides, because the pavement is being sawed earlier, the depth of the sawing needed to initiate cracking can be reduced (Zollinger et al. 1994).

6.4.2 Metallic/Plastic Tape Insertion Method

Single layer and double layers metallic have been inserted into fresh concrete to act as crack inducers in Texas’ studies (Zollinger et al. 1998; McCullough and Dossey 1999; McCullough et al. 2000). These crack inducers were intended to induce bottom-up cracks in the concrete slab. Crack surveys showed that early age saw cuts were more effective than the metallic insertion method for crack induction. In contrast, where the plastic tape was inserted in the top part of CRCP slab in the test section in Illinois (Kohler and Roesler 2004), the field results showed that the cracks developed slightly earlier in the tape insertion method compared to the saw cutting method. It can be attributed to the location of the crack inducer. For the same pavement cross section reduction area, the crack inducer, intended to initiate cracks from the top part of the slab, is much more effective than that initiating cracks from the interior of the concrete, because the higher changes of moisture and temperature occurring at the surface of the slab help to initiate cracking.

6.4.3 Saw Cutting Timing

Timing plays a very important role in achieving the goal of crack induction, particularly at shallow notches. There is an ideal ‘saw-cutting window’. If the timing of the saw cutting operation is too early, ravelling of the concrete will occur because the concrete has not yet developed enough strength to resist the sawing machine. On the other hand, a too late saw cutting operation may result in random cracking due to the build-up of residual stresses (Okamoto et al. 1994). The later time limit is of particular concern because the longer that sawing is delayed, the greater the chance that random cracking may develop (Voigt 2002).

Previous experiences with early age saw cutting have indicated that the notches should be made between the initial and final setting of concrete. In general, the sawing operation should be conducted between 4 and 12 hours after paving (McCullough and Dossey 1999), but this period varies considerably depending on the constituent materials, mix properties, external restraint forces, and environmental factors. In addition, the saw cutting operation should be done without influencing the conventional construction execution scheme. It should not pose a risk to increase the construction difficulty and decrease the construction progress. Exposed aggregate surface is a common practice on Belgian CRCP motorways. In order to protect it against drying out, the concrete is covered with a plastic sheet as soon as the setting retarder has been applied. Therefore, the saw cutting is applied
immediately after the removal of the plastic sheet, which is around 6 to 24 hours after concrete placement.

6.4.4 Saw Cutting Depth

In the case of JPCP, the conventional depth of joint sawing is often taken to be one quarter of the slab thickness for transverse contraction joints in AASHTO 1993. The American Concrete Pavement Association suggested that the depth of the sawcut should be at least one-third of the slab thickness (Fuchs and Jasienshi 1997), which is also common practice in Europe. However, considering the potential of corrosion of the longitudinal steel rebars, CRCP requires a sufficient concrete cover, so the sawcut depth for active crack control in CRCP cannot go as deep as that used in JPCP. Besides, Zollinger at al. (1994) advocate that a shallow cut, usually at least 25 mm, may be adequate if the sawing is done early enough. The argument is that the shallower saw cut takes advantage of the significant changes in moisture and temperature conditions at the surface of the slab to help initiate cracks at the tip of saw cuts. The standard position of the longitudinal reinforcement is above the middle of the slab. In general, the concrete cover over the longitudinal rebars amounts 80 mm in Belgium. Therefore, the adequate depth of the sawcut in CRCP could be within 30 to 60 mm.

6.5 CRCP ACTIVE CRACK CONTROL TEST SECTIONS IN BELGIUM

A new active crack control procedure for CRCP was proposed by Luc Rens. It was inspired by an interesting finding during a field inspection of CRCP roundabouts in Belgium, where he found some transverse (radial) cracks that looked like they were induced by the contraction joint of the adjacent inner circle of the roundabout (Rens and Beeldens 2013). Besides, the idea was based on the American experiences with the shallow saw cut method in active crack control for CRCP. This new active crack control procedure was firstly applied in the reconstruction project of motorway E313.

The reconstruction project E313 between Antwerp and Herentals was conducted in 2012. Figure 6.26 shows the layout of the test sections on E313, which was constructed according to the current standard practice in Belgium: 250 mm thick CRCP slab laid on a 50 mm bituminous interlayer and a lean concrete base. The longitudinal reinforcement steel amounts 0.75%, and the position of the longitudinal steel reinforcement is about 80 mm below the pavement surface. Besides, due to the noise reduction requirement and economic considerations, two-lift construction was adopted for the concrete slab, the thickness of the top and the bottom layer is 50 mm and 200 mm, respectively.

As shown in Figure 6.26(c), during hardening of the concrete partial surface notches were sawn at the outer side of the pavement slab, the length of the notch is
400 mm, and the spacing is 1.20 m. The cut was applied immediately after the
washing out of the surface of the pavement, generally within 16 hours after
concrete placement. Regarding the face of the notches, the end of the notch would
not have a flat face but a curved face as the saw blade is circular as illustrated in
Figure 6.26(b).

The E313 project contains two crack control test sections. During the first phase
of this reconstruction project, the saw cut depth is only 30 mm, which is around
one eighth of the concrete slab thickness. Subsequently, in order to evaluate the
effect of the saw cut depth on the effectiveness of crack initiation, the depth was
increased to 60 mm during the subsequent phase of this project. It should be
mentioned that the time of saw cutting of 60 mm depth section was a few hours
earlier than that of the 30 mm depth test section.

One 500 m long test section at the outer lane with 30 mm depth saw cut and a
1100 m long section also at the outer lane with 60 mm depth sawcut has been
chosen for regular crack surveys right from the placement of concrete. The 500 m
long section with 30 mm saw cut was constructed in July 2012 while the 1100 m
long section with 60 mm saw cut was constructed in September 2012.

Crack spacing surveys were conducted by manual visual survey, walking
along the emergency lane, recording the location and the shape of the cracks, and
define the category of each crack. The influence of traffic loading on crack
development is not included. Experiences of field inspections had shown that the
 crack pattern in the emergency lane was slightly different from that in the traffic
lane. A few more cracks were found in the traffic lane that stopped at the
longitudinal contraction joint. Crack widths at the pavement surface were
measured with a microscope having a resolution of 0.01 mm. Periodical crack
pattern surveys have been performed at the first week, 2 months, 7 months, 1 year
and 1.5 years after construction, respectively.
Figure 6.26 Schematic view of active crack control test section on E313 (a) plan layout of applied notches, joints and lanes; (b) notch face; (c) notch interval, notch length on the pavement surface, and notch depth.
6.6 SAW CUT MODEL

Regarding the notches, the end of the notch (that is, the end "in" the pavement) does not have a flat face but a curved face as the saw blade is circular, as shown in Figure 6.26(b). In order to simplify the estimation of the stress intensity factors, this notch is considered as a corner quarter-elliptical surface notch in a plate of finite thickness.

![Figure 6.27. Schematic view of a corner quarter-elliptical notch in a finite plate.](image)

As discussed in the Section 6.2 of thermal stress calculation of an unnotched concrete pavement, it exhibits high nonlinearity over the depth of the concrete slab. The stress intensity factors for the corner quarter-elliptical crack in a finite plate under pure tension and bending can be found in handbooks (Murakami and Hasebe 2001). The stress intensity factors for a nonlinear stress distribution other than the known stress distribution in handbooks can be calculated using numerical methods. However, this usually involves a substantial computational effort. Therefore, the weight function method proposed by Bueckner (1970) and Rice (1972) is used in this study. Shen and Glinka (1991) have used the weight function method to estimate the stress intensity factor for corner quarter-elliptical cracks. With this method, a stress intensity factor $K$ can be calculated for one-dimensional and two-dimensional cracks by integrating the product of the uncracked stress field $\sigma$ and weight function $m$ along a perimeter $\Gamma$ in the prospective crack plane.

$$K = \int_{\Gamma} \sigma m d\Gamma$$ (6.18)

Universal forms of weight functions for surface corner quarter-elliptical cracks in a finite plate are given as (Glinka and Shen 1991):
For the deepest point in the profile plane, \( A \)

\[
m_A(z, a) = \frac{2}{\sqrt{2\pi(a - z)}} \left[ 1 + M_{1A} \left( 1 - \frac{z}{a} \right)^{1/2} + M_{2A} \left( 1 - \frac{z}{a} \right) + M_{3A} \left( 1 - \frac{z}{a} \right)^{3/2} \right]
\]  

(6.19)

For the surface point in the frontal plane, \( B \)

\[
m_B(z, a) = \frac{2}{\sqrt{\pi z}} \left[ 1 + M_{1B} \left( \frac{z}{a} \right)^{1/2} + M_{2B} \left( \frac{z}{a} \right) + M_{3B} \left( \frac{z}{a} \right)^{3/2} \right]
\]  

(6.20)

The derivation of parameters \( M_{iA} \) and \( M_{iB} \) can be found in Appendix II.

The stress intensity factors vary along the crack front of the elliptical corner crack. In the pure tension and bending loading conditions, the maximum stress intensity factor is found at the deepest point \( A \) while the lowest one is found at the surface point \( B \) (as shown in Figure 6.27). As a non-linear and time dependent stress field \( \sigma(z) \) is present in the pavement, the stress intensity factor for point \( A \) or \( B \) can be calculated by integrating the stress distribution \( \sigma(z) \) and the weight function \( m_A(z, a) \) or \( m_B(z, a) \) along the crack face, respectively.

\[
K^A = \int_0^a \sigma(z) m_A(z, a) dz
\]  

(6.21)

\[
K^B = \int_0^a \sigma(z) m_B(z, a) dz
\]  

(6.22)

It is emphasized that the stress intensity factors can be considered only as average value associated with the boundary layer near the points \( A \) and \( B \), as shown in Figure 6.27. The reason is that the "\( r^{-1/2} \)" singularity of the near crack tip stress field vanishes at the inter-section of three free surfaces. The absence of the fundamental linear elastic fracture mechanics stress field singularity "\( r^{-1/2} \)" implies that the classical stress intensity factor vanishes. It is known that the "\( r^{-1/2} \)" stress singularity occurs only near a crack front that is embedded entirely in the material. A small layer away from the free surface the "\( r^{-1/2} \)" singularity dominates the magnitude of the stress intensity factor. Therefore, it is assumed that the surface layers near the point \( A \) and \( B \) have "\( r^{-1/2} \)" singularity as remaining layers. This approximation may be sufficient to estimate the stress intensity factors at point \( A \) and \( B \).

Both the results of thermal stress calculations and field surveys have indicated that the cracking of the pavement mainly occurs in the negative temperature gradient condition when the tensile stress is located at the top part of the pavement slab. Figure 6.28 shows the results of the stress intensity factors \( K \) at point \( A \) and \( B \) under uniform tensile stress \( \sigma_0 \). It is clearly shown that the larger the notch depth \( a \)
the higher the stress intensity factor $K^A_0$. However, considering the practical issue of the necessary concrete cover of the longitudinal reinforcement rebars of CRCP, there is an upper limit to the notch depth $a$.

![Stress Intensity Factors](image_url)

Figure 6.28 Stress intensity factors $K$ at point A and B of a corner quarter-elliptical notch in a finite width pavement slab under uniform tensile stress $\sigma_0$, the pavement thickness $h=250$ mm.

The longitudinal reinforcement rebars are placed at the depth of 80 mm below the pavement surface in the current standard CRCP structure in Belgium. Therefore, the larger notch depth of 60 mm applied on E313 may be considered as the limitation of the notch depth in this active crack control method. As clearly shown in Figure 6.28, the notch depth of 60 mm has a larger stress intensity factor at point A compared to the notch depth of 30 mm, and a 60 mm notch is therefore more prone to initiate a crack at the tip of the notch. With respect to the notch length $c$ at the pavement surface, the stress intensity factor at point B decreases with increasing notch length $c$ for a given notch depth $a$. However, the increment of the stress intensity factors at point A increases with increasing notch length. The increment of the stress intensity factor at point A is not significant, when the notch length $c$ exceeds 400 mm. The stress intensity factor at point B shows a reverse tendency: the larger the notch length $c$, the lower the stress intensity factor. It can be interpreted that the larger notch length indicates a lower aspect ratio $a/c$, and the lower aspect ratio means a larger curvature at point B and a lower curvature at point A, respectively. In addition, the corner surface saw cuts were not sealed, an increase of the notch length $c$ to more than 400 mm will result in hardly any further increase of the stress intensity factor at point A, but it would increase the risk of the
unsealed notches get deteriorated due to the traffic tire wear on the contrary. Overall, it is concluded that the applied notch length $c$ of 400 mm on E313 is appropriate.

### 6.6.1 Stress Amplification Factors

In case of a typical uncut CRCP pavement, the comparison of the development of the tensile strength and tensile stress can be used to assess when a transverse crack may occur. However, when a saw cut is placed in a pavement, the above cracking criterion fails to capture the cracking tendency as the introduction of a saw cut reduces the stress that is required to cause cracking. Several studies have adopted the principle of Linear Elastic Fracture Mechanics (LEFM) to evaluate the notch sensitivity in concrete pavement. The critical stress $\sigma_{\text{max}}$ for instable crack growth of a corner surface notched concrete pavement as applied on E313 can be expressed by the following equation:

$$\sigma_{\text{max}} = \frac{K_{IC}}{\sqrt{\pi a} \cdot F(a, c, h, W)} \leq f_t$$  \hspace{1cm} (6.23)

Where, $K_{IC}$ is the critical stress intensity factor or fracture toughness of concrete, that can be either directly measured through a three-point bending test (Zollinger et al. 1993) or indirectly estimated through the measured fracture energy and elastic modulus (De Schutter and Taerwe 1997). $F(a, c, h, W)$ is the geometry correction factor and varies along the crack face. This geometry factor can be estimated by the weight function method. In case of this corner quarter-elliptical notch applied on E313, the largest stress intensity factor is found at point A (as shown in Figure 6.27) and thus the critical tensile stress $\sigma_{\text{max}}$ is determined at this point. Lastly, $a$ is the notch length along the pavement depth as shown in Figure 6.27 and $Q$ is the shape factor for an elliptical notch as indicated in Appendix II.

The validity of the applicability of the LEFM of Equation (6.23) requires a minimum notch size that has to be fulfilled. For example, for a fixed fracture toughness $K_{IC}$, the estimated $\sigma_{\text{max}}$ will become infinite large in case of either a small notch size $a$ or a thick concrete structure. This is not true as the estimated critical tensile stress $\sigma_{\text{max}}$ should not exceed the tensile strength of the uncut pavement $f_t$. In other words, this failure stress is limited by the tensile strength criteria when the notch size is smaller than the critical size $(a/D)_{\text{critical}}$. $D$ is a characteristic of the structure dimension, and in the case of a concrete pavement it is chosen as the pavement thickness. Weiss (1999) argued that the critical saw cut size is approximately equal to the maximum aggregate size.

In order to assess how close a pavement is to cracking when a saw cut is made, a stress enlargement factor $\phi_{\text{sawcut}}$ is introduced by dividing the tensile strength $f_t$
by the critical tensile stress $\sigma_{\text{max}}$ for unstable crack growth (Raoufi et al. 2008), as given in Equation (6.24).

$$
\phi_{\text{sawcut}} = \frac{f_t}{\sigma_{\text{max}}} = \frac{f_t}{K_{IC}} \geq 1
\sqrt{\frac{\pi a}{Q}} \cdot F(a, c, h, W)
$$ (6.24)

The stress enlargement factor $\phi_{\text{sawcut}}$ is expected to be 1.0 when there is no saw cut in the pavement, and when the saw cut size is smaller than a critical saw cut size to depth ratio as well. After the critical saw cut size to depth ratio, the stress enlargement factor $\phi_{\text{sawcut}}$ is expected to increase as the saw cut depth increases, as shown in Figure 6.29.

![Figure 6.29](image_url)

**Figure 6.29** Schematic of the effect of the notch size on the stress enlargement factor $\phi_{\text{sawcut}}$.

In contrast to previous analyses of notch sensitivity for the hardened concrete, in this study the stress enlargement factor $\phi_{\text{sawcut}}(t)$ varies with the development of the fracture toughness and the tensile strength of the concrete in the field. Besides, the geometry correction factor $F(a, c, h, W, t)$ depends not only on the geometry and on the location of the crack, but also on the stress distribution in the uncracked component. Because of the varying temperature distribution of the pavement and subsequently the changing stress distribution in the pavement, the geometry correction factor $F(a, c, h, W, t)$ varies as well. However, for simplicity, a uniform stress distribution taken equal to the stress at the pavement surface, along the pavement depth is used in the subsequent calculations. Besides, this uniform
stress distribution is the critical stress condition that leads to the largest stress intensity factor at the point A.

\[
\Phi_{\text{sawcut}}(t) = \frac{f_t(t)}{\sigma_{\text{max}}(t)} = \frac{f_t(t)}{K_{IC}(t)} \sqrt{\frac{\pi a}{Q}} F(a,c,h,W,t)
\] (6.25)

The following Equation (6.26) illustrates a criterion that can be used to indicate the age of the concrete before a saw cut should be placed. It is proposed that unstable cracking will occur when the product of the tensile stress \(\sigma(t)\) at the pavement surface and \(\Phi_{\text{sawcut}}(t)\) exceeds the tensile strength of the concrete. This unstable cracking criterion also indicates the latest age that a specific notch size should be placed.

\[
\begin{align*}
\sigma(t) \cdot \Phi_{\text{sawcut}}(t) & \leq f_t(t) & \text{stable cracking} \\
\sigma(t) \cdot \Phi_{\text{sawcut}}(t) & > f_t(t) & \text{unstable cracking}
\end{align*}
\] (6.26)

### 6.6.2 An Example of Amplification of \(\Phi_{\text{sawcut}}\)

Figure 6.30 illustrates the estimated stress and tensile strength development at the pavement surface for a nighttime constructed CRCP section (km 23.0) on E313. First, the concrete temperature and the corresponding degree of hydration within the pavement slab were estimated through the proposed temperature prediction model in Chapter 4. Then, the development of the tensile strength \(f_t\), elastic modulus \(E_c\), and fracture energy \(G_F\) of the field concrete were calculated by the proposed degree of hydration based concrete properties models at early age in Chapter 5. The critical tensile stress condition \(\sigma_{\text{surface}}\) at the pavement surface, was then calculated using the proposed procedure as shown in section 6.2. The input data of concrete thermal and mechanical properties, pavement structure, construction, curing, and climatic conditions, were explained in section 6.2.5. During the construction phase, the 60 mm deep and 400 mm long corner surface notch with an interval of 1.2 m were made at the age of approximately 16 hours after concrete placement. The first series of transverse cracks were observed at the age between 24 and 31 hours after concrete placement. As shown in Figure 6.30, in case of no saw cut was placed, the estimated tensile stress exceeds the tensile strength of the concrete at an age of about 50 hours after concrete placement, which is much later than the observed cracking time. Therefore, the effect of the saw cuts on the crack initiation and the latest time that the saw cuts should be placed are evaluated using the above-mentioned model in section 6.6.1.

Table 6.16 illustrates the estimated tensile strength \(f_t\), elastic modulus \(E_c\), and fracture energy \(G_F\), tensile stress \(\sigma\) at the age of 10 hours, 12 hours, 16 hours, 24 hours, 32 hours, 56 hours, and 672 hours, respectively, for the section km 23.0 on
E313. The fracture toughness $K_{IC}$ is calculated according to Equation (5.16), using the fracture energy $G_F$. The critical tensile stresses $\sigma_{\text{max}}$ for the 60 mm depth saw cut that is required to induce unstable cracking are calculated by Equation (6.23).

![Figure 6.30](image_url) Effects of sawcut on cracking through stress enlargement factor.

**Table 6.16** The development of stress enlargement factors for the section of km 23.0 on E313 (fracture toughness of concrete estimated by $G_F$).

<table>
<thead>
<tr>
<th>$t$ (hour)</th>
<th>$\alpha_h$ (%)</th>
<th>$E_c$ (MPa)</th>
<th>$G_F$ (N/mm)</th>
<th>$K_{IC}$ (MPa m$^{1/2}$)</th>
<th>$f_t$ (MPa)</th>
<th>$\sigma_{\text{surface}}$ (MPa)</th>
<th>$\sigma_{\text{max}}$ (MPa)</th>
<th>$\Phi_{\text{sawcut}}$ (%)</th>
<th>$\sigma_{\text{surface}} \cdot \Phi_{\text{sawcut}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.141</td>
<td>8195</td>
<td>0.21</td>
<td>0.415</td>
<td>0.44</td>
<td>-0.09</td>
<td>0.87</td>
<td>0.51</td>
<td>-0.05</td>
</tr>
<tr>
<td>12</td>
<td>0.203</td>
<td>14188</td>
<td>0.032</td>
<td>0.675</td>
<td>0.92</td>
<td>-0.58</td>
<td>1.41</td>
<td>0.65</td>
<td>-0.38</td>
</tr>
<tr>
<td>16</td>
<td>0.357</td>
<td>22824</td>
<td>0.046</td>
<td>1.028</td>
<td>1.76</td>
<td>-0.81</td>
<td>2.15</td>
<td>0.82</td>
<td>-0.67</td>
</tr>
<tr>
<td>24</td>
<td>0.508</td>
<td>28740</td>
<td>0.055</td>
<td>1.261</td>
<td>2.40</td>
<td>1.29</td>
<td>2.64</td>
<td>0.91</td>
<td>1.18</td>
</tr>
<tr>
<td>32</td>
<td>0.569</td>
<td>30786</td>
<td>0.058</td>
<td>1.340</td>
<td>2.64</td>
<td>2.28</td>
<td>2.80</td>
<td>0.94</td>
<td>2.15</td>
</tr>
<tr>
<td>48</td>
<td>0.645</td>
<td>33145</td>
<td>0.062</td>
<td>1.431</td>
<td>2.91</td>
<td>2.58</td>
<td>2.99</td>
<td>0.97</td>
<td>2.50</td>
</tr>
<tr>
<td>56</td>
<td>0.664</td>
<td>33707</td>
<td>0.063</td>
<td>1.452</td>
<td>2.98</td>
<td>3.79</td>
<td>3.04</td>
<td>0.98</td>
<td>3.70</td>
</tr>
<tr>
<td>672</td>
<td>0.860</td>
<td>39000</td>
<td>0.070</td>
<td>1.652</td>
<td>3.63</td>
<td>--</td>
<td>3.46</td>
<td>1.05</td>
<td>--</td>
</tr>
</tbody>
</table>

As shown in Table 6.16, the calculated $\sigma_{\text{max}}$ is found however higher than the concrete tensile strength $f_t$ at early age, which is not realistic. It may indicate that the fracture toughness cannot be indirectly calculated by the fracture energy $G_F$. As discussed in section 5.3.5, the fracture toughness can be indirectly estimated by the critical energy release rate $G_f$ and elastic modulus $E_c$ for brittle material based on LEFM. Bažant et al. (2002) presented the different physical meanings of fracture energy $G_f$ and $G_F$: the total fracture energy $G_F$ represents the area under the
complete tensile softening curve of concrete, while, the critical energy release rate $G_f$ represents the area under the initial tangent of the softening curve. Bažant et al. (2002) interprets these two fracture energies to be appropriate for different objectives: $G_f$ is suitable for predicting the maximum loads of the concrete structure, while $G_F$ is suitable for calculating the energy dissipation in the total failure of structures and for determining the tail of the post-peak softening behaviour. Bazant proposed a very rough estimation $G_F / G_f \approx 2.5$ for concrete (Planas et al. 1992; Guinea et al. 1994; Bažant 2002; Bažant et al. 2002). Table 6.17 illustrates the calculated failure stress and the corresponding stress enlargement factors according to the indirect obtained fracture toughness $K_{IC}$ through $G_f$ ($G_f = 0.4 \cdot G_F$). The estimated failure stress is found to be smaller than the tensile strength of concrete except for the very age concrete of 10 hours. For this given notch size of 60 mm notch depth, the stress enlargement factor $\Phi_{sawcut}$ is found to increase rapidly at the early age (before 48 hours) and then approaching a constant value. The estimated unstable cracking time by $G_f$ is about 27 hours, as shown in Figure 6.30, which agrees well with field observations.

Table 6.17 The development of stress enlargement factors for the section of km 23.0 on E313 (fracture toughness of concrete estimated by $G_f$).

<table>
<thead>
<tr>
<th>t (hour)</th>
<th>$\alpha_h$ (°)</th>
<th>$E_c$ (MPa)</th>
<th>$G_f$ (N/mm)</th>
<th>$K_{IC}$ (MPa$\sqrt{m}$)</th>
<th>$f_t$ (MPa)</th>
<th>$\sigma_{surface}$ (MPa)</th>
<th>$\sigma_{max}$ (MPa)</th>
<th>$\Phi_{sawcut}$ (°)</th>
<th>$\sigma_{surface}$ $\times \Phi_{sawcut}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.141</td>
<td>8195</td>
<td>0.008</td>
<td>0.262</td>
<td>0.44</td>
<td>-0.09</td>
<td>0.55</td>
<td>0.80</td>
<td>-0.07</td>
</tr>
<tr>
<td>12</td>
<td>0.203</td>
<td>14188</td>
<td>0.013</td>
<td>0.426</td>
<td>0.92</td>
<td>-0.58</td>
<td>0.89</td>
<td>1.03</td>
<td>-0.60</td>
</tr>
<tr>
<td>16</td>
<td>0.357</td>
<td>22824</td>
<td>0.018</td>
<td>0.648</td>
<td>1.76</td>
<td>-0.81</td>
<td>1.36</td>
<td>1.30</td>
<td>-1.05</td>
</tr>
<tr>
<td>24</td>
<td>0.508</td>
<td>28740</td>
<td>0.022</td>
<td>0.795</td>
<td>2.40</td>
<td>1.29</td>
<td>1.66</td>
<td>1.44</td>
<td>1.86</td>
</tr>
<tr>
<td>32</td>
<td>0.569</td>
<td>30786</td>
<td>0.023</td>
<td>0.845</td>
<td>2.64</td>
<td>2.28</td>
<td>1.77</td>
<td>1.49</td>
<td>3.40</td>
</tr>
<tr>
<td>48</td>
<td>0.645</td>
<td>33145</td>
<td>0.025</td>
<td>0.907</td>
<td>2.91</td>
<td>2.58</td>
<td>1.90</td>
<td>1.53</td>
<td>3.96</td>
</tr>
<tr>
<td>56</td>
<td>0.664</td>
<td>33707</td>
<td>0.025</td>
<td>0.922</td>
<td>2.98</td>
<td>3.79</td>
<td>1.93</td>
<td>1.55</td>
<td>5.86</td>
</tr>
<tr>
<td>672</td>
<td>0.860</td>
<td>39000</td>
<td>0.028</td>
<td>1.045</td>
<td>3.63</td>
<td>--</td>
<td>2.19</td>
<td>1.66</td>
<td>--</td>
</tr>
</tbody>
</table>

Discussion

The tensile property of concrete at very early age, in this case as early as the time of the initial setting of concrete, is essential for the study of cracking control. However, the indirect estimation of fracture toughness through the measured fracture energy $G_F$ by this rough approximation $G_f = 0.4 \cdot G_F$ is worth to be further verified. Actually, there is very limited relevant information available, especially on the direct measurement of the fracture toughness or the complete tensile softening curves for early age concrete. This is mainly due to practical problems, for instance the very early age concrete specimens with low degree of hydration cannot yet support their own weight without applying any loads, to be overcome in the experimental determination of the these properties (Dao et al. 2009). Recently, however, several research projects have adopted a new direct tensile test,
a horizontal tensile test set-up, to obtain the concrete tensile properties at an age as early as 1.5 to 4 hours (Hannant et al. 1999; Dao et al. 2009; Dippenaar 2015). The fracture properties, fracture toughness or fracture energy, at the age of about 6 to 24 hours, were reported through various types of the indirect tensile tests: three-point bending tests (Brameshuber and Hilsdorf 1989; Zollinger et al. 1993; De Schutter and Taerwe 1997); wedge splitting tests (Østergaard 2003; Kim et al. 2004; Østergaard et al. 2004; Gaedicke et al. 2007), and split tension tests (Tang et al. 1996; Tang et al. 1999). The available experimentally determined fracture energy and fracture toughness of concrete at the very early age, more specifically for those having similar compositions to the typical paving concrete, are summarized in Figures 6.31 and Figure 6.32, respectively. Besides, the estimated fracture energy for the hardened concrete according to the available prediction models (Bažant et al. 2002; fib 2010) are plotted in Figure 6.31. The fracture toughness for the hardening concrete is predicted based on the only available fracture toughness prediction model (Zollinger et al. 1993), to my knowledge, in combination with the indirect calculated fracture toughness at 28 days.

The following findings can be concluded from Figure 6.31 and Figure 6.32. First, the fracture energy $G_F$ of concrete and fracture toughness before final set is negligible and it increases very rapidly after the final set of concrete, which is similar to the development of tensile strength of concrete. Second, there is a wide range of measured fracture energy at 28 days due to different concrete compositions and various testing methods. An even larger range is found for the measured fracture toughness of concrete.

![Figure 6.31 Summary of experimentally obtained early age concrete fracture energy for normal concrete.](image-url)
6.6.3 Effectiveness of Crack Initiation through Field Investigation

Periodically crack pattern surveys have been performed on E313, specifically during the first week and 2 months, 7 months, 1 year and 1.5 years after construction, respectively. Table 6.18 shows the effectiveness of the crack initiation on the test sections with different saw cut depths on E313. For the 60 mm depth saw cut section, 99% of the cracks that occurred during the first 4 days, did occur at a saw cut, 21.3% of the saw cuts had propagated into a crack. This percentage rapidly increased to 61.9% about 2 months after construction. After that, the effects of the saw cuts on inducing new cracks beneath the notch became small as the percentage increased to 66.6% after the first winter, 7 months after construction. It indicates that this partial surface saw cut especially induces cracks beneath the notch during the very early age of the pavement, which is normally within the first 2 months after construction. However, the saw cuts remain quite effective soon afterwards, for instance, 43 of the total 97 new occurred cracks (approximately 45%) were located at the notches during the period between 65 and 204 days with the 60 mm depth section. After the first winter on the section with 60 mm depth notches, 78.4% of the cracks were located at the induced saw cuts, while this value is lower for the 30 mm deep notches, 56.5%. It indicates that the larger saw cut depth is more effective in initiating cracks at the design locations. It should however be noted that the saw cutting timing of the 60 mm notches was slightly earlier than that of the 30 mm notches. In both sections, cracks rapidly developed during the first few months with decreasing temperature, whereas very few new cracks occurred during the warmer months.
Table 6.18 Percentage of Cracks Initiated at the Notches on E313

<table>
<thead>
<tr>
<th>Section</th>
<th>Length (m)</th>
<th>Age (day)</th>
<th>Number of Notches (N1)</th>
<th>Number of Cracks (N2)</th>
<th>Number of Cracks at Notches (N3)</th>
<th>Effectiveness of the Notches N3/N1 (%)</th>
<th>Percentage of cracks in category (%)</th>
<th>Distance to nearest notch (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60 mm</td>
<td>1100</td>
<td>1</td>
<td>897</td>
<td>1</td>
<td>1</td>
<td>1.1</td>
<td>100</td>
<td>0.0 0.2 0.4 0.6</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>2</td>
<td>897</td>
<td>73</td>
<td>71</td>
<td>7.9</td>
<td>97.3</td>
<td>0.0 0.2 0.4 0.6</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>3</td>
<td>897</td>
<td>163</td>
<td>161</td>
<td>17.9</td>
<td>98.8</td>
<td>0.0 0.2 0.4 0.6</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>4</td>
<td>897</td>
<td>193</td>
<td>191</td>
<td>21.3</td>
<td>98.9</td>
<td>0.0 0.2 0.4 0.6</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>65</td>
<td>897</td>
<td>664</td>
<td>555</td>
<td>61.9</td>
<td>83.5</td>
<td>2.4 7.7 6.4</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>204</td>
<td>897</td>
<td>762</td>
<td>597</td>
<td>66.6</td>
<td>78.4</td>
<td>3.8 9.8 8.0</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>378</td>
<td>897</td>
<td>775</td>
<td>606</td>
<td>67.6</td>
<td>78.2</td>
<td>3.8 9.9 8.1</td>
</tr>
<tr>
<td></td>
<td>1100</td>
<td>555</td>
<td>897</td>
<td>803</td>
<td>628</td>
<td>70.0</td>
<td>78.2</td>
<td>3.6 10.1 8.1</td>
</tr>
<tr>
<td>30 mm</td>
<td>500</td>
<td>123</td>
<td>422</td>
<td>417</td>
<td>245</td>
<td>58.1</td>
<td>58.7</td>
<td>9.4 15.9 16.0</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>262</td>
<td>422</td>
<td>497</td>
<td>281</td>
<td>66.5</td>
<td>56.5</td>
<td>8.7 17.5 17.3</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>436</td>
<td>422</td>
<td>502</td>
<td>285</td>
<td>67.5</td>
<td>56.5</td>
<td>8.6 17.3 17.6</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>613</td>
<td>422</td>
<td>505</td>
<td>286</td>
<td>67.8</td>
<td>56.6</td>
<td>8.7 17.2 17.5</td>
</tr>
</tbody>
</table>
6.7 CONCLUSIONS

This chapter firstly describes the process of predicting thermal stress development in CRCP during the first few days after concrete placement. With the calculated temperature profiles, the restraint thermal stress is calculated with the superposition method considering the relaxation behaviour of young concrete. Zero-stress temperature can be estimated with this developed thermal stress model. The proposed thermal stress model was then verified with two CRCP projects in Belgium. Detailed parameter analyses were subsequently performed with the major interest to study the effect of the specific climate conditions and the consequence of the plastic sheeting curing method on the development of temperature and thermal stress under Belgium and the Netherlands conditions. In order to eliminate the non-uniform crack pattern, a new partial surface saw cut for active crack control was proposed and applied in E313. The adequate saw cut time for this active crack control method is evaluated by the above-mentioned thermal stress model and parameter analysis for the Belgium climate and CRCP construction practice. Saw cut geometry was then theoretically evaluated with the calculated thermal stresses distribution and the measured concrete mechanical properties by fracture mechanic analysis.

- Simulation results have also shown that it would be non-conservative to neglect the relaxation effect in the early age concrete stress development, since the rapid relaxation of compressive stress after the final set of concrete would cause the zero stress temperature to be higher. Ignoring the relaxation effect leads to overestimation of the cracking time and an increase of the risk of initiation of uncontrolled random cracks before the sawcut implementation, especially for JPCP.

- The simulation results have revealed that the contribution of the shrinkage on the stress development at the early age is minimal while the effect of the temperature is substantial. This is attributed to the moisture retention effect of the adequately applied curing methods (plastic sheet, curing compound). The effect of autogenous shrinkage is also negligible because the water to cement ratio of the commonly used paving concrete mixtures are normally between 0.40 and 0.43. It indicates that the zero stress temperature and the potential cracking time for concrete pavements can be accurately determined only considering the temperature variations when adequate curing methods are implemented.

- The analysis has shown that the Eurocode 2 shrinkage model, which provides the average shrinkage of concrete elements, is unable to quantify the effect of the nonlinear shrinkage on the early age stress development. The Eurocode 2 shrinkage model gives an underestimation of the shrinkage at the top of the slab.

- A parametric sensitivity analysis has shown that the early age concrete temperature (built in-curing, peak temperature) and stress development
(zero-stress temperature and potential cracking time) are closely related with the typical design, construction and climatic conditions of CRCP in Belgium: such as time on the day of the concrete placement, construction season, plastic sheet curing, base temperature, concrete placement temperature, wind speed, cloud cover. Numerical simulation results indicate that the use of plastic sheet curing is feasible for most of the Belgium conditions except construction on a warm and sunny summer day. Caution is necessary under those conditions.

- Concrete placed under warmer weather conditions develops much higher peak temperature and zero stress temperature, and a larger built-in temperature gradient than concrete placed under cold weather conditions. Moreover, concrete placed at different times of a day has a different temperature history, and thus results in different stress and strength development. Therefore, adequate scheduling of the time of concrete placement related to the weather conditions, can avoid the unfavourable too high temperature in the concrete pavement.

- The development of the early age concrete stress is inverse to the temperature, and the development of the temperature follows the daily climatic cycle. The fluctuation in the temperature and stress are higher in the slab surface. The critical tensile stresses were found at the pavement surface in the nighttime, which matched with the field surveys that all the observed early age cracks only propagated during the night.

- The zero stress temperature is not a single temperature, but varies through the depth of the slab and is largest at the pavement surface. The built-in temperature gradient is also not always a negative value, and a positive built-in temperature gradient is observed for cold weather afternoon construction. The use of a plastic sheet can also reduce the built-in temperature gradient.

- The effects of saw cut depth (in the pavement depth direction) and saw cut length (in the pavement transverse direction) on crack initiation are evaluated through the calculation of the stress intensity factors at the critical locations under various stress conditions by the weight function method. Besides considering the requirement of sufficient concrete cover and worrying about the deterioration of the unsealed notch due to traffic tire wear, it is concluded that the applied notch length of 400 mm and notch depth of 60 mm on E313 is appropriate.

- Field evidences have indicated that the particle surface notch is very effective in inducing cracks beneath the applied notches: around 20% of the notches propagated to fully developed cracks after three nights, subse-
quenty increasing to approximately 60% after 2 months, and about 70% after 1.5 years.

- The saw cut depth and saw cut timing influence the effectiveness of the crack induction in the proposed active crack control method. A larger saw cut depth and earlier saw cutting were found to be beneficial to induce cracks at the notches.
Characterization of Early Age Crack Pattern of CRCP

Following construction of a CRCP, temperature and moisture fluctuations induce volume changes in the concrete that are restrained to some degree, both externally by base friction and adjoining members and internally by continuous steel reinforcement bars and even from the differential strains of concrete itself. Thus, the restrained deformations induce compressive and/or tensile stress in the CRCP. Whenever the tensile stresses exceed the tensile strength of the concrete, transverse cracks will form to relieve stresses. Subsequent drops in temperature and the effect of ongoing shrinkage in the concrete tend to reduce the transverse crack spacing further. Moreover, externally induced stresses due to wheel loads could further reduce the crack spacing over time, but at a much slower rate (Suh et al. 1992; Zollinger et al. 1998; ARA Inc. 2003a). Overall, it is generally observed and well accepted that the crack spacing decreases rapidly during the early age of the pavement, up to 1 to 2 years. After this stage, the transverse cracking pattern remains relatively constant until the pavement slab reaches the end of its fatigue life (McCullough and Dossey 1999).

It is generally accepted that the long-term performance of a CRCP is largely determined by its early age behaviour. In the field of concrete materials, the early age is generally defined as the first 28 days of age. In the well-known report of computer-based concrete pavement behaviour (Suh et al. 1992), the early age is understood for the first 72 hours after pavement placement, while the early life is considered up to 1 year. In this study, the early age behaviour is considered in the period that the crack pattern stabilises which is the first 1 to 2 years. The early age CRCP performance indicators include crack spacing and crack width, which affect the CRCP integrity in the long term. Compared to other pavement types, there are
only a few major CRCP distresses that necessitate rehabilitation (Nam 2005). They are punchouts, spalling, and roughness. Among these, spalling and roughness are rarely reported in the CRCP performance in Belgium (Verhoeven 1993; Rens and Beeldens 2010). In this Chapter, firstly, existing crack spacing and crack width prediction models are reviewed. After that, the crack width and crack spacing data from two recently constructed CRCP sections in Belgium were collected and analysed for the effect of longitudinal reinforcement percentage and active crack control method on the crack pattern of CRCP.

### 7.1 CRACK WIDTH AND CRACK SPACING PREDICTION

Different models have been proposed to predict the crack width in CRCP (Vetter 1933; Reis et al. 1965; Palmer et al. 1988; Sato et al. 1989; Won 1990; Jiménez et al. 1992; van Breugel et al. 1998; ARA Inc. 2003a; Kohler and Roesler 2005). Among those, the Mechanistic-Empirical Pavement Design Guide (MEPDG) is a pavement design tool that are current widely used in United States (ARA Inc. 2004). It includes a module to predict the crack spacing and crack width for CRCP. In the Dutch CRCP design method, a reinforced tension bar model developed at the Section of Concrete Structure of the Delft University of Technology (hereafter refereed as Delft Tension Bar Model) is used for the design of the longitudinal reinforcement of CRCP. In the present study, the MEPDG and Delft Tension Bar model are briefly discussed.

#### 7.1.1 MEPDG

The CRCP crack spacing and crack width prediction module in MEPDG is based on the work of Reis et al. (1965) on axially reinforced tensile members (ARA Inc. 2003a). The main assumption of Reis’ work is that the concrete tensile stress due to the restraint of reinforcement steel is uniformly distributed across the cross section of the pavement slab. Selezneva et al. (2003) presented a mean crack spacing formula in the MEPDG for the final constant transverse crack spacing pattern according to Reis’ work. It is derived by static equilibrium of the forces due to volumetric change in the concrete, because of the restraint stress through the bond to the reinforcement and the base friction. The MEPDG crack spacing formula is presented as follows:

\[
\tilde{L} = \frac{f_{t28} - C_{curtting} \sigma_0 (1 - \frac{2 \zeta}{\mu_{PCC}})}{F + \frac{U_m P_s}{c_1 d_s}} 
\]

Where,
- \( \tilde{L} \) = predicted mean crack spacing, [mm];
- \( f_{t28} \) = concrete tensile strength at 28 days, [MPa];
With respect to the crack width, the MEPDG module defines an average crack width at the location of the reinforcement. It first estimates the free concrete tensile strain due to the shrinkage and the temperature drop, and then subtracts the concrete tensile strain due to the longitudinal concrete stress induced by the restraint with reinforcement and friction with base. The crack width is finally multiplied by the slab length between two adjacent transverse cracks. The generalized crack width prediction formulation of CRCP in MEPDG is presented below, in which the subscript ‘i’ indicates the variables that incrementally change per month, while the subscript ‘m’ is representing the variables that change cyclically during the year, regardless of age (Kohler and Roesler 2006).

\[
cw_i = C_{cw} \overline{L} [\varepsilon_{shri} + \alpha_c (T_{zs} - T_{\zeta_m}) - \frac{c_{2i} f_{ot}}{E_{ci}}] \cdot 1000
\]  

(7.2)

Where,

- \(cw_i\) = average crack width at the depth of steel, [mm];
- \(C_{cw}\) = local calibration constant, \(C_w = 1\) is used for global calibration, [-];
- \(\overline{L}\) = predicted mean crack spacing, [m];
- \(\varepsilon_{shri}\) = unrestrained concrete shrinkage at the depth of the reinforcement, [mm/mm];
- \(\alpha_c\) = coefficient of thermal expansion of concrete, [1/°C];
- \(T_{zs}\) = the zero stress concrete temperature, [°C];
- \(T_{\zeta_m}\) = concrete temperature at the depth of the reinforcement, [°C];
- \(c_{2i}\) = the second bond stress coefficient, [-];
- \(f_{ot}\) = maximum longitudinal concrete tensile stress at the steel level, [MPa];
- \(E_{ci}\) = elastic modulus of concrete, [MPa].

The detailed description and determination of the relevant variables in Equation (7.1) and Equation (7.2) are left out for brevity and can be found in the final draft of the MEPDG documentations (ARA 2004).

As stated above, the CRCP crack width prediction model in MEPDG defines a monthly mean crack width at the location of the reinforcement. Many field investigations have revealed through drilled cores that the crack width varies...
along the depth of the pavement slab. Wider crack openings are found at the pavement surface and the crack becomes narrow or even invisible at the bottom of the slab, mainly due to the larger drying shrinkage and temperature contraction at the upper part of the pavement slab. In order to compare the crack width from various sections and projects, a standard crack width definition at a fixed depth is required. In addition, the temperature condition including the average temperature and the temperature differential has to be defined. Kohler and Roesler (2006) proposed a refined crack width equation to calculate the crack width at any depth based on the MEPDG model. The main assumption of this refined equation is that both the concrete tensile stresses caused by the steel restraint and the base frictional stresses are treated as uniform through the pavement depth, as follows:

\[
cw_i(z) = C_{cw} \bar{L} \left[ (\varepsilon_{shr_i}(z) + \alpha_C (T_{zs}(z) - T(z)) \right.
\]

\[
- \frac{c_{2t}}{E_{ci}} \left( \frac{L \mu_P \bar{P}}{c_{1t}} + C_i \sigma_0 (1 - \frac{2z}{h_{PC}}) + \frac{\bar{L}}{2} F \right) \right] \cdot 1000 \quad (7.3)
\]

Where \( z \), [mm], is the depth along the pavement slab, and \( z = 0 \) is at the pavement surface.

From a practical standpoint, the crack width at the pavement surface is the most convenient location for measurement. As shown in the refined crack width Equation (7.3), the closer the reinforcement to the pavement surface, the narrower the crack width at the pavement surface. This explains the current common construction practice of placing the longitudinal reinforcement above the mid-depth of the slab to control the crack width at the surface. When the shrinkage at the surface and the depth of the steel reinforcement bars are known, the crack width at the pavement surface obtained from field measurements can be easily shifted to the steel depth for theoretical analyses.

Considering the effect of temperature conditions on crack width, Kohler and Roesler (2005) adjusted the crack width to the standard temperature condition, 0°C temperature at the depth of the steel and zero temperature differences.

### 7.1.2 Delft Tension Bar Model

In the Dutch design method a model for the design of the longitudinal reinforcement of CRCP is included based on reinforced concrete tensile members (Noakowski 1985; van Breugel 1991; van Breugel et al. 1998). Noakowski presented an analytical solution for both the transfer length \( l_t \) and the mean crack width \( cw_{mo} \) for a not fully developed crack pattern. This model is presented here briefly. As shown in Figure 7.1 stage I is the uncracked phase in which the member behaves linear elastically defined by the stiffness \( EA_{cs} \). If the cracking force \( N_{cr} \) or the crack strain \( \varepsilon_{cr} \) is reached, the first crack is formed, and then the tensile
member is in stage II of the not fully developed crack pattern. An imposed deformation can increase, which results in the gradual formation of new cracks. It is assumed that all cracks are formed at the same cracking force. Stage II ends when the crack pattern has become fully developed, the mean steel strain is $\varepsilon_{f_{dc}}$. The tensile member is now in stage III, an increase of the load or imposed deformation results in the widening of already existing cracks, and no new cracks are formed. If the steel stress $\sigma_s$ in a crack reaches the yields stress of the steel, an uncontrolled widening of the existing crack occurs. It is called stage IV, and the crack width models are no longer valid.

![Figure 7.1 Force-strain relationships for a reinforced concrete tensile member (Noakowski 1985).](image)

The mean crack width $c_{w_{mo}}$ in the not fully developed crack pattern, stage II, is described as follows (Noakowski 1985; van Breugel et al. 1998):

$$c_{w_{mo}} = 2 \left[ \frac{0.4d_s}{f_{cc,m,o}E_s} \sigma_{s,cr} (\sigma_{s,cr} - n\sigma_{cr}) \right]^{0.85} \quad (7.4)$$

Where,

- $c_{w_{mo}}$ = average crack width in the uncompleted crack pattern, [mm];
- $E_s$ = elastic modulus of longitudinal reinforcement bars, [MPa];
- $E_c$ = elastic modulus of concrete at 28 days, [MPa];
- $n = \frac{E_s}{E_c}$, the ratio of Young’s modulus of reinforcement bar and concrete;
- $d_s$ = diameter of longitudinal reinforcement bar, [mm];
- $\sigma_{cr}$ = concrete tensile stress just before cracking, [MPa]
\[
\sigma_{s,cr} = \text{longitudinal reinforcement steel stress just before cracking, [MPa];}
\]

\[
f_{cc,m,0} = \text{mean cube concrete compressive strength at 28 days for loading of short duration, [MPa].}
\]

The tensile stress \(\sigma_{cr}\) in the concrete slab just before the moment of cracking is taken as 60% of the mean tensile strength after 28 days for loading of short duration to take into account that cracking starts well within 28 days after construction of the concrete layer.

\[
\sigma_{cr} = 0.6 f_{ct,m,0} = 0.54\left[1.05 + 0.05(f_{cc,k,0} + 8)\right] \tag{7.5}
\]

Where,

\[
f_{cc,k,0} = \text{characteristic cube concrete compressive strength at 28 days for loading of short duration, [MPa];}
\]

\[
f_{ct,m,0} = \text{mean concrete tensile strength at 28 days for loading of short duration, [MPa].}
\]

Considering the balance of the horizontal force and the bending moments, the reinforcement steel stress \(\sigma_{s,cr}\) in the crack just after the moment of cracking is equal to:

\[
\sigma_{s,cr} = \sigma_{cr} \frac{h_{PCC}(1 + n P_s)}{2(h_{PCC} - \zeta) P_s} \tag{7.6}
\]

Where,

\[
\zeta = \text{concrete cover, the depth of longitudinal reinforcement bar below the pavement surface, [mm];}
\]

\[
P_s = \text{percentage of longitudinal reinforcement;}
\]

\[
h_{PCC} = \text{thickness of the concrete pavement slab, [mm].}
\]

Considering the serviceability limit state and taking into account long term and repeated loading, the maximum crack width \(c_{w_{max}}\) is equal to:

\[
c_{w_{max}} = \gamma_{so} \gamma_{\infty} c_{w_{mo}} \tag{7.7}
\]

Where,

\[
\gamma_{so} = \text{factor to consider the variation of the crack width;}
\]

in case of the uncompleted crack pattern, \(\gamma_{so} = 1.3\);

\[
\gamma_{\infty} = \text{factor to consider longing or long duration or repeated loading;}
\]

when steel stress less than 295 MPa, \(\gamma_{\infty} = 1.3\).

The longitudinal reinforcement has to be designed such that the maximum crack width \(c_{w_{max}}\) should not exceed the allowable crack width that is according to the environmental classes of the concrete structures. According to the NEN-EN 206-1 (2001) and Eurocode 2 (2004), the following environmental classes are valid for CRCP: XC4 (cyclic wet and dry), XD3 and XF4 (high water saturation with de-
An allowable crack width of 0.4 mm for CRCP is obtained in Europe for the above-mentioned environmental classes. The longitudinal reinforcement has to be designed such that the allowable crack width is not exceeded.

It should be mentioned that in case of a not fully developed crack pattern, there is strictly speaking no mean crack spacing. However, the mean crack spacing can be determined in the fully developed crack pattern through the transfer length \( l_t \). When the concrete cracks, the concrete tensile force must carried by the reinforcing steel. At a certain distance from a crack, the so-called transfer length \( l_t \), the undisturbed situation is again reached (Braam and Frenay 2004). Noakowski (1985) assumed that the crack pattern is fully developed when all the crack spacings vary between \( l_t \) and \( 2l_t \). Thus, the mean crack spacing \( \bar{L} \) is calculated as follows:

\[
l_t = 1.2cw_{m0} \frac{E_s}{\sigma_{s,cr}}
\]

\[
\bar{L} = 1.5l_t = 1.8cw_{m0} \frac{E_s}{\sigma_{s,cr}}
\]

In the fully developed crack pattern, the mean crack width is:

\[
cw_{mv} = \frac{\bar{L}}{E_s} \left( \sigma_s - 0.5\sigma_{s,cr} \right)
\]

**Discussion**

A simple example of E17 is used to show the ability of the above-mentioned two models to predict the mean crack spacing and crack width of CRCP. The required input parameters are summarized in Table 7.1. The calculated mean crack spacing and maximum crack width in the cold season are presented in Figure 7.2.
Compared to the field measured values, the MEPDG model gives a reasonable prediction value for both mean crack spacing and crack width, while the Delft Tension Bar model predicts too low mean crack spacing. The Delft Tensile Bar model only provides a way to determine the needed amount of reinforcement based on the allowable crack width. It cannot predict the mean crack spacing for the not fully developed crack pattern. Delft Tensile Bar model takes into account the influences of the longitudinal reinforcement (percentage, concrete cover, and reinforcement bar diameter), the pavement slab thickness, and the concrete properties at 28 days on the crack pattern of CRCP. However, it does not consider the factors of temperature condition (such as air temperature during construction, heat of cement hydration, time of construction during a day and a year), shrinkage of the concrete, and the coefficient of thermal expansion of concrete. These above-mentioned neglected factors have proven to have significant effects on the cracking behaviour of CRCP, as shown in the predicted results by MEPDG (see Figure 7.3). Besides, Figure 7.4 illustrates the estimated crack width by those two models for the case of E17 with a zero-stress temperature of 30 °C. It shows that the estimated crack width by MEPDG is larger and more sensitive to the reinforcement ratio than that of the Delft Tension bar model.

Figure 7.3 Influences of zero-stress temperature and concrete cover on the estimated crack pattern by MEPDG model.
Figure 7.4 Comparison crack width prediction models in MEPDG and Delft Tension bar model at the case of E17 with $T_{zs}=30$ °C.

Table 7.1 General input parameters in MEPDG crack spacing and crack width prediction model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Value</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCC 28 day compressive strength</td>
<td>$f'_{c28}$</td>
<td>MPa</td>
<td>50</td>
<td>Measured in cylinders 100*200 mm</td>
</tr>
<tr>
<td>PCC elastic modulus</td>
<td>E$_{PCC}$</td>
<td>MPa</td>
<td>4.0e4</td>
<td>Measured in cylinders 100*200 mm</td>
</tr>
<tr>
<td>PCC 28 day tensile strength</td>
<td>$f_t$</td>
<td>MPa</td>
<td>4.0</td>
<td>Measured in cylinders 100*200 mm</td>
</tr>
<tr>
<td>PCC coefficient of thermal expansion</td>
<td>$\alpha_{PCC}$</td>
<td>1/°C</td>
<td>10.6e-6</td>
<td>Estimated by Eurocode2</td>
</tr>
<tr>
<td>PCC Poisson's ratio</td>
<td>$\mu_{PCC}$</td>
<td>–</td>
<td>0.15</td>
<td>MEPDG documentation</td>
</tr>
<tr>
<td>Ultimate shrinkage</td>
<td>$\varepsilon_\infty$</td>
<td>–</td>
<td>5.67e-4</td>
<td>Measured in prisms 100*400 mm</td>
</tr>
<tr>
<td>Cement content</td>
<td>CC</td>
<td>kg/m$^3$</td>
<td>400</td>
<td>Concrete mixture design in E313</td>
</tr>
<tr>
<td>Thermal diffusivity</td>
<td>$\Upsilon_{PCC}$</td>
<td>m$^2$/s</td>
<td>1.3e-6</td>
<td>MEPDG documentation</td>
</tr>
<tr>
<td>Slab thickness</td>
<td>$h_{pcc}$</td>
<td>mm</td>
<td>250</td>
<td>Pavement design</td>
</tr>
<tr>
<td>Depth to steel</td>
<td>$\zeta$</td>
<td>mm</td>
<td>90</td>
<td>Pavement design</td>
</tr>
<tr>
<td>Steel bar diameter</td>
<td>$d_b$</td>
<td>mm</td>
<td>20</td>
<td>Pavement design</td>
</tr>
<tr>
<td>Slab/Base friction coefficient</td>
<td>$F$</td>
<td>MPa/mm</td>
<td>2.0e-3</td>
<td>MEPDG documentation</td>
</tr>
<tr>
<td>Modulus of subgrade reaction for curling</td>
<td>$k$</td>
<td>MPa/mm</td>
<td>0.109</td>
<td>Pavement design</td>
</tr>
<tr>
<td>Average seasonal ambient temperature for the season of construction</td>
<td>$T_{air}$</td>
<td>°C</td>
<td>15</td>
<td>Weather records</td>
</tr>
<tr>
<td>Minimum average seasonal ambient temperature</td>
<td>$T_{min}$</td>
<td>°C</td>
<td>1.7</td>
<td>Weather records</td>
</tr>
<tr>
<td>Minimum average seasonal temperature at the depth of steel</td>
<td>$T_{steel,min}$</td>
<td>°C</td>
<td>3.6</td>
<td>Calculated by MEPDG model</td>
</tr>
<tr>
<td>Relative humidity in the concrete at the depth of steel</td>
<td>$r_{hPCC,\zeta}$</td>
<td>%</td>
<td>85</td>
<td>MEPDG documentation</td>
</tr>
</tbody>
</table>
7.2 CRACK PATTERN SURVEY METHOD

7.2.1 Crack Spacing Survey Method

Manual crack spacing surveys have been conducted on E17 (near De Pinte), and on E313 (near Herentals) test sections periodically, as shown in Figure 7.5, although the manual surveys are laborious and risky for the personnel, due to the traffic hazards. Lane closures at E17 and E313 are not allowed due to high traffic intensity. Therefore, the crack spacing survey was conducted by slowly walking along the edge of the pavement (emergency lane), record the location and the shape of cracks, and define the category of each crack. It should be noted that for both test sections all crack spacing surveys were performed on the emergency lane mainly because of safety reasons and based on the consideration of hindering the traffic as less as possible. The emergency lane had exactly the same structure as the traffic lanes. After the road was opened to traffic, traffic control and safety trucks were provided by the local road authorities.

Figure 7.5 Manual crack pattern surveys, on E17 (left) and on E313 (right) with green markers.

In the De Pinte Test section, the length of each section for the crack spacing survey is 500 m, 1000 m and 500 m, respectively. The percentage of reinforcement in these test sections is 0.75%, 0.70%, and 0.65% (plus 20 kg/m³ steel fibers), respectively. More detailed crack surveys were done in a 100 m long subsection of each section. The location of each crack was measured by a distance-measuring wheel with a resolution of 0.05 m, putting a marker every 5 m interval, and on the pavement surface at the starting and ending points for each subsection. One problem with regard to the recorded location of cracks by distance measuring wheel is that it lacks repeatability resulting in inconsistencies in different crack spacing surveys due to the non-straight path of the wheel in practice. Therefore, the crack spacing data of different surveys were corrected according to the location marker on the pavement surface.

In the active crack control test section of Herentals, a 500 m long section with 30 mm saw cut and 1100 m long section with 60 mm saw cut were selected for the
The method of crack spacing surveys was the same as that of the test section in De Pinte except the crack location measurement. The 1.2 m interval surface notches were used as the location indicators and quite consistent crack location data were obtained.

<table>
<thead>
<tr>
<th>Road</th>
<th>Time</th>
<th>Crack Pattern Survey</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Date</td>
<td>1st</td>
</tr>
<tr>
<td>E17</td>
<td>Date</td>
<td>18-22/08/11</td>
</tr>
<tr>
<td></td>
<td>Age (day)</td>
<td>1-5</td>
</tr>
<tr>
<td>E313 60mm</td>
<td>Date</td>
<td>11-15/09/12</td>
</tr>
<tr>
<td></td>
<td>Age (day)</td>
<td>1-5</td>
</tr>
<tr>
<td>E313 30mm</td>
<td>Date</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Age (day)</td>
<td>–</td>
</tr>
</tbody>
</table>

Periodically crack pattern surveys have been performed during the first 4 days after construction and furthermore on in-service roads about 2 months, 7 months, 1 year, and 2 years after construction, respectively, as summarized in Table 7.2.

### 7.2.2 Crack Width Survey Method

Despite the fact that the crack width is widely recognized as a vital variable influencing the performance of CRCP, the value of most available crack width data is limited by lacking a clear explanation of how the measurements are done, the location of the crack width measurement and in what temperature conditions the pavement was. As discussed previously, most of the crack width prediction models for reinforced tensile members define a crack width at the location of the reinforcement, i.e. the well-known CRCP crack width prediction model in MEPDG. However, a direct way to measure this value in the field is still lacking, especially when the pavement is in service. On the contrary, nearly all the previous investigators measured the crack width on the pavement surface because it is the most convenient location. In order to evaluate the effects of several changes in Belgium design concepts (such as steel fiber, the percentage of longitudinal reinforcement) on the crack width, a reliable and precise crack width measurement method is fundamental and necessary. In this study, different devices have been adopted to evaluate the crack width on the pavement surface. It includes the optical microscope, digital microscope, and image analysis, as shown in Figure 7.6.

Limited information is known about the variability of the width of a single crack. An analysis of crack widths measured over and between the longitudinal reinforcement along the transverse direction had shown that there was no statistically significant difference between both mean crack width and an individual crack (Braam and Frenay 2004). Therefore, for each measured transverse crack, the width was measured at five locations, i.e. at a distance of 100, 200, 300, 400, and 500 mm from the edge of the pavement.
Chapter 7 Characterization of Early Age Crack Pattern of CRCP

Figure 7.6 Field crack width measurements on the pavement surface.

Measured Crack Width using Microscope

Many investigators have used an optical microscope or comparator either directly on the surface or at several millimetres below the surface to measure the crack width. Initially, a Peak Type 2054-40 hand microscope was used for evaluating the crack width on the pavement surface. It has a resolution of 0.02 mm and a range of 2.0 mm. However, there are many doubts about the correctness and reliability of the crack width measurements on the pavement surface after the field measurement experiences. Firstly, the resolution of this manual crack width inspection by microscope and comparator is limited to 0.02 and 0.05 mm, respectively, which is not sufficient to capture the change of crack width due to the varying design and construction features, and fluctuating environmental conditions. Moreover, the most unfavourable drawback of this method is the poor repeatability between different surveys, even worse between different investigators because of the difficult to define edge of the crack. Figure 7.7 shows a typical crack at the pavement surface. An arbitrarily defined crack edge will lead to an error of more than 0.2 mm, which is nearly the same as the absolute value of the crack width on the surface.
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Figure 7.7 Image obtained by digital microscope (magnification 200X).

**Measured Crack Width using Digital Microscope**

Due to the poor repeatability and limited accuracy of crack width measurement by the optical Microscope as stated above, a digital microscope, Dino-Lite Pro AM-413FVT, is used for the subsequent field surveys. This device has a maximum magnification of 200X and stores the crack pictures at a resolution of 1.3 Mb. With the obtained digital pictures, the crack width on the pavement surface can be evaluated in two ways. The first method determines the crack width by the user-defined lines within the image using the built-in software of this digital microscope. Using a powerful zoom function, the crack width can be determined accurately. A mean crack width is obtained by measuring several locations in a crack, as shown in Figure 7.8. The resolution of each pixel is 0.006 mm and the corresponding resolution for this crack width measurement method is about 0.01 mm.

Figure 7.8 Crack width analysis procedures by measuring user defined lines using the built-in software.
The second method uses the automatic image analysis. This automatic method involves two steps, crack detection and crack analysis. Figure 7.9 shows the procedure of the proposed crack width measurement by image analysis. In the step of crack detection, thresholding is the most critical operation. In general, the larger the contrast in intensity between the crack and the background, the easier the thresholding operation is. It should be noted that the background brightness of the obtained image is generally not uniform because of the quality of light on the concrete surface and too much noise on the surface because of the structure’s exposure to the environment. Therefore, a global image threshold algorithm proposed by Otsu (1975) is used to compute a global threshold value to convert a grayscale picture to a binary image, as shown in Figure 7.9. The noise in the binary image is removed subsequently through the morphological structuring treatment. Then, the edge of the crack on the pavement surface is obtained by Canny method (Canny 1986). Eventually, the crack width is obtained by dividing the area between the boundaries of the crack to the length of the centre line. A MATLAB code was developed for the proposed algorithm.

![Figure 7.9 Crack width analysis procedures by image analysis](image_url)
The resolution of the latter crack width measurement is the same as the user-defined lines measurement method, which depends on the real size of one pixel of the image. The latter method is however preferred as it provides a more reliable and repeatable crack width in statistical sense and it is user friendly and suitable for large-scale measurements in practice.

The quantitative results of the above mentioned crack width measurements on the CRCP surface are given in Table 7.3. The crack width measurements based on image analysis yield results comparable with those measured by traditional optical microscope measurement and the user-defined lines measurement using digital microscope. The two methods by digital microscope also have a better accuracy than that of the optical microscope due to the higher magnification capacity of the digital microscope. However, the accuracy of the proposed algorithm is limited by the background of the cracks. It is suitable for the early age cracks that have a distinct crack edge before the road is opened. For the in-service pavement, the crack edge may be ground off by the traffic tire and filled up with dust, which makes the edge of crack difficult to be detected. A more advanced crack width algorithm through image analysis is required for in-service CRCPs in further studies.

Table 7.3 Comparison of field crack width measurement methods

<table>
<thead>
<tr>
<th>Crack width measurement method</th>
<th>Crack 1</th>
<th>Crack 2</th>
<th>Crack 3</th>
<th>Crack 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optical microscope</td>
<td>0.38</td>
<td>0.33</td>
<td>0.28</td>
<td>0.27</td>
</tr>
<tr>
<td>Digital microscope</td>
<td>0.399</td>
<td>0.279</td>
<td>0.279</td>
<td>0.276</td>
</tr>
<tr>
<td>Image analysis</td>
<td>0.342</td>
<td>0.292</td>
<td>0.264</td>
<td>0.272</td>
</tr>
</tbody>
</table>

7.2.3 Crack Width Change

In order to obtain the horizontal crack width variation due to temperature variation, a procedure was adopted to measure the horizontal crack width change by a LVDT. The measurement concerns the distance between two fixed points, as shown in Figure 7.10.

Two sets of studs were attached to both sides of the selected cracks within the subsections on the edge of the pavement. The distance between two studs is 100 mm, the upper set of studs is 30 mm below the pavement surface, and the bottom set of studs is at the depth of the longitudinal steel reinforcement. An external LVDT was used to measure the horizontal crack width changes at each selected crack one by one. The used LVDT is Sylvac 905-1301 having a range of 12.5 mm and a resolution of 0.001 mm. The crack width change is calculated as the difference between two measurements at different temperature and age conditions. This method thus provides only information on the changes in crack width due to environmental effects. The absolute crack width cannot be obtained because the initial crack width was not measured as the fixed points could only be installed
after the crack occurred. These measurements could only be done a few times during the construction period because all the studs were buried into a gutter that was constructed alongside the emergency lane.

Figure 7.10 Crack width change measurements by LVDT.

7.3 CRACK SPACING RESULTS

Crack spacing data partly come from a few previous studies and the majority comes from the periodical field investigations at the recently constructed CRCP sections on E17 near De Pinte and on E313 near Herentals in Belgium since 2011. It should also be mentioned that a stable crack spacing pattern requires a complete environmental cycle that may be 1 to 2 years after construction. The CRCP sections on E17 were constructed in summer (August 2011) and they experienced cold winters in 2012 and 2013. The 30 mm and 60 mm notch depth sections on E313 were constructed in summer (July 2012) and fall (September 2012), respectively, and they thus experienced the cold winter of 2013. Therefore, it may be reasonable to assume that the short period crack spacing data will represent the crack behaviour at long term.

7.3.1 Average Crack Spacing

Table 7.4 and Table 7.5 summarize the statistics of the field-surveyed crack spacing data on E17 and E313, respectively. It shows that the number of cracks increases rapidly during the first year for both active and non-active crack control sections. After about 2 winters, it is found that the crack pattern became stabilized for all sections since very few new cracks occurred. The average crack spacing of the convectional CRCP on E17 under the current design concept ranges from 1.00 m to 1.27 m at the age of after 2 winters. For the active crack control sections on E313, the mean crack spacings ranged from 1.00 and 1.35 m for the 30 and 60 mm notch depth sections, respectively, after 2 winters.
Table 7.4 Crack spacing statistic of each field surveys on E17 (non-active crack control).

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Percentage of reinforcement</th>
<th>Length (m)</th>
<th>Number of cracks</th>
<th>Crack spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>4</td>
<td>0.75%</td>
<td>500</td>
<td>88</td>
<td>5.64</td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>1000</td>
<td>260</td>
<td>3.86</td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>500</td>
<td>150</td>
<td>3.30</td>
</tr>
<tr>
<td>60</td>
<td>0.75%</td>
<td>500</td>
<td>259</td>
<td>1.92</td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>1000</td>
<td>314</td>
<td>1.58</td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>500</td>
<td>358</td>
<td>1.39</td>
</tr>
<tr>
<td>223</td>
<td>0.75%</td>
<td>500</td>
<td>388</td>
<td>1.29</td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>1000</td>
<td>734</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>500</td>
<td>476</td>
<td>1.05</td>
</tr>
<tr>
<td>370</td>
<td>0.75%</td>
<td>500</td>
<td>425</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>1000</td>
<td>767</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>500</td>
<td>491</td>
<td>1.05</td>
</tr>
<tr>
<td>587</td>
<td>0.75%</td>
<td>500</td>
<td>431</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>1000</td>
<td>792</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>500</td>
<td>499</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note: * there is 20 kg/m³ steel fibers in the 0.65% section.

Table 7.5 Crack spacing statistic of each field surveys on E313 (active crack control).

<table>
<thead>
<tr>
<th>Saw cut depth</th>
<th>Age (days)</th>
<th>Length (m)</th>
<th>Number of cracks</th>
<th>Crack spacing (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>30 mm</td>
<td>123</td>
<td>500</td>
<td>416</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td>262</td>
<td>500</td>
<td>496</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td>436</td>
<td>500</td>
<td>501</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>613</td>
<td>500</td>
<td>504</td>
<td>1.00</td>
</tr>
<tr>
<td>60 mm</td>
<td>4</td>
<td>1080</td>
<td>191</td>
<td>5.67</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>1080</td>
<td>665</td>
<td>1.63</td>
</tr>
<tr>
<td></td>
<td>204</td>
<td>1080</td>
<td>761</td>
<td>1.42</td>
</tr>
<tr>
<td></td>
<td>378</td>
<td>1080</td>
<td>774</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td>555</td>
<td>1080</td>
<td>802</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Table 7.4 and Table 7.5 indicate that the average crack spacing with the current design concept is better than the design concept 1 where the average crack spacing
varies from 0.4 m to 0.6 m (as shown in Table 2.2). However, it is commonly accepted nowadays that the shorter average crack spacing is not the only parameter that determines the behaviour of CRCP. It is just one of the many elements playing a role in the creation of punchout. The new CRCP design procedure described in MEPDG does not provide recommendations on the control of minimum crack spacing due to the numerous factors that affect this variable including the reinforcement percentage. However, a maximum average crack spacing of 1.8 m is recommended. In addition, the characteristics of crack spacing of current design concept in Belgium may be quite different from that of most United States CRC pavements. It should be noted that the cross-section of CRCP is a bit different from that of most United States CRCP’s in that the PCC base has the same thickness as CRCP. Most United States CRCP’s are layered on a 100 mm base (lean concrete or asphalt) on aggregate or treated sub-base on natural subgrade. Belgium has a maritime climate, with usually mild winters, and cool summers, which is not as severe as that in some states in United States. Figure 7.11 shows the crack maps for three subsections over time at De Pinte on E17, which is useful to visualize the distribution of the crack patterns. The crack map clearly shows the different crack development and the randomly occurring crack pattern for varying percentage of reinforcement of the conventional CRCP test sections.

Figure 7.11 Crack maps for test sections on E17 at De Pinte: (a) 0.75%, (b) 0.70%, and (c) 0.65% and steel fiber.
Chapter 7 Characterization of Early Age Crack Pattern of CRCP

Effect of Percentage of Longitudinal Reinforcement

It is a generally accepted theory among researchers that the higher the percentage of longitudinal reinforcement in CRCP, the larger the number of cracks, a result of the bond between concrete and steel reinforcement restricting concrete movements. However, the average crack spacing of the three test sections with different reinforcement on E17 at De Pinte is not as expected and is quite contrary to the generally accepted theory in the very early ages, as can be seen in Figure 7.12. At the very early age of the pavement, the section with 0.75% longitudinal reinforcement had the largest average crack spacing while the section with 0.65% plus steel fiber had the smallest average crack spacing. It seems that the longitudinal reinforcement percentage has little influence on the crack spacing in the very early age. However, as time increased, the growth rate of the number of cracks in the 0.75% section is faster than in the other two sections. It indicates the effect of longitudinal reinforcement on the average crack spacing in later pavement ages. The possible explanation is that the bond between the reinforcement and concrete is not yet fully developed during the very early age, and thus it is difficult to control the meandering early age cracks only through a reinforcement design.

![Figure 7.12 Crack spacing development with time at E17, De Pinte.](image)

Effect of Placement Time on the Day and Curing time

Figure 7.13 displays the development of the number of transverse cracks in three 100 m long subsections, surveyed in detail, with different longitudinal percentage of reinforcement on E17 during the first few days after concrete placement. In subsection ① no cracks were observed during the night following the construction day even though the concrete temperature drop was up to 10°C. Then 12 cracks were observed after the second night after construction. During the subsequent
two nights, only one new crack occurred within the 100 m subsection. The subsections ② and ③ were constructed at 2 am and 12 am, on 19th August 2011, respectively. The initial curing time for those two subsections was 13 hours and 8 hours, respectively. After the night following the construction day, 29 cracks were found in subsection ② and 12 cracks in subsection ③. After the subsequent two nights, 3 and 4 new cracks were observed in the second subsection and 7 and 8 new cracks occurred in the third subsection.

Figure 7.13 Crack pattern development of three 100 m subsections during the first week after concrete placement, E17, Ghent, August 2011: km 45.0-45.1 with 0.75% longitudinal reinforcement hereafter is defined as subsection ①, km 46.0-46.1 with 0.70% is defined as subsection ②, and km 46.4-46.5 with 0.65% is defined as subsection ③, respectively.

Figure 7.14 illustrates the development of the number of cracks in all 100 m long subsections on E17 at an age of 4 days, 60 days, and 979 days, respectively. In the early age, at 4 days, the time of placement and plastic sheet curing show a high correlation with the number of early age cracks. In the sections having the same percentage of longitudinal reinforcement of 0.70%, it shows that the morning-constructed sections have a larger number of cracks than those constructed in the afternoon. For instance, 48 cracks were observed at the age of 4 days for subsection km 46.2-46.3 that was constructed at 3 am of August 19, 2011 while only 17 cracks were found on section km 45.3-45.4, which was placed at 1 pm of the August 18, 2011. As discussed in Section 6.3, the paving time on the day and the plastic sheet curing have a substantial effect on the temperature and the stress development under warm temperature construction. It is difficult to isolate the effect of the time of placement on the day from the effect of duration of plastic sheet curing even for the sections having the same percentage of longitudinal reinforcement, since both
of them are different for each subsection. The peak slab temperature and zero-stress temperature are predicted by the proposed temperature prediction model in Chapter 4 and the early age stress prediction model in Chapter 6. The placement time and plastic sheet curing duration are illustrated in Figure 7.14 and the other inputs were described in previous section 6.2.5.

![Figure 7.14](image-url)

(a) Time of placement on the day

(b) Plastic sheet curing

Figure 7.14 The effect the time of placement on the day and the plastic sheet curing duration on the early age cracking.

The predicted peak slab temperature is 36.9°C, 39.5°C, and 37.0°C for the three detailed surveyed subsections ①, ②, and ③, respectively. The corresponding
estimated zero-stress temperature is 33.7°C, 36.9, and 31.9°C, respectively. It indicates that the peak slab temperature and zero-stress temperature, which depend on the construction and curing conditions, have a substantial effect on crack development at early age. It shows that the earlier the time of placement on the day, the greater the occurrence of transverse cracks due to the higher peak slab temperature. Moreover, it is found that the appropriate duration of plastic sheet curing is beneficial to prevent substantial temperature drop and drying shrinkage during the followed night after placement, as shown on subsection ①. Lastly, it also shows that the effect of placement time and plastic sheet curing does not level out with time. The similar tendency of the number of the cracks in each section is found at the age of both 4 days and 979 days, indicating the effect of the peak slab temperature and the zero-stress temperature on the long-term crack spacing.

**Effect of Active Crack Control Method**

The average crack spacing of the 30 mm saw cut section on E313 is 1.00 m at the age of 613 days, while it is 1.35 m for the 60 mm saw cut section at the age of 555 days (Figure 7.15). In the non-active crack control section on E17, the average crack spacing is 1.18 m after 587 days, and 1.17 m after 979 days. The small average crack spacing of the 30 mm saw cut section on E313 could be attributed to the fact that initially many randomly uncontrolled cracks occurred in between notches at the early age due to too late saw cutting together with the smaller notch depths. Then many new cracks were induced at a notch due to stress concentration soon afterwards. The deeper saw cut section has a more regular crack pattern due to the earlier saw cut timing and the deeper notches that made cracks much more prone to initiate at the designated location.

![Figure 7.15 Crack spacing development with time at E313.](image-url)
**Crack Spacing Distribution**

Figure 7.16 compares the cumulative crack spacing distribution between non-active and active crack control sections. Comparisons show that the saw cut sections on E313 have a much better crack spacing distribution. The 60 mm notch section has only 14.0% of the total number of cracks spaced less than 0.6 m (closely spaced cracks) approximately 18 months after paving. Besides, 75.0% of the crack spacing falls into the desirable range, 0.6 m to 2.4 m. In contrast, the non-active crack control section on E17 has about 51.8% of total cracks less than 0.6 m and only 29.4% of cracks within the ideal range at the same age that is considered as an undesirable crack spacing distribution. Among the active crack control sections, the deeper the saw cut, the better the crack spacing distribution.

![Cumulative Crack Spacing Distribution](image)

**Figure 7.16** Comparison of cumulative crack spacing distribution.

The uniformity of the crack spacing can also be indicated by the moving average spacing of five consecutive cracks. The moving average spacing of five consecutive cracks is not only useful in identifying the locations of clustered cracks (group of cracks with average crack spacing less than 0.6 m) but also can be used to identify the extent of a pavement section that exhibits an “acceptable” crack pattern (Tayabji et al. 1998b). The acceptable values of the average spacing of five moving consecutive cracks are assumed to be between 0.9 and 1.8 m. Figure 7.17 presents the comparison of the moving average spacing of five consecutive cracks on both active crack control test sections and the non-active crack control section. It is shown that the 60 mm saw cut section has the best crack pattern, followed by the 30 mm section and finally the non-active crack control section of E17.
Figure 7.17 Average crack spacing distribution based on five consecutive cracks (a) E313, 60 mm deep saw cut; (b) E313, 30 mm deep saw cut; (c) E17, De Pinte, 0.75%.
7.3.2 Cluster Cracking

One typical crack spacing feature of CRCP under the current design concept in Belgium is the high percentage of clusters of closely spaced cracks, as shown in Figure 7.18. Clustered cracks typically act as an indicator for punch-out development. The probability of two, three, four or five consecutive cracks occurring within a range of distances can be chosen as an indicator to evaluate the evidence of cluster cracking within a particular pavement segment. Zollinger et al. (1998) developed an algorithm to calculate the probability of cluster cracking as:

\[
\text{PROB} (\text{distance between } r \text{ consecutive transverse cracks} < \text{Distance } X) = \sum \frac{\text{Number of two crack group spaced at an interval within distance } X}{\text{Total number cracks included in entire crack distribution} - (r - 1)}
\]

(7.11)

Where, \( r = 2, 3, \) or 4.

Figure 7.18 Cluster of closely spaced cracks in the passive crack control section on E17.

Table 7.6 shows the cluster cracking probability of different numbers of consecutive cracks. The probability of cluster cracking of the active crack control test sections on E313 is much lower than the passive crack control section on E17. For instance, the probability of two and three consecutive cracks with an average spacing less than 0.6 m of the section on E17 after 12 months is 50.77% and 25.84%, respectively, and subsequently increasing to 51.76% and 31.13% after 19 months. By contrast, both active crack control sections in E313 show much lower probability of two and three consecutive cracks less than 0.6 m at the same age. Among the active crack control method, the deeper 60 mm saw cut section shows a lower probability of cluster cracking compared to the 30 mm sawcut section, although they cannot directly be compared because the time of saw cutting was different.
7.3.3 Crack Shape and Crack Face

Field surveys revealed that the non-active crack control section on E17 exhibited a fair amount of meandering, divided and Y-cracks, as shown in Figure 7.19(a). In contrast, there are no undesirable cracks in the 60 mm deep active crack control test section during the first 4 days after construction, and the 193 transverse cracks are all perfectly straight as shown in Figure 7.19(b). However, the subsequent field surveys indicated that both active crack control sections exhibited few meandering cracks and occasionally divided cracks and Y-cracks, especially in the 30 mm depth saw cut section. However, field surveys also showed that the 60 mm depth saw cut section had much less meandering, divided, Y-cracks and closely spaced cracks than that of 30 mm depth saw cut section. It illustrates that a deeper saw cut is better to achieve a better crack pattern. Moreover, the shallow depth partial notch has a potential risk that eventually cracks may develop at all notches, and then the chances for more closely spaced cracks grow due to the existence of randomly occurred cracks adjacent to notches in the early age. Someone may be worried that the straight transverse crack pattern will reduce the load transfer efficiency due to the relative low aggregate interlock. It is not a concern because of the adopted two-lift construction in the active crack control test section. The upper layer has a smaller aggregate size ($d_{\text{max}} = 6.3\text{mm}$) and a rather straight crack. Field drilled cores have however indicated a good aggregate interlock in the bottom layer ($d_{\text{max}} = 31.5\text{mm}$) of the pavement slab as shown in Figure 7.20.
Figure 7.19 Crack shape (a) Natural cracks on E17 2 months after construction; (b) induced crack in the 60 mm deep saw cut section on E313 2 days after construction.

Figure 7.20 Field drilled cores in 30 mm notch depth test section on E313.

7.4 CRACK WIDTH RESULTS

Crack width has been considered one of the most important factors determining the performance of CRCP. The basic principle of CRCP is that the longitudinal reinforcement keeps cracks tight. A tight crack will maintain a good load transfer and prevent water to get into the crack, which would cause reinforcement corrosion, especially in regions where de-icing salts are used. MEPDG calculates and requires a maximum crack width of 0.5 mm at steel depth to minimize the possibility of corrosion of the reinforcement. In Europe, according to EN 1992 and EN 206, an allowable crack width of 0.4 mm for CRCP is proposed for most environments classes that the pavements are subjected to.
Chapter 7 Characterization of Early Age Crack Pattern of CRCP

7.4.1 Crack Width Profile and Shape

Cores drilled on E17 and E313 show that the cracks at the pavement surface look relatively wider than expected. However, a little bit deeper, a few millimetres beneath the pavement surface, the crack width suddenly decreases and becomes invisible above the location of the longitudinal reinforcement. The crack is probably less harmful than it looks like at the pavement surface.

Effect of Percentage of Longitudinal Reinforcement

Table 7.7 shows the crack width periodically measured by a microscope on the pavement surface in the rehabilitation project of E17 at De Pinte. Initially, the crack widths were measured by a microscope on the pavement surface where there is a large scatter of the crack width. During the subsequent surveys, the crack width was measured inside the cracks at some depth (several millimetres) below the pavement surface. Meanwhile, the concrete pavement surface temperature was also recorded. All cracks within a subsection of every section were measured and the cracks occurred at different ages of the pavement.

Table 7.7 Measured crack widths on surface of E17 at De Pinte by microscope.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Percentage of longitudinal reinforcement (%)</th>
<th>Temperature of pavement surface (°C)</th>
<th>Number of cracks</th>
<th>Number of readings</th>
<th>Crack width</th>
<th>Mean</th>
<th>Max.</th>
<th>Min.</th>
<th>Stdv.</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.75%</td>
<td>30.3</td>
<td>8</td>
<td>40</td>
<td>0.169</td>
<td>0.22</td>
<td>0.10</td>
<td>0.043</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>31.4</td>
<td>9</td>
<td>45</td>
<td>0.209</td>
<td>0.35</td>
<td>0.08</td>
<td>0.069</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>30.6</td>
<td>12</td>
<td>60</td>
<td>0.286</td>
<td>0.43</td>
<td>0.16</td>
<td>0.067</td>
<td></td>
</tr>
<tr>
<td>223</td>
<td>0.75%</td>
<td>8.8</td>
<td>14</td>
<td>42</td>
<td>0.117</td>
<td>0.30</td>
<td>0.03</td>
<td>0.076</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.70%</td>
<td>19.7</td>
<td>11</td>
<td>33</td>
<td>0.257</td>
<td>0.50</td>
<td>0.125</td>
<td>0.096</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.65%*</td>
<td>23.7</td>
<td>11</td>
<td>33</td>
<td>0.219</td>
<td>0.45</td>
<td>0.075</td>
<td>0.093</td>
<td></td>
</tr>
</tbody>
</table>

Note: * there is 20 kg/m³ steel fibers in the 0.65% section.

As can be seen in Table 7.7, the measured crack width on the pavement surface for all sections at both ages ranged from 0.08 to 0.50 mm at different temperature condition. The crack widths are all below 0.50 mm to prohibit the penetration of water. However, the cracks on the surface are relatively wider than expected. It should be noted that the measurements were done at relatively high temperature, and only half a year after construction. It probably is more innocent than it looks. The cracks are filled with dust and dirt that could attribute to less water penetration through the crack. In addition, as mentioned before, drilled cores have shown that the crack width suddenly decreases and becomes invisible along the depth. Historically, the CRCP under the current design concept shows excellent
Chapter 7 Characterization of Early Age Crack Pattern of CRCP

performance. Verhoeven (1993) also found that the degree of corrosion was extremely low for a pavement service of 20 years.

Table 7.7 also shows that the higher the longitudinal reinforcement percentage, the smaller the crack width at the pavement surface. The addition of the steel fibers seems to reduce a bit the crack width in the project of De Pinte. However, due to doubts about the correctness and reliability of the crack width measurements on the pavement surface, only qualitative conclusions on the effect of the longitudinal reinforcement percentage and the steel fibers on the crack width can be made based on the currently available data of this project.

**Effect of Active Crack Control Method**

Table 7.8 shows the comparison of crack width periodically measured by a microscope on the pavement surface for both active and non-active crack control test sections. The measured average crack width at the pavement surface on E313 was approximately 0.17 mm at an average pavement temperature of 20°C, and around 0.22 mm at an average pavement temperature of 6.0°C. The crack width on the passive crack control section of E17 is on average slightly larger than that of E313. The crack on the surface meets with the crack width limitation of CRCP in Europe, 0.40 mm.

**Table 7.8 Comparison of Crack Width on the Pavement Surface of Active and Non-active Crack Control Sections.**

<table>
<thead>
<tr>
<th>Section</th>
<th>Percentage of reinforcement</th>
<th>Season</th>
<th>Temperature of pavement surface (°C)</th>
<th>Number of cracks</th>
<th>Crack width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>E17 De Pinte</td>
<td>0.75%</td>
<td>Summer</td>
<td>30.3</td>
<td>8</td>
<td>0.169</td>
</tr>
<tr>
<td></td>
<td>0.75%</td>
<td>Winter</td>
<td>2.2</td>
<td>10</td>
<td>0.312</td>
</tr>
<tr>
<td>E313 30 mm</td>
<td>0.75%</td>
<td>Summer</td>
<td>21.0</td>
<td>11</td>
<td>0.198</td>
</tr>
<tr>
<td></td>
<td>0.75%</td>
<td>Winter</td>
<td>4.2</td>
<td>11</td>
<td>0.232</td>
</tr>
<tr>
<td>E313 60 mm</td>
<td>0.75%</td>
<td>Summer</td>
<td>20.5</td>
<td>17</td>
<td>0.152</td>
</tr>
<tr>
<td></td>
<td>0.75%</td>
<td>Winter</td>
<td>8.0</td>
<td>12</td>
<td>0.201</td>
</tr>
</tbody>
</table>

**7.4.2 Crack Width Movement**

Table 7.9 shows the crack width movement at different locations along the depth of the concrete slab on E17 in the summer. The crack movement varied with the vertical position. Larger crack movements at the pavement surface may be due to a positive temperature gradient and greater drying shrinkage in the daytime. It may indicate the bending behaviour of the slab due to a nonlinear temperature gradient. Unfortunately, no crack width measurements could be done during the nighttime. The daily crack movement at the location of the longitudinal reinforce-
ment is 0.140 mm, 0.143 mm, 0.158 mm for 3 sections when the temperature variation is 8.2°C to 10.6°C. The daily variation of crack movement is large compared to the absolute crack width. It once again indicates the importance to measure the temperature together with the crack width.

Table 7.9 Comparison of the change of crack width on E17 at De Pinte

<table>
<thead>
<tr>
<th>Crack width change</th>
<th>Section</th>
<th>Temperature change</th>
<th>Crack number</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>ΔW1</td>
<td></td>
<td></td>
<td>1 2 3 4 5 6 7</td>
<td></td>
</tr>
<tr>
<td>0.75%</td>
<td>22.0→30.2°C</td>
<td>0.132 0.202 0.328 0.182 0.190 0.170 0.206 0.201</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ΔW2</td>
<td></td>
<td></td>
<td>0.136 0.212 0.222 0.195 0.154 0.127 0.158 0.172</td>
<td></td>
</tr>
<tr>
<td>ΔW3</td>
<td></td>
<td></td>
<td>0.131 0.178 0.192 0.161 0.133 0.084 0.102 0.140</td>
<td></td>
</tr>
<tr>
<td>ΔW1</td>
<td>0.70%</td>
<td>20.9→31.5°C</td>
<td>0.070 0.058 0.260 0.142 0.240 0.180 0.078 0.147</td>
<td></td>
</tr>
<tr>
<td>ΔW2</td>
<td></td>
<td></td>
<td>0.169 0.093 0.205 0.149 0.243 0.190 0.163 0.173</td>
<td></td>
</tr>
<tr>
<td>ΔW3</td>
<td></td>
<td></td>
<td>0.155 0.088 0.143 0.110 0.211 0.188 0.105 0.143</td>
<td></td>
</tr>
<tr>
<td>ΔW1</td>
<td>0.65%</td>
<td>24.7→34.0°C</td>
<td>0.084 0.048 0.064 0.098 0.070 0.132 0.066 0.080</td>
<td></td>
</tr>
<tr>
<td>ΔW2</td>
<td></td>
<td></td>
<td>0.263 0.230 0.159 0.121 0.113 0.179 0.186 0.179</td>
<td></td>
</tr>
<tr>
<td>ΔW3</td>
<td></td>
<td></td>
<td>0.241 0.197 0.136 0.113 0.104 0.145 0.171 0.158</td>
<td></td>
</tr>
</tbody>
</table>

*: ΔW1 represents the crack width change on the surface measured by Microscope; ΔW2 represents the crack width change at a location 30 mm beneath the pavement surface measured by LVDT; ΔW3 represents the crack width change at the location of the longitudinal reinforcement steel bars by LVDT.

7.5 CONCLUSIONS AND RECOMMENDATIONS

7.5.1 Summary of Findings

Based on field observations of the crack pattern of several newly constructed CRCP under the current design concept in Belgium, the following conclusions can be drawn:

Conventional CRCP test sections
- The crack pattern is characterized as low mean crack spacing (approximately 1.0 m after 2.7 years in service) along with a high percentage of clusters of closely spaced cracks, which is similar to that of design concept 1.
- A steel percentage varying from 0.70% to 0.75% does not have a significant effect on the mean crack spacing, the crack spacing distribution, and the probability of clusters of closely spaced cracks. Before more field data become available to quantify the effects of longitudinal reinforcement on crack spacing and crack width, it is recommended to maintain the current longitudinal percentage of reinforcement of 0.75%.
- Field surveys of conventional CRCP sections indicate that it is difficult to reduce the probability of a non-uniform crack pattern significantly, such as closely spaced cracks, meandering and Y-cracks, by slightly adjusting the amount of longitudinal steel. The non-uniform crack pattern is inevi-
table and common in conventional CRCP roads, but may lead to punchout development.

- The crack widths measured on the pavement surface satisfy the requirement for all test sections. Another finding is that the higher the longitudinal reinforcement percentage, the smaller the crack width on the pavement surface. However, due to doubts about the correctness and reliability of the crack width measurements on the pavement surface, only qualitative conclusions on the effect of the longitudinal reinforcement percentage and the steel fibers on the crack width can be made based on the current available data of this project.

- Nevertheless, it has been found that all sections constructed under the current design concept behave excellently without any deterioration and maintenance. However, considering the long term durability of the pavement, further inspection and investigation of existing roads is needed to investigate whether the clusters of closely spaced cracks lead to punchout or not. In addition, a reliable, efficient, and precise crack width measurement method is needed.

**CRCP active crack control test sections**

- This active crack control method can significantly decrease the percentage of short spaced cracks and cluster cracks. Field investigations revealed that the cracks were much straighter and more regular for the active crack control section.

- Saw cut depth and saw cut timing influence the effectiveness of crack induction in this active control method. A larger saw cut depth and earlier saw cutting helps to induce cracks at the notches.

- The crack widths for the active crack control section were slightly smaller than those on the passive crack control section.

### 7.5.2 Recommendations

- More research efforts are needed to develop a practical and reliable crack width measurement method. It will be beneficial for evaluating the crack width development (residual shrinkage, relaxation), the effect of crack occurrence time on crack width, effect of crack spacing on crack width, and the effect of crack width in the punchout prediction model.

- After about 1.5 years on the E313, on one hand, not all the notches have initiated a crack, and on the other hand, there are cracks in between the notches. It should be investigated whether in future additional cracks, due to environmental effects and/or traffic loadings will occur at the currently non-cracked notches. If so, this results in clusters of closely spaced with all their disadvantages.
Conclusions and Recommendations

8.1 INTRODUCTION

The main objective of this study was to develop a method to further optimize the crack spacing pattern of CRCP. A new early entry method, partial surface notch, was proposed and adopted first in a reconstruction project in Belgium. Four main steps were needed to analyse the timing and depth of the saw cut of this active crack control method: predict the early age concrete temperature, measure the early age concrete properties, calculate the early age concrete stress for an uncut pavement, and optimize the timing and depth of the notch through a fracture mechanic based saw cut model. Extensive field investigations were conducted on two new CRCP sections in Belgium to evaluate the effect of the longitudinal reinforcement percentage and the active crack control method on the early age crack pattern of CRCP. The details of the work conducted in this study were discussed in Chapter 3 to Chapter 7. In each of the chapters, the important findings were reported at the end of the chapter. In this final chapter, the general conclusions and recommendations that resulted from the research described in this thesis are presented.

8.2 GENERAL CONCLUSIONS

8.2.1 Concrete Temperature in CRCP at Early Age

- The heat of hydration of concrete made of blended slag cement can be described by the De Schutter hydration model, and the corresponding model parameters can be determined by the isothermal calorimeter method.
• A critical review of current heat flux models at the pavement surface in the current widely used concrete pavement temperature models was performed. Adjustments of those heat flux models for the specific condition of concrete were suggested when necessary.
• A wind correction factor is required to calculate the convective heat flux at the pavement surface when directly using the wind speed data obtained from the standard weather stations. The wind speed at 2.0 m above the pavement surface is about 62% of that reported from the standard weather stations that are measured at 10 m above the ground. Using the uncorrected wind speed from the standard weather stations leads to an overestimation of the convective heat flux at the pavement surface, and finally results in an underestimation of the temperature gradient within the pavement.
• A theoretical model is presented to describe the heat flux by convection when a plastic sheet is used for curing of the concrete. Simulation results show that both the shortwave and longwave optical properties of the plastic sheet have a large effect on the extent of heating of the concrete beneath certain plastic sheets. This suggests the possible use of a certain type of plastic sheet according to its optical properties to minimize the pavement temperature for summer construction.
• The proposed temperature model was verified by comparing the predicted and measured concrete temperatures for two projects in Belgium. The predicted temperatures show a satisfying match with field measured data. It may be concluded that the proposed temperature model is able to give a good prediction of the early age concrete temperature development for concrete pavements, especially for the Belgian CRCP roads.
• A procedure to generate reliable climate inputs was proposed by using easily to obtain daily maximum and minimum climatic data from the weather forecast. It provides real-time data for the early age concrete temperature prediction program, and it would be ideal during construction periods by allowing the contractor to modify design or construction operations based on the predicted results.

8.2.2 Concrete Fracture Energy at Early Age
• The applied unnotched parabolic shape concrete specimens, the used tension test set-up with three hinges, and the applied test procedures worked well to obtain the complete softening curves for the Belgium CRCP concrete mixtures, even at the very early age of only 24 hours.
• The fracture energy, uniaxial tensile strength and the elastic modulus of the concrete were determined by the obtained complete softening curves.
The fracture energy, uniaxial tensile strength, and elastic modulus were all found to increase with age going towards a horizontal asymptote as concrete hardened in a tested age range of 1 day to 90 days. The development rate of the fracture energy was found to be higher as compared to the tensile strength and the stiffness. Based on the measurement results on hardening concrete specimens, a degree of hydration-based description for the uniaxial tensile strength, elastic modulus, and fracture energy has been elaborated.

- Higher fracture energy, uniaxial tensile strength, and elastic modulus are observed for the concrete specimens with maximum aggregate size 6.3 mm, which does not coincide with the tendency from the literature. Besides, a larger scatter of the measured results is observed for the specimen with a maximum aggregate size 16 mm, which has an aspect ratio of 3.125. It indicates that this aspect ratio is too small, and might be an insufficient representative volume element to determine the fracture energy through this parabolic shape specimen. Due to the shape of the split mould, insufficient compaction at the center of the specimen as found in the ACRe project may result in weak locations in the center of the specimens, especially for the specimen type 16, and this would obscure the actual tensile strength and fracture energy of the concrete.

8.2.3 Concrete Stress Development in CRCP at Early Age

- Simulation results have also shown that it would be non-conservative to neglect the relaxation effect in the early age concrete stress development, since the rapid relaxation of compressive stress after the final set of concrete would cause the zero stress temperature to be higher. Ignoring the relaxation effect leads to overestimation of the cracking time and an increase of the risk of initiation of uncontrolled random cracks before the sawcut implementation, especially for JPCP.

- The simulation results have revealed that the contribution of the shrinkage on the stress development at the early age is minimal while the effect of the temperature is substantial. This is attributed to the moisture retention effect of the adequately applied curing methods (plastic sheet, curing compound). The effect of autogenous shrinkage is also negligible because the water to cement ratio of the commonly used paving concrete mixtures is normally between 0.40 and 0.43. It indicates that the zero stress temperature and the potential cracking time for concrete pavements can be accurately determined only considering the temperature variations when adequate curing methods are implemented.
The analysis has shown that the Eurocode 2 shrinkage model, which provides the average shrinkage of concrete elements, is unable to quantify the effect of the nonlinear shrinkage on the early age stress development. The Eurocode 2 shrinkage model gives an underestimation of the shrinkage at the top of the slab.

A parametric sensitivity analysis has shown that the early age concrete temperature (built-in-curing, peak temperature) and stress development (zero-stress temperature and potential cracking time) are closely related with the typical design, construction and climatic conditions of CRCP in Belgium, such as time on the day of the concrete placement, construction season, plastic sheet curing, base temperature, concrete placement temperature, wind speed, cloud cover. Numerical simulation results indicate that the use of plastic sheet curing is feasible for most of the Belgium conditions except construction on a warm and sunny summer day. Caution is necessary under those conditions.

Concrete placed under warmer weather conditions develops much higher peak temperature and zero stress temperature, and a larger built-in temperature gradient than concrete placed under cold weather conditions. Moreover, concrete placed at different times of a day has a different temperature history, and thus results in different stress and strength development. Therefore, adequate scheduling of the time of concrete placement related to the weather conditions, can avoid the unfavourable too high temperature in the concrete pavement.

The development of the early age concrete stress is inverse to the temperature, and the development of the temperature follows the daily climatic cycle. The fluctuation in the temperature and stress are higher in the slab surface. The critical tensile stresses were found at the pavement surface in the nighttime, which matched with the field surveys that all the observed early age cracks only propagated during the night.

The zero stress temperature is not a single temperature, but varies through the depth of the slab and is largest at the pavement surface. The built-in temperature gradient is also not always a negative value, and a positive built-in temperature gradient is observed for cold weather afternoon construction. The use of a plastic sheet can also reduce the built-in temperature gradient.

8.2.4 Active Crack Control for CRCP

Field investigations have revealed that the transverse cracks are much straighter and more regular spaced for the active crack control section of CRCP. The active crack control method can significantly reduce the per-
percentage of short-spaced cracks and cluster cracks and thus will reduce the risk of punchout development in the long-term of CRCP.

- Field evidences have indicated that the partial surface notch is very effective in inducing cracks beneath the applied notches: around 20% of the notches propagated to fully developed cracks after three nights, subsequently increasing to approximately 60% after 2 months, and about 70% after 1.5 years.

- The saw cut depth and saw cut timing influence the effectiveness of the crack induction in the proposed active crack control method. A larger saw cut depth and earlier saw cutting were found to be beneficial to induce cracks at the notches.

- The effects of saw cut depth (in the pavement depth direction) and saw cut length (in the pavement transverse direction) on crack initiation are evaluated through the calculation of the stress intensity factors at the critical locations under various stress conditions by the weight function method. Besides considering the requirement of sufficient concrete cover and worrying about the deterioration of the unsealed notch due to traffic tire wear, it is concluded that the applied notch length of 400 mm and notch depth of 60 mm on E313 is appropriate.

- The estimated final set time gives the lower limit for the saw cutting operation to avoid ravelling, and the predicted cracking time gives the upper limit of the saw cutting window before the initiation of random occurring natural cracks. A procedure is developed to enable the calculation of the end of the saw cutting window using the tensile stress from an uncut pavement, the tensile strength of concrete, and a stress enlargement factor. The stress enlargement factor is determined according to the principle of linear elastic fracture mechanics, and it depends on the fracture toughness and the tensile strength of the concrete, and the notch geometry and dimension.

- The crack widths for the active crack control section were slightly smaller than those on CRCP sections without notches.

### 8.2.5 Crack Pattern of CRCP in Belgium

- The crack pattern of the current standard CRCP design concept in Belgium is characterized as low crack spacing (approximately 1.0 m after 2.7 years in service) along with a high percentage of clusters of closely spaced cracks. It should be mentioned that these CRCPs behave excellent and are hardly subjected to any deterioration mainly due to the good base support.
• A steel percentage varying from 0.70% to 0.75% does not have a significant effect on the mean crack spacing, the crack spacing distribution, and the probability of clusters of closely spaced cracks. Before more field data become available to quantify the effects of the longitudinal reinforcement on crack spacing and crack width, it is recommended to maintain the current longitudinal reinforcement percentage of 0.75%.

• Field surveys of conventional CRCP sections indicate that it is difficult to reduce the probability of a non-uniform crack pattern significantly, such as closely spaced cracks, meandering and Y-cracks, by slightly adjusting the amount of longitudinal steel. The non-uniform crack pattern is inevitable and common in conventional CRCP roads, but may lead to punchout development.

• The crack widths measured at the pavement surface satisfy the requirement for all test sections. Another finding is that the higher the longitudinal reinforcement percentage, the smaller the crack width at the pavement surface. However, due to doubts about the correctness and reliability of the crack width measurements on the pavement surface, only qualitative conclusions on the effect of the longitudinal reinforcement percentage and the steel fibers on the crack width can be made based on the current available data of this project.

8.3 RECOMMENDATIONS

• A pavement moisture prediction model is recommended to incorporate into modelling the early age temperature, the shrinkage, and the induced stresses for CRCP at the early age. It enables to accurately determine the effect of evaporation cooling on the concrete temperature, and the effect of nonlinear moisture loss within the pavement slab on the shrinkage-induced stresses.

• The accuracy of the proposed early age temperature and stress prediction model relies on the determination of the ultimate degree of hydration and the critical degree of hydration (considered as the final set of concrete) of the concrete mixtures. Further research and testing on those subjects for various concrete mixtures and compositions is highly recommended.

• In order to accurately determine the latest time of saw cutting in a concrete pavement before randomly cracks occur, the concrete properties, including tensile strength, elastic modulus, and fracture energy or fracture toughness, for very early age concrete (less than 24 hour after mixing with water) should be quantified in the future. Considering the difficulties and drawbacks, the proposed deformation-controlled uniaxial tensile test of parabolic shape specimens in the present study might be not suited
for the very early age concrete. The indirect method, such as a wedge splitting test, might be an alternative and practical option to obtain the softening curves for the very early age concrete.

- In this study, the stress calculation was performed to determine the cracking time of the first series of cracks in CRCP. It is based on the assumption that the CRCP is considered as completely restrained before transverse cracking. However, this restraint condition is not valid for the later age transverse cracking. The degree of restraint and the bond stress and slip relationship between the longitudinal reinforcement rebars and the surrounding concrete should be quantified to study the mechanism of later-age transverse cracking in CRCP.

- Further studies are suggested to optimize the notch timing, geometry, and interval for the Belgium design, construction, and climatic conditions.

- More research efforts are needed to develop a practical and reliable crack width measurement method for CRCP to get more reliable crack width data that enables better analyses of its influence on the long-term performance of CRCP.

- Since the surveyed CRCP sections were too young to have punchouts, a clear relationship between the proposed active crack control method and punchouts could not be established. The field investigation on those surveyed sections needs to be continued to identify the effect of the active crack control method and the effect of the early age crack pattern on the long-term performance of CRCP.

- As variability is inherent in the material properties, the construction, and the environmental conditions, a reliability based analysis of early age cracking in CRCP is recommended for further research. The Monte Carlo method might fulfil this purpose.
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List of Symbols

Latin Capital Characters

\( A_c \)  
projected area of the fracture plane that is perpendicular to the loading direction in the uniaxial tension test

\( A_g \)  
area of the cross section of the glue layers in the uniaxial tension test

\( A_{\text{min}} \)  
area of the narrowest cross section of the parabolic shape specimen used in the uniaxial tension test

\( A_s \)  
area of the cross section of the steel cap in the uniaxial tension test

\( A_z \)  
area of the cross section of the parabolic shape specimen at the depth \( z \), see the coordinate system defined in Figure 5.1

\( C \)  
a correction factor in Equation 4.19 depending on heat flow condition

\( C_{\text{curling}} \)  
Bradbury’s curling stress coefficient, Chapter 6

\( C_{cw} \)  
a local calibration factor in MEPDG crack width prediction model

\( DAY \)  
the number of the day of the year

\( E_c \)  
elastic modulus of concrete in linear elastic stage

\( E_p \)  
activation energy for P-reaction

\( E_s \)  
activation energy for S-reaction

\( E_{\text{steel}} \)  
elastic modulus of steel in linear elastic stage

\( E_{\text{glue}} \)  
elastic modulus of glue in linear elastic stage

\( F \)  
applied load in the uniaxial tension test

\( F_{\text{cloud}} \)  
cloud cover factor

\( F_m \)  
measured external force by load cell

\( F_{cr} \)  
applied external force on the fractured surface of the tested specimen

\( F_{sw} \)  
dead load of the middle hinge, the top steel cap and the upper half of the concrete specimen
List of Symbols

\( F_u \) corrected peak load on the fracture surface
\( F_{0.1}, F_{0.5} \) 10% and 50% of the corrected peak load, respectively
\( G_f \) fracture energy of concrete, determined by the uniaxial tension test
\( G_F \) size independent fracture energy of concrete
\( G_{IC} \) critical energy release rate of concrete
\( I_f \) daily solar radiation intensity factor
\( K \) stress intensity factor
\( K^A, K^B \) stress intensity factor at given crack tip, the superscriptions stands for the corresponding crack tip as shown in Figure 6.28
\( K_{IC} \) fracture toughness of concrete
\( \bar{L} \) mean crack spacing estimated by MEPDG
\( M_{iA}, M_{iB} \) parameters in determining stress intensity factor at given crack tip based on the weight function method proposed by Glinka et al.
\( N \) cloud cover
\( P_b \) percentage of longitudinal reinforcement
\( P_{slag} \) percentage of the replacement of slag in the Blended cement
\( Q \) elliptical shape constant
\( Q_{max} \) maximum heat of hydration
\( Q_{P,max} \) maximum heat of hydration for P-reaction
\( Q_{S,max} \) maximum heat of hydration for S-reaction
\( R \) universal gas constant
\( R_Z \) radius of the cross section area of the specimen at the depth \( z \)
\( R_{np} \) net radiation at the pavement surface in case of plastic sheet curing
\( R_{np,s} \) net radiation at the plastic sheet surface
\( T \) concrete temperature
\( T_a \) air temperature
\( T_{axial} \) axial uniform temperature causes concrete pavement slab to expand or contract evenly through its depth
\( T_{dp} \) dew point
\( T_{ebi} \) effective built-in equivalent linear temperature gradient
\( T_l \) Linke turbidity that describes the dispersion and pollution of the atmosphere
\( T_{linear} \) equivalent linear temperature gradient of pavement slab
\( T_{max} \) maximum air temperature of a day based on weather forecast or the monthly maximum air temperature based on historical records
\( T_{min} \) minimum air temperature of a day based on weather forecast
or the monthly minimum air temperature based on historical records

\( T_{\text{nonlinear}} \) nonlinear temperature component that causes internal self-equilibrating strain

\( T_{n,k} \) concrete temperature along the pavement slab at a given age, the subscriptions \( n \) and \( k \) are defined in Figure 4.17

\( T_{ps} \) temperature of plastic sheet

\( T_s \) temperature at the pavement surface

\( T_{sky} \) effective sky temperature

\( T_{sr} \) air temperature at sun-rise

\( T_{ss} \) air temperature at sun-set

\( T_{zs} \) zero-stress concrete temperature

\( T_{\text{zone}} \) time zone for a given location

\( T_{\zeta m} \) concrete temperature at the depth of the reinforcement

\( U_m \) maximum bond stress between concrete and longitudinal reinforcement rebars

\( W_f \) the applied external work that is needed to fracture the specimen

**Latin Small Characters**

\( a, b, c, a_s \) regression constants for De Schutter hydration model

\( c \) specific heat capacity

\( c_{c1}, c_{c2}, c_b \) specific heat capacity of each pavement layer

\( c_1, c_2 \) bond stress coefficients for the crack width prediction model in MEPDG

\( cw_{\text{max}} \) maximum crack width of CRCP in the phase of the uncompleted crack pattern, estimated by Delft Tension Bar model

\( cw_{mf} \) mean crack width of CRCP in the phase of fully developed crack pattern, estimated by Delft Tension Bar model

\( cw_{mo} \) mean crack width of CRCP in the phase of the uncompleted crack pattern, estimated by Delft Tension Bar model

\( d_1, d_2, ..., d_n \) thickness of the insulation layers at the pavement surface

\( d_s \) diameter of the longitudinal steel reinforcement bars

\( f_{cc,k,o} \) concrete characteristics cube compressive strength at 28 days for loading of short duration

\( f_{cc,m,o} \) mean compressive strength of concrete at 28 days for loading of short duration

\( f_{ct,m,o} \) mean tensile strength of concrete at 28 days for loading of short duration

\( f_t \) uniaxial tensile strength of concrete

\( f_{t28} \) concrete tensile strength at 28 days
List of Symbols

\( f_\sigma \) maximum longitudinal concrete tensile stress at the depth of the steel in CRCP, in the MEPDG crack width prediction model

\( h_0 \) overall convective heat transfer coefficient for pavement surface with \( n \) insulation layers

\( h_{\text{conv}} \) convective heat transfer coefficient for pavement surface without insulation layers

\( h_{\text{conv,ps}} \) convective heat transfer coefficient for pavement surface in case of plastic sheet curing, calculated by Equation 4.19

\( h_{\text{conv,ps,CIMS}} \) convective heat transfer coefficient for pavement surface with insulation layers, estimated by CIMS regression equations

\( h_{\text{pec}} \) thickness of concrete pavement slab

\( h_{\text{specimen}} \) height of concrete specimen in the uniaxial tension test

\( k \) modulus of substructure reaction at top of layer under consideration

\( k_1, k_2, \ldots, k_n \) thermal conductivity of the insulation layers at the pavement surface

\( l \) radius of the relative stiffness of the pavement slab

\( l_t \) effective bond stress transfer length

\( m, n, p \) roughness parameters in the McAdams’ model in Chapter 4

\( q \) overall heat generation rate by the hydration of cementitious materials

\( q_0 \) solar constant that is the solar radiation per unit area that would be incident on a plane perpendicular to the sunlight, measured above the Earth’s atmosphere

\( q_{\text{conv}} \) convective heat transfer at pavement surface

\( q_{\text{ins}} \) instantaneous solar radiation reached to the pavement surface

\( q_{\text{ir}} \) net thermal irradiation for the pavement surface

\( q_s \) heat generation rate of S-reaction

\( q_{s,\text{max},T} \) maximum heat generation rate of S-reaction in the isothermal condition of a given temperature \( T \)

\( q_{\text{sol}} \) absorbed solar radiation at the pavement surface

\( q_{\text{solar,peak}} \) peak value of the overall solar radiation that reached to the pavement surface at a given day

\( q_{\text{solar,direct}} \) direct solar radiation that reached to the pavement surface in a bright day

\( q_{\text{solar,diffuse}} \) diffuse solar radiation that reached to the pavement surface

\( q_p \) heat generation rate of P-reaction

\( q_{p,\text{max},T} \) maximum heat generation rate of P-reaction in the isothermal condition of a given temperature \( T \)
List of Symbols

\( r_p \)  
degree of P-reaction

\( r_s \)  
degree of S-reaction

\( t \)  
the actual calculated time

\( t_{et} \)  
the Equation of time

\( t_{sn} \)  
time of solar noon

\( t_{sr} \)  
time of sunrise

\( t_{ss} \)  
time of sunset

\( u_{0.1}, u_{0.5} \)  
deformation of the concrete specimen at 10% and 50% of the corrected peak load in the uniaxial tension test, respectively

\( v_{wind} \)  
wind speed near the pavement surface, here chosen as 2 m above the surface

\( v_{10} \)  
wind speed measured at the standard height of 10 m from the weather stations

\( w \)  
crack opening of concrete specimen in the uniaxial tension test

\( w_{cr} \)  
maximum crack opening of concrete specimen in the uniaxial tension test

\( x \)  
distance from the pavement surface

\( z \)  
deepth of the specimen in the ordinate system as shown in Figure 5.2.

Greek Characters

\( \alpha \)  
degree of hydration of cementitious material

\( \alpha_f \)  
degree of hydration at the final set of concrete

\( \alpha_{CTE} \)  
coefficient of thermal expansion of concrete

\( \alpha_h \)  
degree of hydration of concrete under field condition

\( \alpha_p \)  
shortwave reflectivity of the pavement surface

\( \alpha_u \)  
ultimate degree of hydration of cementitious material

\( \alpha_{wind} \)  
a constant in the wind speed correction model in Chapter 4

\( \beta \)  
a slope parameter in the FHP hydration model, see Chapter 2

\( \beta_{wind} \)  
a constant in the wind speed correction model in Chapter 4

\( \gamma_{abs} \)  
absorptivity of shortwave solar radiation at the pavement surface

\( \gamma_{so} \)  
factor to take variation of the crack width in the phase of uncompleted crack pattern in the Delft Tension bar model

\( \gamma_{\infty} \)  
factor to take care of loadings of long duration or cyclic loadings in the Delft Tension bar model

\( \delta \)  
solar declination angle

\( \delta \)  
corrected average overall deformation in the uniaxial tension test

\( \delta_e \)  
elastic deformation of the specimen before the peak load

\( \delta_p \)  
plastic deformation of the specimen before the peak load
List of Symbols

\( \delta_{pu} \) plastic deformation of the specimen at the peak load
\( \varepsilon_l \) emissivity of plastic sheet
\( \varepsilon_{sh} \) unrestrained shrinkage of concrete
\( \varepsilon_{sky} \) effective sky emissivity
\( \varepsilon_p \) emissivity of concrete
\( \zeta \) concrete cover, the depth of the longitudinal reinforcement bar below the pavement surface in CRCP
\( \lambda \) thermal conductivity
\( \lambda_c, \lambda_a, \lambda_b \) thermal conductivity of each pavement layers
\( \nu \) Passion’s ratio of concrete
\( \rho_c, \rho_a, \rho_b \) density of each pavement layers
\( \rho_l \) longwave reflectivity of the plastic sheet
\( \rho^*, \rho_{ir}^* \) variables representing the multiple reflections of the shortwave and longwave radiations between the plastic sheet and pavement surface
\( \sigma \) Stefan-Boltzmann constant
\( \sigma_0 \) Westergaard’s nominal stress factor
\( \sigma_{cr} \) concrete tensile stress just before cracking
\( \sigma_N \) nominal failure stress
\( \sigma_{s,cr} \) longitudinal steel tensile stress just before cracking
\( \tau \) a time parameter for FHP hydration model, see Chapter 2
\( \tau_s \) shortwave transmissivity of plastic sheet
\( \tau_l \) longwave transmissivity of plastic sheet
\( \varphi_{lat} \) latitude for a given location
\( \varphi_{lon} \) longitude for a given location
\( \varphi_{se} \) solar elevation angle
\( \varphi_{sz} \) solar zenith angle
\( \omega \) hour angle
\( \omega_0 \) hour angle at the sunrise or sunset
\( \Delta x_c, \) space increment in each pavement layers
\( \Delta x_a, \Delta x_b \)
\( \Delta t \) time increment
# List of Abbreviation

<table>
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<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tr>
<td>CRCP</td>
<td>Continuously Reinforced Concrete Pavements</td>
</tr>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ACRe</td>
<td>Asphalt Concrete Response</td>
</tr>
<tr>
<td>AME</td>
<td>Absolute Mean Error</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>CIMS</td>
<td>Computer Interactive Maturity System</td>
</tr>
<tr>
<td>CRCP</td>
<td>Continuously Reinforced Concrete Pavements</td>
</tr>
<tr>
<td>CTE</td>
<td>Coefficient of Thermal Expansion</td>
</tr>
<tr>
<td>EICM</td>
<td>Enhance Integrated Climatic Model</td>
</tr>
<tr>
<td>EPA</td>
<td>United States Environmental Protection Agency</td>
</tr>
<tr>
<td>FCM</td>
<td>Fictitious Crack Model</td>
</tr>
<tr>
<td>FEBELCEM</td>
<td>Federation of the Belgian Cement Industry</td>
</tr>
<tr>
<td>FEMMASSE</td>
<td>Finite Element Modulus for Material Science and Structural Engineering</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>FPZ</td>
<td>Fracture Process Zone</td>
</tr>
<tr>
<td>GGBFS</td>
<td>Ground Granulated Blast Furnace Slag</td>
</tr>
<tr>
<td>HIPERPAV</td>
<td>HiGH PERformance concrete PAVing</td>
</tr>
<tr>
<td>HYMOSTRUC</td>
<td>HYdration, MOmorphology and STRUCture</td>
</tr>
<tr>
<td>JPCP</td>
<td>Jointed Plain Concrete Pavements</td>
</tr>
<tr>
<td>KNMI</td>
<td>Royal Netherlands Meteorological Institute</td>
</tr>
<tr>
<td>LEFM</td>
<td>Linear Elastic Fracture Mechanics</td>
</tr>
<tr>
<td>LTPP</td>
<td>Long-Term Pavement Performance</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transformer</td>
</tr>
<tr>
<td>MEPDG</td>
<td>Mechanistic-Empirical Pavement Design Guide</td>
</tr>
<tr>
<td>NCHRP</td>
<td>National Cooperative Highway Research Program</td>
</tr>
<tr>
<td>NOAA</td>
<td>National Oceanic and Atmospheric Administration</td>
</tr>
<tr>
<td>Abbreviation</td>
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<td>--------------</td>
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<tr>
<td>RES</td>
<td>Sum of the RESidual</td>
</tr>
<tr>
<td>RMSE</td>
<td>Root Mean Square Error</td>
</tr>
<tr>
<td>RMI</td>
<td>the Royal Meteorological Institute of Belgium</td>
</tr>
<tr>
<td>TMAC²</td>
<td>Temperature and Moisture Analysis for Curing Concrete</td>
</tr>
<tr>
<td>UTT</td>
<td>Uniaxial Tensile Test</td>
</tr>
<tr>
<td>VENCON</td>
<td>Dutch Structural Design Method for Jointed Plain and Continuously Reinforced Concrete Pavements</td>
</tr>
<tr>
<td>WMO</td>
<td>World Meteorological Organization</td>
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Appendix I

Matlab Codes for Calculation of Early Age Concrete Temperature and Stress

Part A

In this part, a Matlab code designed to generate the real-time climatic inputs for the temperature prediction model based on limited weather forecast data is presented. The detail of the procedure is presented in Section 4.4.

```matlab
% Real-time climatic inputs generation through weather forecast data
% Overall Input Informations
para=xlsread('weather inputs.xlsx','Input Informations');
deg_lati=para(1,1); % latitude of worksite, ‘east’ is positive and ‘west’ is negative, in degree
deg_long=para(2,1); % longitude of worksite, in degree
Timezone=para(3,1); % time zero, positive of east
t_year=para(4,1); % year
t_month=para(5,1); % month
t_day=para(6,1); % day
Linke=para(7,1); % Linke turbidity, normally 4 for Belgium condition
Tair_MonthMax=para(8,1); % monthly maximum air temperature, [°C]
Tair_MonthMin=para(9,1); % monthly minimum air temperature, [°C]
RH_MonthMax=para(10,1); % monthly maximum relative humidity, [%]
RH_MonthMin=para(11,1); % monthly minimum relative humidity, [%]
Wind_Month=para(12,1); % monthly mean wind speed, [m/s]
%
% Sunset and sunrise time calculated by NOAA model
DAY=fix(275*t_month/9)-fix((t_month+9)/12)+(1+fix((t_year-4*fix(t_year/4)+2)/3))+t_day-30; % DAY is the number of the days elapsed in a year
f_DAY=2*pi/365*(DAY-1); % the fractional of the year, in radians
eqtime=229.18*(0.000075+0.001868*cos(f_DAY)-0.032077*sin(f_DAY)-0.014615*cos(2*f_DAY)...
...-0.040849*sin(2*f_DAY)); % the Equation of time, in minutes
decl=0.006918-0.399912*cos(f_DAY)+0.070257*sin(f_DAY)-0.006758*cos(2*f_DAY)-0.000907*sin(2*f_DAY)-0.002697*cos(3*f_DAY)+0.00148*sin(3*f_DAY); % decl is the solar declination angle, in radians
HourAngle_0=(180/pi)*acos((sin(-0.83*pi/180)-sin(deg_lati*pi/180)*sin(decf))... /
/(cos(deg_lati*pi/180)*cos(decf))); % hour angle at solar noon, in degree
t_SolarNoon=12+Timezone-deg_long/15-eqtime/60; % local solar noon time, in hour
N_sr=t_SolarNoon-HourAngle_0/15; % sunrise time in local time, in hour
N_ss=t_SolarNoon+HourAngle_0/15; % sunset time in local time, in hour
```

N_sr=fix(N_sr*60);
N_ss=fix(N_ss*60);
% Every minute climatic inputs generation
% Solar radiation
for i=1:1440
    HourAngle(i)=(i-60*Timezone+4*deg_long+eqtime)/4-180; % hour angle at time i, in degree
    eg_se(i)=asin(sin(deg_lati*pi/180)*sin(decl)+cos(deg_lati*pi/180)*cos(decl)*cos(HourAngle(i)*pi/180)); % solar elevation angle, in degree, by Slob and Moona, 1991
    if sin(deg_se(i))<0.0 % night time
        qsolar(i)=0; % overall solar radiation
    elseif sin(deg_se(i))<0.17&&sin(deg_se(i))>=0.0
        qsolar(i)=666*sin(deg_se(i));
    else % when solar elevation angle higher than 5 degree
        qsolar_L(i)=1367*exp(-Linke/(0.9+9.4*sin(deg_se(i))))*sin(deg_se(i)); % direct solar radiation
        qsolar_D(i)=40.3+41.3*Linke*sin(deg_se(i)); % diffuse solar radiation
        qsolar(i)=qsolar_D(i)+qsolar_L(i);
    end
end
% Air temperature by De wit, 1978
T_ss=Tair_MonthMin+(Tair_MonthMax-Tair_MonthMin)*sin((N_ss-N_sr-120)/(N_ss-N_sr)*pi);
for i=1:1440
    if i<=N_sr+120 % mid-night to the time that 2 hours after sunrise time
        Tair(i)=T_ss*exp(log(Tair_MonthMin/T_ss)*(i+1440-N_ss)/(26*60+N_sr-N_ss));
    elseif i>N_ss
        Tair(i)=T_ss*exp(log(Tair_MonthMin/T_ss)*(i-N_ss)/(26*60+N_sr-N_ss));
    else % mid-night to the time that 2 hours after sunrise time
        Tair(i)=Tair_MonthMin+(Tair_MonthMax-Tair_MonthMin)*sin((i-N_sr-120)/(N_ss-N_sr)*pi);
    end
end
% Relative humidity
RH_ss=RH_MonthMax+(RH_MonthMin-RH_MonthMax)*sin((N_ss-N_sr-120)/(N_ss-N_sr)*pi);
for i=1:1440
    if i<N_sr+120
        RH(i)=RH_ss*exp(log(RH_MonthMax/RH_ss)*(i+1440-N_ss)/(26*60+N_sr-N_ss));
    elseif i>N_ss
        RH(i)=RH_ss*exp(log(RH_MonthMax/RH_ss)*(i-N_ss)/(26*60+N_sr-N_ss));
    else
        RH(i)=RH_MonthMax+(RH_MonthMin-RH_MonthMax)*sin((i-N_sr-120)/(N_ss-N_sr)*pi);
    end
end
% Dew point
for i=1:1440
    Tdp(i)=log(RH(i)/100)+17.67*Tair(i)/(243.5+Tair(i))*243.5/(17.67*log(RH(i)/100) ... +17.67*Tair(i)/(243.5+Tair(i)));
end
% Wind speed
for i=1:1440
    Onewind(i)=Wind_Month;
end
Part B

In this part, a Matlab code designed for the prediction of concrete pavement temperature at the early age according to the procedures described in Chapter 4 is presented.

```matlab
% Temperature Prediction Model
% Overall Input Informations
% Pavement Structure and Thermal Parameters
para=xlsread('Input_E17_6m_slag.xlsx','Input Informations'); % an example for the project E17
hcond=para(1:5,1); % heat conductivity, [J/kg]
den=para(1:5,2); % density, [kg/m^3]
hc=para(1:5,3); % specific heat capacity, [J/kg/K]
th=para(1:5,4); % thickness, [m]
n_layer=length(para(1:5,1)); % number of layers, [-]
m=para(1:5,5); % number of nodes of each layers, [-]

% Slag Cement Thermal Properties
C=para(8,1); % cement content, [kg/m^3]
R=para(9,1); % universal gas constant, J/mol/K
w_c=para(10,1); % water to cement ratio
slag=para(11,1); % percentage of slag
a_p=para(12,1); % coefficient of P reaction under isothermal condition, [-]
b_p=para(13,1); % coefficient of P reaction under isothermal condition, [-]
c_p=para(14,1); % coefficient of P reaction under isothermal condition, [-]
qp_max_20=para(15,1)/3.6; % peak heat production rate of P reaction at 20, [W/kg]
Qp_max_20=para(16,1); % total heat of hydration of P reaction at 20, [J/kg]
Ea=para(17,1); % apparent activation energy of P reaction, [J/mol]
rp_ini=para(18,1); % degree of hydration at the starting of P reaction, [-]
a_s=para(19,1); % coefficient of S reaction under isothermal condition, [-]
qs_max_20=para(20,1)/3.6; % peak heat production rate of P reaction at 20, [W/kg]
Qs_max_20=para(21,1); % total heat of hydration of P reaction at 20, [J/kg]
Es=para(22,1); % apparent activation energy of P reaction, [J/mol]
r_s=para(23,1); % degree of hydration at the starting of S reaction, [-]
r_pb=para(24,1); % degree of hydration of P reaction when the S reaction starts
Qmax=Qp_max_20+Qs_max_20; % maximum potential heat, [J/kg]

ah_u=(1.031*w_c)/(0.194+w_c)+0.3*slag; % ultimate degree of hydration
ah_critical=0.26*w_c; % critical degree of hydration of the development of strength, [-]

% Boudary Conditions
emissivity_p=para(27,1); % concrete surface emissivity, [-]
absorptivity_p=para(28,1); % the concrete slab surface absorptivity, [-]
T_ConstantGround=para(29,1); % constant ground temperature, [C]

% Construction Informations
th_insulation=para(31,1); % thickness of insulation layer, [m]
hcond_insulation=para(32,1); % thermal conductivity of insulation layer, [W/m/K]
t_curing=3600*para(33,1); % time of initial curing method, the time of plastic sheeting, [hour]
T_ConcreteInitial=para(34,1); % concrete temperature during placement, [C]

% Optical Propertise of plastic sheets
transmissivity_s=para(8,6); % transmissivity of plastic sheet to short wave radiation, [-]
reflectivity_s=para(9,6); % reflectivity of plastic sheet to short wave radiation, [-]
transmissivity_L=para(10,6); % transmissivity of plastic sheet to long wave radiation, [-]
reflectivity_L=para(11,6); % reflectivity of plastic sheet to long wave radiation, [-]
emissivity_L=1-reflectivity_L-transmissivity_L;
reflectivity_p=1-absorptivity_p;
stefan=5.669e-8; % stefan-boltzmann constant, [-]

% Concrete Properties
```
Appendix I

%ultimate elasticity of modulus at 28 days,[MPa]
Ec_ultimate=para(36,1);

%coefficient of ultimate elasticity of modulus at 28 days,[\text{-}]
Coe_Ec=para(37,1);

%ultimate tensile strength at 28 days,[MPa]
ft_ultimate=para(38,1);

%coefficient of ultimate tensile strength at 28 days,[\text{-}]
Coe_ft=para(39,1);

%ultimate fracture energy at 28 days,[N/m]
G_ultimate=para(40,1);

%coefficient of fracture energy at 28 days,[\text{-}]
Coe_G=para(41,1);

%coefficients of thermal expansion,[\text{-}]
a_COTE=para(42,1);

%Poisson ratio,[\text{-}]
v=para(43,1);

%Numerical Implementation

t_prediction=24*3600*para(45,1); %duration of prediction,[second]
delta_t=para(46,1); %time step,[\text{-}]
t_placement=3600*para(48,1); %concrete placement time
emissivity_asphalt=para(49,1); %ground surface emissivity of asphalt interlayer,[\text{-}]
absorptivity_asphalt=para(50,1); %the asphalt surface absorptivity,[\text{-}]

%Climatic Inputs, either using the recorded
climate=xlsread('Input_E17_6m_slag.xlsx','Climate')
%(1)daily temperature
t_climate=24*3600*climate(:,2); % original time increment of weather station climate data
t_calculate=t_prediction+t_placement;
Tair_calculate=climate(:,3);
Tair=interp1(t_climate,Tair_calculate,(0:delta_t:t_calculate)', 'linear'); %[\text{C}]
%(2)dew point
Tdp_calculate=climate(:,5);
Tdp=interp1(t_climate,Tdp_calculate,(0:delta_t:t_calculate)', 'linear'); %[\text{C}]
%(3)wind velocity
wind_raw=xlsread('Input_E17_6m_slag.xlsx','wind')
time_wind=24*3600*wind_raw(:,1);
wind_climate=wind_raw(:,4)/3.6*0.62; % 0.62 is wind correction factor
wind=interp1(time_wind,wind_climate,(0:delta_t:t_calculate)', 'linear'); %[\text{m/s}]
%(4)relative humidity
RH_calculate=climate(:,4);
RH=interp1(t_climate,RH_calculate,(0:delta_t:t_calculate)', 'linear');
%(5)solar radiation
solar_calculate=climate(:,6);
qsolar=interp1(t_climate,solar_calculate,(0:delta_t:t_calculate)', 'linear'); %[\text{W/m}^2]

%preprocessing of the mesh, the initial and the boundary conditions
%general information
%thermal properties of each layers
for i=1:n_layer
    delta_x(i)=th(i)/m(i);
    DcCc(i)=den(i)*hc(i);
    ac(i)=hcond(i)/DcCc(i);
    cov(i)=ac(i)*delta_t/delta_x(i)^2;
end
%Number fo space and time nodes
M=sum(m)+1; % total number of nodes
N=(t_prediction+t_placement)/delta_t+1; % total number of time steps
N_prediction=t_prediction/delta_t+1; % total number of time steps
N_placement=t_placement/delta_t+1; % total number of time steps of curing duration
N_curing=(t_curing+t_placement)/delta_t+1; % total number of time steps of curing duration
%initial condition
T=zeros(M,N); % temperature at node M of time step N
rp=zeros(M,N); % degree of reaction of P reaction in-situ
rs=zeros(M,N);  % degree of reaction of s reaction in-situ
qp=zeros(M,N);  % heat generation rate of P reaction in-situ
qs=zeros(M,N);  % heat generation rate of S reaction in-situ
qc=zeros(M,N);  % total heat generation rate in situ
ah=zeros(M,N);  % degree of hydration in-situ

T(M,1:N)=T_ConstantGround;  % constant ground temperature at any time t
r=zeros(4,N);
b1=zeros(1,N);
b2=zeros(1,N);
b3=zeros(1,N);
% initial temperature profile
Tini=xlsread('Input_E17_6m_slag.xlsx','Initial Temperature');
Tinibase=Tini(:,2);
T_plastic=zeros(1,N);
for i=1:M
    T(i,1)=Tinibase(i);  % initial temperature profile beneath the concrete slab
end
% heat flux coefficients of the plastic sheet surface
a1=(1-reflectivity_s)*transmissivity_s*(1-reflectivity_p+transmissivity_s*reflectivity_p)/(1-reflectivity_p*reflectivity_s);
a2=(1-reflectivity_L)*transmissivity_L*(transmissivity_L+emissivity_p*(1-transmissivity_L))/(1-reflectivity_L+reflectivity_L*emissivity_p);
a3=-emissivity_L*(2*(1-emissivity_p)*(1-transmissivity_L-reflectivity_L)/(1-reflectivity_L+reflectivity_L*emissivity_p));
a4=emissivity_p*(1-(transmissivity_L+reflectivity_L))/(1-reflectivity_L+reflectivity_L*emissivity_p);
% heat flux coefficients of pavement surface
a5=transmissivity_s*(1-reflectivity_p)/(1-reflectivity_p*reflectivity_s);
a6=emissivity_p*transmissivity_L/(1-reflectivity_L+reflectivity_L*emissivity_p);
a7=emissivity_p*emissivity_L/(1-reflectivity_L+reflectivity_L*emissivity_p);
a8=-emissivity_L*emissivity_p/(1-reflectivity_L+reflectivity_L*emissivity_p);
%-----------------------------------------------------------------------------------------------------------------------------

%% Before concrete placement
for j=1:N_placement
    for i=1:M
        if i<=m(1)
            T(i,j)=0;
        elseif i==m(1)+1
            T(i,j)=hconv(j);  % HIPERPAVE II Heat Convection Model
        end
    end
    hconv(j)=3.727*1.79*(0.9*(T(i,j)+Tair(j))+32)^-0.181*(abs(T(i,j)-Tair(j))^0.266)*(1+2.857*wind(j))^0.5;
    % heat convection
    qconv(j)=hconv(j)*(Tair(j)-T(i,j));
    % irradiation-Bentz Model
    epsilon_sky(j)=0.787+0.764.*log((Td(j)+273)/273);  % sky emissivity
    T_sky(j)=epsilon_sky(j)^0.25.*Tair(j);  % T_sky is the effective sky temperature
    qir(j)=emissivity_asphalt*stefan*((T_sky(j)+273)^4-(T(i,j)+273)^4);
    % solar
    qsol(j)=absorptivity_asphalt*qsolar(j);
    % heat flux at the surface
    q(j)=qconv(j)+qir(j)+qsol(j);
    % heat conduction
\[
T(i,j+1) = T(i,j) + 2\cdot h_{\text{cond}}(2)/D_{\text{Cc}}(2) \cdot \delta_x(2)^2 \cdot \frac{(T(i+1,j) - 2\cdot T(i,j) + T(i-1,j))}{\delta_t} \\
+ 2\cdot \frac{\delta_t}{D_{\text{Cc}}(2)/\delta_x(2)} \cdot q(j);
\]

% node in asphalt interlayer surface
\begin{verbatim}
else if \( i = 12 \)
\end{verbatim}
\[
T(i,j+1) = T(i,j) + a_c(2) \cdot \frac{\delta_t}{\delta_x(2)^2} \cdot (T(i+1,j) - 2\cdot T(i,j) + T(i-1,j));
\]

% interface between asphalt interlayer and CTB subbase
\begin{verbatim}
else if \( i = m(2)+m(1)+1 \)
\end{verbatim}
\[
T(i,j+1) = T(i,j) + 2\cdot h_{\text{cond}}(2)/(D_{\text{Cc}}(2)\cdot \delta_x(2) + D_{\text{Cc}}(3)\cdot \delta_x(3)) \cdot \frac{\delta_t}{\delta_x(2)} \cdot (T(i-1,j) - T(i,j)) \\
+ 2\cdot h_{\text{cond}}(3)/(D_{\text{Cc}}(3)\cdot \delta_x(3) + D_{\text{Cc}}(4)\cdot \delta_x(4)) \cdot \frac{\delta_t}{\delta_x(3)} \cdot (T(i+1,j) - T(i,j));
\]

% nodes in CTB subbase
\begin{verbatim}
else if \( i = m(3)+m(2)+m(1)+2 \)
\end{verbatim}
\[
T(i,j+1) = T(i,j) + ac(3) \cdot \frac{\delta_t}{\delta_x(3)^2} \cdot (T(i+1,j) - 2\cdot T(i,j) + T(i-1,j));
\]

% interface between CTB and gravel
\begin{verbatim}
else if \( i = m(3)+m(2)+m(1)+1 \)
\end{verbatim}
\[
T(i,j+1) = T(i,j) + 2\cdot h_{\text{cond}}(3)/(D_{\text{Cc}}(3)\cdot \delta_x(3) + D_{\text{Cc}}(4)\cdot \delta_x(4)) \cdot \frac{\delta_t}{\delta_x(3)} \cdot (T(i-1,j) - T(i,j)) \\
+ 2\cdot h_{\text{cond}}(4)/(D_{\text{Cc}}(4)\cdot \delta_x(4) + D_{\text{Cc}}(5)\cdot \delta_x(5)) \cdot \frac{\delta_t}{\delta_x(4)} \cdot (T(i+1,j) - T(i,j));
\]

% nodes in gravel layer
\begin{verbatim}
else if \( i = m(4)+m(3)+m(2)+m(1)+2 \)
\end{verbatim}
\[
T(i,j+1) = T(i,j) + (rp(i,j)*Qp_{\text{max}} + rs(i,j)*Qs_{\text{max}})/Q_{\text{max}} \cdot ah_u;
\]

% at the bottom of the subgrade
\begin{verbatim}
else
\end{verbatim}
\[
T(M,j+1) = T_{\text{ConstantGround}};
\]

end
end
end % Temperature prediction after concrete placement
% Initial step at the concrete placement
\begin{verbatim}
for i=1:M
\end{verbatim}
\[
j=N_{\text{placement}};
\]

if \( i = m(1)+1 \)
\[
T(i,j) = T_{\text{ConcreteInitial}};
\]

else
\[
T(i,j) = T(i,j);
\]

end
end %
\begin{verbatim}
for i=1:m(1)+1
\end{verbatim}
\[
j=N_{\text{placement}};
\]

rp(i,j) = rp_{\text{ini}};
\[
rs(i,j) = 0;
\]

qp(i,j) = C*qp_{\text{max}} \cdot (c_p \cdot (\sin(rp(i,j)*\pi))^{a_p} \cdot \exp(-b_p*rp(i,j))) \cdot \exp(Ea/R*(1/293-1/(273+T(i,j))));
\[
qs(i,j) = C*qs_{\text{max}} \cdot (\sin(rs(i,j)*\pi))^{a_s} \cdot \exp(\exp(Ea/R*(1/293-1/(273+T(i,j))));
\]

qc(i,j) = qp(i,j) + qs(i,j);
\[
ah(i,j) = (rp(i,j)*Qp_{\text{max}} + rs(i,j)*Qs_{\text{max}})/Q_{\text{max}} \cdot ah_u;
\]

end %
\begin{verbatim}
for i=1:M
\end{verbatim}
\[
j=N_{\text{placement}};
\]

end %

hcond1(i,j) = (1.33 - 0.33 * ah(i,j)) * hcond(1);
if i == 1
    T(i,j) = Tair(j);
else
    hconv(j) = 3.727 * 1.79 * (0.9 * (T(i,j) + Tair(j)) + 32)^0.181 * (abs(T(i,j) - Tair(j)))^0.266;
end

% calculate the temperature of the plastic sheet
hconv0(j) = 3.727 * 1.79 * (0.9 * (T(i,j) + Tair(j)) + 32)^0.181 * (abs(T(i,j) - Tair(j)))^0.266;

epsilon_sky(j) = 0.787 + 0.764 * log((Td(j) + 273) / 273);

% effective sky temperature
T_sky(j) = epsilon_sky(j)^0.25 * Tair(j);

b1(1,j) = a3 * stefan;
b2(1,j) = hconv(j) + hconv0(j);
b3(1,j) = -hconv(j)^0.25 * T(1,j) + Tair(j)^0.25;

c = [b1(1,j) 0 0 b2(1,j) b3(1,j)];

if max(abs(r)) > 0
    T_plastic_real = r;
else
    T_plastic = T(1,j) + 273;
end

% net radiation heat flux at the pavement surface
qsol(j) = a5 * qsolar(j);
qir(j) = a6 * stefan * T^4 + a7 * stefan * T_plastic^4 + a8 * stefan * T^4;
qconv(j) = hconv0(j) * (T_plastic - 273 - T(i,j));
q(j) = qsol(j) + qir(j) + qconv(j);

% heat conduction
T(i,j+1) = T(i,j) + 2 * hcond1(i,j) * factorial(i, delta_x(j))^2 * T(1,j) - T(2,j) / delta_x(j)^2;

appendix i

%interface 5
elseif i==m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond(4)/(DcCc(4)*delta_x(4)+DcCc(5)*delta_x(5))*delta_t/delta_x(4)*(T(i-1,j)-T(i,j)) +2*hcond(5)/(DcCc(4)*delta_x(4)+DcCc(5)*delta_x(5))*delta_t/delta_x(5)*(T(i+1,j)-T(i,j));
else j<=M-1&&i>=m(4)+m(2)+m(1)+2
T(i,j+1)=T(i,j)+ac(5)*delta_t/delta_x(5)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
end
%
till the removal of plastic sheeting
for j=N_placement+1:N_curing
%heat of hydration
for i=1:m(1)+1
rp(i,j)=rp(i-1)+delta_t*qp(i-1)/Qp_max_20/C;
rs(i,j)=rs(i-1)+delta_t*qs(i-1)/Qs_max_20/C;
if rs(i,j)<1
rs(i,j)=rs(i,j);
else
rs(i,j)=1.0;
end
if rp(i,j)<r_pb
rs(i,j)=rs_ini;
qp(i,j)=C*qp_max_20*(c_p*(sin(rp(i,j)*pi))^a_p*exp(-b_p*rp(i,j)))*exp(Ea/R*(1/293-1/(273+T(i,j))));
qs(i,j)=C*qs_max_20*(sin(rs(i,j)*pi))^a_s*exp(Es/R*(1/293-1/(273+T(i,j))));
qc(i,j)=qp(i,j)+qs(i,j); ah(i,j)=(rp(i,j)*Qp_max_20+rs(i,j)*Qs_max_20)/Qmax*ah_u;
else
qp(i,j)=C*qp_max_20*(c_p*(sin(rp(i,j)*pi))^a_p*exp(-b_p*rp(i,j)))*exp(Ea/R*(1/293-1/(273+T(i,j))));
qs(i,j)=C*qs_max_20*(sin(rs(i,j)*pi))^a_s*exp(Es/R*(1/293-1/(273+T(i,j))));
qc(i,j)=qp(i,j)+qs(i,j); ah(i,j)=(rp(i,j)*Qp_max_20+rp(i,j)*Qs_max_20)/Qmax*ah_u;
end
end
%heat conductivity based on degree of hydration
hcond1(i,j)=(1.33-0.33*ah(i,j))*hcond(1);
end
%calculate
for i=1:M
if i==1
%calculate the temperature of the plastic sheet
if T(1,j)>Tair(j)
hconv(j)=3.727*1.79*(0.9*(T(1,j)+Tair(j))+32)^-0.181*(abs(T(1,j)-Tair(j)))*0.266*(1+2.857*wind(j))^{0.5};
hconv0(j)=3.727*0.89*(0.9*(T(1,j)+Tair(j))+32)^-0.181*(abs(T(1,j)-Tair(j)))*0.266;
else
hconv(j)=3.727*0.89*(0.9*(T(1,j)+Tair(j))+32)^-0.181*(abs(T(1,j)-Tair(j)))*0.266*(1+2.857*wind(j))^{0.5};
hconv0(j)=3.727*0.89*(0.9*(T(1,j)+Tair(j))+32)^-0.181*(abs(T(1,j)-Tair(j)))*0.266;
end
Appendix I

% effective sky temperature
epsilon_sky(j)=0.787+0.764.*log((Td(j)+273)/273); % sky emissivity
T_sky(j)=epsilon_sky(j)^0.25.*Tair(j); % T_sky is the effective sky temperature
b1(1,j)=-a3*stefan;
b2(1,j)=hconv(j)+hconv0(j);
b3(1,j)=[-hconv(j)*Tair(j)+273]+hconv0(j)*(T(1,j)+273)+a1*qsolar(j)+a2*stefan*((T_sky(j)+273)).^4...
+a4^4*stefan*(T(1,j)+273).^4];
c=[b1(1,j) 0 0 b2(1,j) b3(1,j)]; % coefficients of heat flux balance equation of the surface of plastic sheet
r=roots(c);
A==real(r);
T_plastic_real=r(A);
for k=1:2
    if T_plastic_real(k)<=T(1,j)+273 && T_plastic_real(k)>=Tair(j)+273
        T_plastic(1,j)=T_plastic_real(k);
    end
end % net radiation heat flux at the pavement surface
qsol(j)=a5*qsolar(j);
qu(j)=a6*stefan*((T_sky(j)+273))^4+a7*stefan*T_plastic(1,j)^4+a8*stefan*(T(1,j)+273)^4;
qconv(j)=hconv0(j)*(T_plastic(1,j)-273-T(1,j));
qu(j)=qsol(j)+qu(j)+qconv(j); % heat conduction
% surface
T(1,j+1)=T(1,j)+2*hcond1(i,j)/DcCc(1)*delta_t/delta_x(1)^2*(T(2,j)-T(1,j))+2*delta_t/DcCc(1)/delta_x(1)*q(j) +delta_t/DcCc(1)*qc(1,j);
elseif i<=m(1)/Branch & i>=2
T(i,j+1)=T(i,j)+hcond1(i,j)/DcCc(1)*delta_t/delta_x(1)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j)) +q(i)*delta_t/DcCc(1);
elseif i=m(1)+1
T(i,j+1)=T(i,j)+2*hcond(2)/(DcCc(2)*delta_x(2)+DcCc(3)*delta_x(3))*delta_t/delta_x(2)*(T(i-1,j)-T(i,j)) +2*hcond(3)/(DcCc(2)*delta_x(2)+DcCc(3)*delta_x(3))*delta_t/delta_x(3)*(T(i+1,j)-T(i,j));
elseif i=m(3)+m(1)+1
T(i,j+1)=T(i,j)+ac(2)*delta_t/delta_x(2)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
elseif i=m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond(3)/(DcCc(2)*delta_x(2)+DcCc(3)*delta_x(3))*delta_t/delta_x(2)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j)) +2*hcond(4)/(DcCc(3)*delta_x(3)+DcCc(4)*delta_x(4))*delta_t/delta_x(3)^2*(T(i+1,j)-T(i,j));
elseif i=m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+ac(4)*delta_t/delta_x(4)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
elseif i=m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond(4)/(DcCc(4)*delta_x(4)+DcCc(5)*delta_x(5))*delta_t/delta_x(4)^2*(T(i+1,j)-T(i,j)) +2*hcond(5)/(DcCc(5)*delta_x(5)+DcCc(6)*delta_x(6))*delta_t/delta_x(5)^2*(T(i+1,j)-T(i,j));
elseif i=m(5)+m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+ac(5)*delta_t/delta_x(5)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
elseif i=m(5)+m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond(5)/(DcCc(5)*delta_x(5)+DcCc(6)*delta_x(6))*delta_t/delta_x(5)^2*(T(i+1,j)-T(i,j)) +2*hcond(6)/(DcCc(6)*delta_x(6)+DcCc(7)*delta_x(7))*delta_t/delta_x(6)^2*(T(i+1,j)-T(i,j));
elseif i>=M-1 & i>=m(6)+m(5)+m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+ac(6)*delta_t/delta_x(6)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
else
\[ T_{(M,j+1)} = T_{\text{ConstantGround}}; \]
\[ \text{end} \]
\[ \text{end} \]
\[ \text{end} \]
\[ \%\text{After the removal of plastic sheeting} \]
\[ \text{for } j = N_{\text{curing}} + 1:N \]
\[ \%\text{heat of hydration} \]
\[ \text{for } i = 1:m(1)+1 \]
\[ rp(i,j) = rp(i,j-1) + \text{delta}_t^*qp(i,j-1)/Qp_{\text{max}}^{20}/C; \]
\[ rs(i,j) = rs(i,j-1) + \text{delta}_t^*qs(i,j-1)/Qs_{\text{max}}^{20}/C; \]
\[ \text{if } rs(i,j) < 1 \]
\[ rs(i,j) = rs(i,j); \]
\[ \text{else} \]
\[ rs(i,j) = 1.0; \]
\[ \text{end} \]
\[ \text{if } rp(i,j) < r_{pb} \]
\[ qp(i,j) = C^*q_{p_{\text{max}}^{20}}^*(c_p*(\sin(rp(i,j)*pi)^a_p*exp(-b_p*rp(i,j)))*exp(Ea/R*(1/293-1/(273+T(i,j))))); \]
\[ qs(i,j) = C^*q_{s_{\text{max}}^{20}}^*(\sin(rs(i,j)*pi)^a_s*exp(Es/R*(1/293-1/(273+T(i,j))))); \]
\[ q(i,j) = qp(i,j) + qs(i,j); \]
\[ ah(i,j) = (rp(i,j)*Qp_{\text{max}}^{20} + rs(i,j)*Qs_{\text{max}}^{20})/Q_{\text{max}}^*ah_u; \]
\[ %Qc(i,1) = C^*Q_{\text{max}}^*ah(i,1); \]
\[ \text{else} \]
\[ \text{if } rs(i,j) == 0 \]
\[ rs(i,j) = rs_{\text{ini}}; \]
\[ \text{end} \]
\[ \text{end} \]
\[ hcond_1(i,j) = (1.33 - 0.33*ah(i,j))^*hcond(1); \]
\[ \text{end} \]
\[ \%\text{calculate temperature} \]
\[ \text{for } i = 1:M \]
\[ \text{if } i == 1 \]
\[ \%\text{convection-HIPERPAVE II Heat Convection Model} \]
\[ \text{if } T(1,j) >= \text{Tair}(j) \]
\[ hconv(j) = 3.727^*1.79*(0.9*(T(1,j)-Tair(j))+32)^{-0.181*(abs(T(1,j)-Tair(j))^{0.266})^*(1+2.857^*\text{wind}(j))^0.5}; \]
\[ \text{else} \]
\[ hconv(j) = 3.727^*0.89*(0.9*(T(1,j)-Tair(j))+32)^{-0.181*(abs(T(1,j)-Tair(j))^{0.266})^*(1+2.857^*\text{wind}(j))^0.5}; \]
\[ \text{end} \]
\[ \%\text{heat convection} \]
\[ \text{qconv}(j) = hconv(j)^*\text{Tair}(j)-T(1,j)); \]
\[ \%\text{irradiation-Bentz Model} \]
\[ \text{epsilon}_s_{\text{sky}}(j) = 0.787 + 0.764^*\text{log}((Tdp(j)+273)/273); \]
\[ \%\text{sky emissivity} \]
\[ T_{\text{sky}}(j)^*\text{epsilon}_s_{\text{sky}}(j)^*0.25^*\text{Tair}(j); \]
\[ \%\text{Tsky is the effective sky temperature} \]
\[ qir(j) = \text{emissivity}_p^*\text{stefan}^*((T_{\text{sky}}(j)+273)^4-(T(1,j)+273)^4); \]
\[ \%\text{solar} \]
qsol(j)=absorptivity_p*qsolar(j);
% heat flux at the surface
q(j)=qconv(j)+qir(j)+qsol(j);
% heat conduction
% surface
T(1,j+1)=T(1,j)+2*hcond1(i,j)/DcCc(1)*delta_t/delta_x(1)^2*(T(2,j)-T(1,j))+2*delta_t/DcCc(1)/delta_x(1)*q(j) +delta_t/DcCc(1)*qc(1,j);
elseif i<=m(1)&&i>=2
T(i,j+1)=T(i,j)+hcond1(i,j)/DcCc(1)*delta_t/delta_x(1)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j)) +2*delta_t/DcCc(1)/delta_x(1)*q(j) +delta_t/DcCc(1)*qc(i,j);
%interface 2
elseif i==m(1)+1
T(i,j+1)=T(i,j)+2*hcond1(i,j)/(DcCc(1)*delta_x(1)+DcCc(2)*delta_x(2))*delta_t/delta_x(1)*((T(i-1,j)-T(i,j))
+2*hcond2)/(DcCc(1)*delta_x(1)+DcCc(2)*delta_x(2))*delta_t/delta_x(2)*(T(i+1,j)-T(i,j))
+delta_x(1)*delta_t*qc(i,j)/(DcCc(1)*delta_x(1)+DcCc(2)*delta_x(2));
elseif i==m(2)+m(1)
T(i,j+1)=T(i,j)+ac(2)*delta_t/delta_x(2)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
%interface 3
elseif i==m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond2/(DcCc(2)*delta_x(2)+DcCc(3)*delta_x(3))*delta_t/delta_x(2)*(T(i-1,j)-T(i,j))
+2*hcond3)/(DcCc(2)*delta_x(2)+DcCc(3)*delta_x(3))*delta_t/delta_x(3)*(T(i+1,j)-T(i,j))
+delta_x(2)*delta_t*qc(i,j)/(DcCc(2)*delta_x(2)+DcCc(3)*delta_x(3));
elseif i<=m(3)+m(2)+m(1)&&i>=m(2)+m(1)+2
T(i,j+1)=T(i,j)+ac(3)*delta_t/delta_x(3)^2*(T(i+1,j)-2*T(i,j)+T(i-1,j));
%interface 4
elseif i==m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond3)/(DcCc(3)*delta_x(3)+DcCc(4)*delta_x(4))*delta_t/delta_x(3)*(T(i-1,j)-T(i,j))
+2*hcond4)/(DcCc(3)*delta_x(3)+DcCc(4)*delta_x(4))*delta_t/delta_x(4)*(T(i+1,j)-T(i,j))
+delta_x(3)*delta_t*qc(i,j)/(DcCc(3)*delta_x(3)+DcCc(4)*delta_x(4));
elseif i<=m(4)+m(3)+m(2)+m(1)&&i>=m(3)+m(2)+m(1)+2
T(i,j+1)=T(i,j)+ac(4)*delta_t/delta_x(4)^2*(T(i-1,j)-2*T(i,j)+T(i-1,j));
%interface 5
elseif i==m(4)+m(3)+m(2)+m(1)+1
T(i,j+1)=T(i,j)+2*hcond4)/(DcCc(4)*delta_x(4)+DcCc(5)*delta_x(5))*delta_t/delta_x(4)*(T(i-1,j)-T(i,j))
+2*hcond5)/(DcCc(4)*delta_x(4)+DcCc(5)*delta_x(5))*delta_t/delta_x(5)*(T(i+1,j)-T(i,j))
+delta_x(4)*delta_t*qc(i,j)/(DcCc(4)*delta_x(4)+DcCc(5)*delta_x(5));
elseif i<=m-1&&i>=m(4)+m(3)+m(2)+m(1)+2
T(i,j+1)=T(i,j)+ac(5)*delta_t/delta_x(5)^2*(T(i-1,j)-2*T(i,j)+T(i-1,j));
%bottom
else
T(M,j+1)=T_ConstantGround;
end
end
end
Part C

In this part, a Matlab code designed for the prediction of the early age concrete stress development in CRCP considering the time-dependent relaxation (Van Breugel relaxation model for early age concrete) and shrinkage (estimated by Eurocode 2 shrinkage model) is presented. The stress and stiffness of the concrete are assumed to start developing at the critical degree of hydration which is estimated by the model by Schindler and depends on the water to cement ratio. The stiffness and tensile strength of concrete are determined through laboratory tests and described as a function of the degree of hydration. The details of the calculation are presented in Chapter 6.

```matlab
% mean compressive strength at 28 days
fck=50;
% coefficient s in the determination of drying shrinkage
alpha_ds1=4;
% coefficient s in the determination of drying shrinkage
alpha_ds2=0.12;
% ambient relative humidity
RH=65;
% correction factor depending on the notional size of the pavement slab
th_notional=250;
% correction factors considering the ambient relative humidity
beta_RH=1.55*(1-(RH/100)^3);
% ultimate drying shrinkage
epsilon_cd_0=0.85*((220+110*alpha_ds1)*exp(-alpha_ds2*fck/10))*0.000001*beta_RH*Kh;
% ultimate autogenous shrinkage
epsilon_ca_ul=2.5*(fck-10)*0.000001;
% Total shrinkage
for i=1:11
    for j=1:N_prediction
        epsilon_total(i,j)=(1-exp(-0.2*(j*delta_t/86400)^0.5))*epsilon_ca_ul;
    else
        epsilon_total(i,j)=(1-exp(-0.2*(j*delta_t/86400)^0.5))*epsilon_ca_ul+((j*delta_t/86400-t_curing/86400)... /
                 ((j*delta_t/86400-t_curing/86400)+0.04*(th_notional^3)^0.5))*epsilon_cd_0;
    end
end
end
% calculation starting from the time of concrete placement
for i=1:11
    for j=1:N_prediction
        TS(i,j)=T(i,j+N_placement-1);
        ahS(i,j)=ah(i,j+N_placement-1);
    end
end
% time dependent concrete tensile strength and elastic modulus,
ft=zeros(11,N_prediction);  % 11 is the number of concrete slab layers
Ec=zeros(11,N_prediction);  % 11 is the number of concrete slab layers
for i=1:11
    for j=1:N_prediction
        if ahS(i,j)<=ah_critical
            ft(i,j)=0;
            Ec(i,j)=0;
        else
            ft(i,j)=ft_ultimate.*((ahS(i,j)-ah_critical)./(ah_u-ah_critical))^Coe_ft;
            Ec(i,j)=Ec_ultimate.*((ahS(i,j)-ah_critical)./(ah_u-ah_critical))^Coe_Ec;
        end
    end
end
```
% Total thermal stress without relaxation
StressN=zeros(11,N_prediction);
DStressN=zeros(11,N_prediction);
for i=1:11
    for j=1:N_prediction-1
        if ahS(i,j)<=ah_critical
            DStressN(i,j)=0;
        else
            DStressN(i,j)=Ec(i,j).*a_COTE.*(TS(i,j)-TS(i,j+1))+(epsilon_total(i,j)-epsilon_total(i,j+1))/(1-v);
        end
    end
end
% Total thermal stress with relaxation
% 1. Relaxation model for hardening concrete proposed by van Breugel
Relaxation=zeros(N_prediction,N_prediction,11);
DStress=zeros(11,N_prediction);
for q=1:11
    for i=1:N_prediction
        for j=i:N_prediction
            if ahS(q,j)<=ah_critical
                Relaxation(i,j)=1;
            else
                Relaxation(i,j,q)=exp(-((ahS(q,j)/ahS(q,i)-1)+1.34*(0.42^1.65).*(((j-i)/20).^(0.3).*((j-i)/20).^0.3))).%van Bruegel Relaxation Model
            end
        end
    end
end
% 2. Thermal stress increment for each step due to thermal strain change
for i=1:11
    for j=1:N_prediction-1
        if ahS(i,j)<=ah_critical
            DStress(i,j)=0;
        else
            DStress(i,j)=Ec(i,j).*a_COTE.*(TS(i,j)-TS(i,j+1))/(1-v);
        end
    end
end
% 3. Total thermal stress at time t
Stress=zeros(11,N_prediction);
for q=1:11
    for j=1:N_prediction
        for i=1:j
            if ahS(q,j)<=ah_critical
                dStress(i,j,q)=0;
            else
                dStress(i,j,q)=DStress(q,i).*Relaxation(i,j,q);
            end
        end
    end
end
In this part, the stress intensity factors are determined by the weight function method. Parameters $M_{iA}$ and $M_{iB}$ for a corner quarter elliptical surface crack in a finite slab are calculated by the following equations for the reference stress distributions.

$M_{iA}$ and $M_{iB}$ in weight function $m_A(z, a)$ and $m_B(z, a)$ in Equation (15) can be derived by using known stress intensity factors for the reference stress distributions in handbook. The stress intensity factors for quarter-elliptical corner crack under the uniform stress field $\sigma(z) = \sigma_0$ and the linearly decreasing stress field $\sigma(z) = \sigma_0(1 - \frac{z}{a})$ have normalized as follows (Glinka, et.al, 1991):

For the deepest point in the profile plane, A

$$K_0^A = \sigma_0 \sqrt{\frac{ma}{Q}F_0^A}$$ uniform stress field $\sigma(z) = \sigma_0$ \hspace{1cm} (A-1a)

$$K_1^A = \sigma_0 \sqrt{\frac{ma}{Q}F_1^A}$$ linearly decreasing stress field $\sigma(z) = \sigma_0(1 - \frac{z}{a})$ \hspace{1cm} (A-1b)

For the surface point in the frontal plane, B

$$K_0^B = \sigma_0 \sqrt{\frac{ma}{Q}F_0^B}$$ uniform stress field $\sigma(z) = \sigma_0$ \hspace{1cm} (A-1c)

$$K_1^B = \sigma_0 \sqrt{\frac{ma}{Q}F_1^B}$$ linearly decreasing stress field $\sigma(z) = \sigma_0(1 - \frac{z}{a})$ \hspace{1cm} (A-1d)

Where, $F$ is the geometry correction factor, which depends on the geometry and location of the crack, and on the stress distribution in the uncracked slab. $Q$ is the shape factor of an ellipse and is given by the square of the complete elliptic integral of the second kind. The following empirical equation for $Q$ is used:

$$Q = 1 + 1.464(a/c)^2 \quad \text{for} \quad 0 \leq a/c \leq 1 \hspace{1cm} (A-2)$$
Substitution of Equation (A-1) into Equation (16) results in four independent equations containing parameter $M_{iA}$ and $M_{iB}$.

$$K_0^A = \sigma_0 \frac{\pi a}{Q} F_0^A$$

$$= \int_0^a \frac{2\sigma_0}{\sqrt{2\pi (a-z)}} \left[ 1 + M_{1A} \left( 1 - \frac{z}{a} \right)^{1/2} + M_{2A} \left( 1 - \frac{z}{a} \right) + M_{3A} \left( 1 - \frac{z}{a} \right)^{3/2} \right] \, dz \quad (A-3a)$$

$$K_1^A = \sigma_0 \frac{\pi a}{Q} F_1^A$$

$$= \int_0^a \frac{2\sigma_0}{\sqrt{2\pi (a-z)}} \left[ 1 + M_{1A} \left( 1 - \frac{z}{a} \right)^{1/2} + M_{2A} \left( 1 - \frac{z}{a} \right) + M_{3A} \left( 1 - \frac{z}{a} \right)^{3/2} \right] \, dz \quad (A-3b)$$

$$K_0^B = \sigma_0 \frac{\pi a}{Q} F_0^B$$

$$= \int_0^a \frac{2\sigma_0}{\sqrt{2\pi (a-z)}} \left[ 1 + M_{1B} \left( \frac{z}{a} \right)^{1/2} + M_{2B} \left( \frac{z}{a} \right) + M_{3B} \left( \frac{z}{a} \right)^{3/2} \right] \, dz \quad (A-3c)$$

$$K_1^B = \sigma_0 \frac{\pi a}{Q} F_1^B$$

$$= \int_0^a \frac{2\sigma_0}{\sqrt{2\pi (a-z)}} \left[ 1 + M_{1B} \left( \frac{z}{a} \right)^{1/2} + M_{2B} \left( \frac{z}{a} \right) + M_{3B} \left( \frac{z}{a} \right)^{3/2} \right] \, dz \quad (A-3d)$$

Six independent equations are required to calculate parameters $M_{iA}$ and $M_{iB}$, two additional equation can be formulated by satisfying the conditions that crack surface curvature at the crack center is zero and the weight function for surface point B must vanish for $z = a$. It can be written in the following form:

$$\left. \frac{\partial^2 m_A(x,a)}{\partial x^2} \right|_{x=0} = 0 \quad (A-3e)$$

$$1 + M_{1B} + M_{2B} + M_{3B} = 0 \quad (A-3f)$$

Therefore, the six unknown parameters $M_{iA}$ and $M_{iB}$ can be determined by solving equation 16.

$$M_{1A} = \frac{2\pi}{\sqrt{2Q}} \left( 2F_0^A - 3F_1^A \right) - \frac{24}{5} \quad (A-4a)$$

$$M_{2A} = 3 \quad (A-4b)$$

$$M_{3A} = \frac{6\pi}{\sqrt{2Q}} \left( -F_0^A + 5F_1^A \right) + \frac{8}{5} \quad (A-4c)$$

$$M_{1B} = \frac{3\pi}{\sqrt{Q}} \left( -3F_0^B + 5F_1^B \right) - 8 \quad (A-4d)$$

$$M_{1A} = \frac{15\pi}{\sqrt{Q}} \left( 2F_0^B - 3F_1^B \right) + 15 \quad (A-4e)$$

$$M_{1A} = \frac{3\pi}{\sqrt{Q}} \left( -7F_0^B + 10F_1^B \right) - 8 \quad (A-4f)$$
X. Zhang and G. Glinka (1996) proposed closed formed expressions of geometry correction factors for a large range of relative depth \((0.1 \leq a/h \leq 1)\) and aspect ratios \((0.2 \leq a/c \leq 0.8)\) based on the tabular form from Shiratori and Misoshi (1992) with an accuracy better than 1.5%. it should be noted that the data of Shiratori and Misoshi was obtained by finite element method and the accuracy is with a few percent.

\[
F_0^A = A_{00} + A_{01}(a/h) + A_{02}(a/h)^2 + A_{03}(a/h)^3 + A_{04}(a/h)^4
\]

Where:
\[
A_{00} = 1.0141 + 0.016(a/c) + 0.186(a/c)^2 - 0.111(a/c)^3
\]
\[
A_{01} = -0.599 + 1.953(a/c) - 1.310 (a/c)^2 - 0.028(a/c)^3
\]
\[
A_{02} = 4.927 - 13.216(a/c) + 6.747 (a/c)^2 + 1.918(a/c)^3
\]
\[
A_{03} = -1.293 + 1.857(a/c) + 12.906 (a/c)^2 - 13.441(a/c)^3
\]
\[
A_{04} = -0.572 + 3.073(a/c) - 10.797 (a/c)^2 + 8.393(a/c)^3
\]

\[
F_1^A = A_{10} + A_{11}(a/h) + A_{12}(a/h)^2 + A_{13}(a/h)^3 + A_{14}(a/h)^4
\]

Where:
\[
A_{10} = 0.500 - 0.323(a/c) + 0.213(a/c)^2 - 0.052(a/c)^3
\]
\[
A_{11} = -0.507 + 1.373(a/c) - 0.740 (a/c)^2 - 0.184(a/c)^3
\]
\[
A_{12} = 3.468 - 9.028(a/c) + 6.349 (a/c)^2 - 0.135(a/c)^3
\]
\[
A_{13} = -1.359 + 1.731(a/c) + 5.357 (a/c)^2 - 6.370(a/c)^3
\]
\[
A_{14} = -0.162 + 2.977(a/c) - 8.250 (a/c)^2 + 5.804(a/c)^3
\]

\[
F_0^B = [B_{00} + B_{01}(a/h) + B_{02}(a/h)^2 + B_{03}(a/h)^3 + B_{04}(a/h)^4] a/c
\]

Where:
\[
B_{00} = 3.340 - 4.495(a/c) + 3.016 (a/c)^2 - 0.7278(a/c)^3
\]
\[
B_{01} = 0.2318 - 0.2261(a/c) - 1.658 (a/c)^2 + 1.504(a/c)^3
\]
\[
B_{02} = 22.95 - 100.9(a/c) + 152.2 (a/c)^2 - 72.92(a/c)^3
\]
\[
B_{03} = -39.16 + 194.1(a/c) - 302.0 (a/c)^2 + 145.9(a/c)^3
\]
\[
B_{04} = 30.80 - 142.9(a/c) + 212.6 (a/c)^2 - 99.92(a/c)^3
\]

\[
F_1^B = [B_{10} + B_{11}(a/t) + B_{12}(a/t)^2 + B_{13}(a/t)^3 + B_{14}(a/t)^4] a/c
\]

Where:
\[
B_{10} = 2.831 - 3.840(a/c) + 2.477 (a/c)^2 - 0.511(a/c)^3
\]
\[
B_{11} = 4.600 - 20.498(a/c) + 29.001 (a/c)^2 - 13.226(a/c)^3
\]
\[
B_{12} = -4.019 + 15.057(a/c) - 12.624 (a/c)^2 + 2.677(a/c)^3
\]
\[
B_{13} = 9.682 - 15.932(a/c) - 8.848 (a/c)^2 + 13.910(a/c)^3
\]
\[
B_{14} = -1.141 - 9.176(a/c) + 30.228 (a/c)^2 - 19.195(a/c)^3
\]
Curriculum Vitae

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Publications


