Early-age behavior of continuously reinforced concrete pavements (CRCP)

Freek Speleman
Student number: 01300156

Supervisors: Prof. dr. ir. Hans De Backer, Prof. ir. Pieter De Winne

Master's dissertation submitted in order to obtain the academic degree of
Master of Science in Civil Engineering

Academic year 2018-2019
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Freek Speleman

31th May 2019
Acknowledgements

This Master Thesis is the result of two semesters of research done at Ghent University and can be seen as the icing on the cake of my engineering education at this university. In this preface, I would like to thank the people who made this possible and who helped in obtaining the academic degree of Master of Science in Civil Engineering.

First of all, I would like to thank prof. dr. ir. Hans De Backer for making this subject an available option to conduct research into. I developed interest in this subject through the years, during courses about concrete technology and advanced road engineering. The subject of roads was one of the topics that appealed to me the most during the education and within this subject, CRCP seemed challenging to investigate and not yet explored to the last detail. I would also like to thank him along with prof. ir. Pieter De Winne for providing new insights during the meetings and for showing the best direction to head to during the further research and the modelling.

I would also like to thank ir. Muhammad Kashif for being my mentor during the research and for being very helpful. He provided a wealth of information and was always ready to answer any question on the subject. I wish him the best of luck with obtaining his PhD and with the continuation of his career.

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To end, I would like to thank my parents for giving me the chance to do this studies at Ghent University. During these years, they supported me financially and mentally. Furthermore, I would like to thank my brothers, sister, family and friends for giving me a perfect environment at home and at school to bring this engineering education to a successful conclusion.

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Ghent University
Faculty of Engineering and Architecture
Department of Civil Engineering
Chairman: Prof dr. ir. Peter Troch

Supervisors
Prof. dr. ir. Hans De Backer
Prof. ir. Pieter De Winne

Abstract

Continuously Reinforced Concrete Pavement (CRCP) is a type of concrete roads that is frequently used in Belgium, especially on roads with heavy traffic, as the long-term performance is high and the maintenance is low. In CRCP, free cracking is allowed but this crack pattern is controlled by the continuously longitudinal reinforcement. However, this crack pattern is often not as desired and is often responsible for distresses such as punchouts. Therefore, active crack control is implemented by applying surface notches where the cracks will initiate by means of cutting. In this research, the early-age behavior of CRCP is investigated. More specifically, the evolution of the temperature, stresses and cracks throughout a CRCP section in the first few days is examined. The influence of active crack control on this early-age behavior is investigated, as well as the influence of some important aspects in the field of CRCP, such as the boundary conditions in the sense of external air temperatures and the convection coefficient of the concrete. Also the influence of bondslip modelling between the reinforcement and the concrete will be examined and compared to the case of regular embedded reinforcement. To this end, a 3D finite element (FE) model is made using the FE software program DIANA. In this software program, a CRCP section was modelled staying reasonably true to the reality. A clear influence of both the ambient temperature and the concrete convection coefficient was discovered and also the modelling of bondslip showed some clear effects. However, it is desired to conduct a testing program to validate the results obtained by the finite element modelling.

Keywords: continuously reinforced concrete pavement, finite element modelling, active crack control, early-age behavior, tensile stresses
Early-age behavior of continuously reinforced concrete pavements (CRCP)

Freek Speleman

Supervisors: Hans De Backer, Pieter De Winne

Abstract Continuously Reinforced Concrete Pavement (CRCP) is a type of concrete roads that is used more and more throughout the last few decades. In CRCP, free cracking is allowed but this crack pattern is controlled by the continuously longitudinal reinforcement. However, this crack pattern is often not as desired and is often responsible for distresses such as punchouts. Therefore, active crack control is implemented by applying surface notches where the cracks will initiate by means of cutting. In this research, the early-age behavior of CRCP is investigated. More specifically, the evolution of the temperature, stresses and cracks throughout a CRCP section in the first few days is examined, in order to study the influence of some important aspects in the field of CRCP, such as the boundary conditions in the sense of external air temperatures and the convection coefficient of the concrete, or the modelling of active crack control and bondslip between the concrete and the longitudinal reinforcement. To this end, a 3D finite element model is made using the software program DIANA.

Keywords continuously reinforced concrete pavement, finite element modelling, active crack control, early-age behavior, tensile stresses

I. INTRODUCTION

A. Basics of CRCP

Continuously reinforced concrete pavement (CRCP) is an excellent pavement solution for heavily loaded roads with a long life performance and minimum maintenance. Continuous longitudinal steel reinforcement is provided without any transverse joints except the construction joints where required for the construction purposes, in order to control the longitudinal shrinkage of concrete. It overcomes the defects caused by transverse joints in a common concrete pavement, improves the stability of vehicles, and increases overall capacity of pavement. The change in temperature and moisture conditions causes volumetric changes in the concrete, leading to the formation of cracks. Those cracks appear in the form of a random crack pattern all over the width of pavement which lead to structural distress in the pavement such as punchouts. These early age cracks are held tightly closed by the longitudinal reinforcement.

A technique to control the crack pattern, is active crack control, also called induced cracking. It was found that this technique achieved transverse cracks sooner and straighter and at a more regular interval. The technique can thus reduce the probability of a non-uniform crack pattern and even prevent punchouts.

B. History of CRCP in Belgium

CRCP is used in Belgium on a large scale for more than forty years. It was first applied in the 1950s on some experimental sites, but it was only in the late 1960s that the technique was becoming more largely applied and in 1970, the first large-scale construction was started. Along the years since then, there have been used different concepts, which were called Concept 1, Concept 2 and Concept 3.

In Table 1, an overview is given of the characteristics of the current design concept [1]. In the models used in this research, these characteristics will be used. This will be reflected in the slab thickness, the reinforcement depth and the steel lay-out of the longitudinal and the transverse reinforcement.

Table 1 Overview of the CRCP characteristics for Design Concept 3

<table>
<thead>
<tr>
<th>Design Concept 3</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Period</td>
<td>2010 – current</td>
</tr>
<tr>
<td>Slab thickness</td>
<td>25 cm</td>
</tr>
<tr>
<td>Reinforcement depth</td>
<td>8 cm</td>
</tr>
<tr>
<td>Interlayer</td>
<td>Asphalt interlayer</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>Exposed aggregate</td>
</tr>
<tr>
<td>Longitudinal reinforcement</td>
<td>0.75 %</td>
</tr>
<tr>
<td>Longitudinal bars</td>
<td>ø 20 mm every 18 cm</td>
</tr>
<tr>
<td>Transverse bars</td>
<td>ø 12 mm every 70 cm</td>
</tr>
<tr>
<td>Air-entraining agent</td>
<td>Yes</td>
</tr>
</tbody>
</table>

II. METHODOLOGY

A. Finite element method

The finite element method (FEM) is a numerical method used for solving engineering or mathematical problems. These problems are solved by dividing a large system into a finite amount of smaller, simpler parts: the finite elements. The equations used to model these finite elements are then put together into a large system of equations describing the entire problem. The division into smaller parts has several advantages: a complex geometry can be represented accurately, different material properties can be included, the total solution can be represented easily and the local effects are considered and can be investigated. The modelling of the CRCP section in DIANA in order to perform the FE analysis is done by consecutively defining the geometry, the materials and their properties, the structural constraints, the temperature profile and the mesh. After the modelling of the CRCP section, the results can be computed by defining the settings of the analysis.

F. Speleman is a student at the Civil Engineering Department, Ghent University (UGent), Ghent, Belgium. E-mail: Freek.Speleman@UGent.be.
B. Software description

As mentioned before, the software used for this research is DIANA (DIspacement ANAlyzer). It is a finite element code, based on the displacement method, which is under development since 1972. It is a program with extensive material, element and procedure libraries and pre- and postprocessors to define the model and to view the results in a graphical interactive environment. [2] The most extended capabilities of DIANA lay in the field of (reinforced) concrete and soil. A wide range of material aspects can be considered, such as cracking of concrete, yielding of steel, creep and shrinkage or aging. For the casting of the concrete, heat generation and maturing of young hardening concrete can be simulated.

III. MODELLING

In this section, all the different parameters and characteristics which should be defined in DIANA are being discussed.

A. Geometry

The geometry of the model is based on the design Concept 3 for CRCP in Belgium. Its characteristics and dimensions were listed in Table 1 and these will be used in the model. The width is modelled to be 1.8 m and the length 2.4 m for symmetry reasons. In Figure 1, the geometry as modelled in DIANA can be seen for the models including active crack control.

![Figure 1 Model of the CRCP section](image)

B. Concrete material properties

When defining the concrete material properties, first all the aspects must be selected which have to be included. These aspects are the following: total strain crack model, shrinkage, creep, heat flow and young hardening concrete. In this section, the required parameters are listed.

1) Model code

To model the concrete, it is chosen to use a concrete model code from the library, for which some parameters are fixed according to the specific code. In this model, the Eurocode 2 EN 1992-1-1 is selected, as it is the main code used in Flanders. The Eurocode 2 EN 1992-1-1 model code gives both the characteristic cylinder-compressive strength $f_{ck}$ and the characteristic cube-compressive strength $f_{cub}$. Both values are part of the name of the normal weight concrete class. A normal weight concrete is selected, along with a concrete class of C45/55, which means that $f_{ck} = 45$ MPa and $f_{cub} = 55$ MPa. DIANA uses this input to set the stress–strain diagram in the compressive and tensile regime.

The aggregate type Sandstone is selected and the cement type Class N is chosen, which is the normal cement type used in concrete roads. In Table 2, the parameters can be seen which are fixed in DIANA for the defined concrete type.

Table 2 Fixed parameters in DIANA for the chosen concrete type

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus $E_{cm}$</td>
<td>25,3982 GPa</td>
</tr>
<tr>
<td>Poisson ratio $\nu$</td>
<td>0.2</td>
</tr>
<tr>
<td>Thermal expansion coefficient $a_t$</td>
<td>$1 \times 10^{-6}$/°C</td>
</tr>
<tr>
<td>Density $\rho$</td>
<td>2400 kg/m³</td>
</tr>
<tr>
<td>Mean uniaxial tensile strength $f_{um}$</td>
<td>3.79545 MPa</td>
</tr>
<tr>
<td>Mean compressive strength $f_{cm}$</td>
<td>53 MPa</td>
</tr>
</tbody>
</table>

2) Creep and shrinkage

Creep and shrinkage are both physical properties of concrete which are related to a change in shape or in volume of the concrete structure. In DIANA, these two properties can be selected separately to be taken into account in the analysis, but the parameters that are needed in DIANA are in the same window as creep during drying and shrinkage are inseparable. The parameters that are required for definition of creep and shrinkage in the concrete, are the following: ambient temperature, nominal size of the member and relative ambient humidity and the concrete age at the end of the curing period.

The ambient temperature is different for each different temperature profile. When a constant temperature profile is applied, then this constant temperature will be applied as ambient temperature. When a realistic temperature profile is applied, then the average temperature throughout this profile will be applied as the ambient temperature. To give more detail to the environmental conditions, also the ambient humidity can be defined. It is chosen to leave this at the default value of 80 %. The concrete age at the end of the curing period is one day: this is the time after which the sheeting will be removed from the concrete. The sheeting is done in order to prevent drying out of the fresh concrete. The nominal size of the member is equal to $2A_c/u$, where $A_c$ is the cross-sectional area of the concrete and $u$ is the perimeter in contact with the atmosphere. It is found that this nominal size is equal to 0.439 m.

3) Heat flow

In order to model the heat flow, the following data are required: the heat conductivity of the concrete, the heat capacity of the concrete and the definition of the hydration heat via an adiabatic curve.

The value that is applied for the heat conductivity is 3 W/m°C, as found in the PhD thesis from D. Ren [1]. For the specific heat capacity, in this same source a value of 1000 J/kg°C is found. Since the concrete used in this model has a density of 2400 kg/m³, the heat capacity that should be applied is $2.4 \times 10^6$ J/m³°C.

A very important aspect in the field of the early-age behavior of CRCP is the heat development due to hydration. This is the heat generated when water and cement react. In DIANA, the heat of hydration must be modelled by defining the adiabatic heat curve. This curve gives the adiabatic heat development in function of time, and is an increasing function. The most heat is produced in the first few days, after which the heat production goes to zero, which means that the adiabatic curve will converge to a maximum value after a few days. In Graph 1, the adiabatic heat curve can be seen for concrete...
using cement type CEM III/A 42.5, which is the relevant cement type in this research, according to De Schutter [3].

![Concrete with CEM III/A 42.5 Graph]

Graph 1 Adiabatic temperature rise in function of time of concrete with CEM III/A 42.5

Along with the adiabatic heat development goes the definition of the initial temperature of the concrete. This is set to be 20 °C, as this is the initial temperature seen on the graph.

C. Steel material properties

In Table 3, the steel properties can be seen as defined in DIANA. The steel that is used is BE 500 S. It is chosen to model no plastic hardening for the steel. This is decided because the yield strength will definitely never be reached for this type of application.

Table 3 Steel properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>200 GPa</td>
</tr>
<tr>
<td>Yield strength</td>
<td>500 MPa</td>
</tr>
<tr>
<td>Mass density</td>
<td>7850 kg/m³</td>
</tr>
</tbody>
</table>

D. Convection

Convection is the heat transfer due to the movement of molecules within fluids such as gases or liquids. In this case, the fluid is the ambient air. The heat transfer takes place because the heated fluid is forced to move away from the heat source, carrying energy with it.

A heat transfer coefficient of 6 W/m²°C is applied for the first part. In the second part, where the influence of the convection coefficient will be studied, this parameter will be raised to respectively 8 W/m²°C and 10 W/m²°C. In order to prevent drying out of the fresh concrete, a sheet is applied after concrete placement. This is modelled by setting the heat transfer coefficient to 0 W/m²°C during this period of sheeting. This is a simplification, as also the sheet has a heat transfer coefficient. However, this value is very small compared with the value 6 W/m²°C, so the simplification is justified. As mentioned before, it is chosen to apply a curing period of only one day.

E. Loads

The only loads that are modelled for this research, are the self-weight of the model and the environmental temperature conditions as an external loading. For the temperature profiles, it is chosen to use two constant temperature profiles of 17°C and -17°C in order to cover some extreme circumstances, as well as the profiles from four months in order to cover several parts of the year and thus several temperature conditions: the temperature profiles from January, April, July and October of the year 2018 in Brussels are applied.

F. Supports

In DIANA, a fixed rotation and/or translation in one or more directions can be assigned to a face of the slab. This can be done by using the support settings. In Table 4, an overview can be seen of the faces which have an imposed fixed translation, along with the direction of this fixed translation and the reasoning behind it.

Table 4 Settings for the supports in DIANA

<table>
<thead>
<tr>
<th>Faces</th>
<th>Fixed translation</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom face</td>
<td>Z-direction</td>
<td>The slab is supported at the bottom by the base</td>
</tr>
<tr>
<td>One of the two transverse side</td>
<td>Y-direction</td>
<td>For the transverse continuity: symmetry</td>
</tr>
<tr>
<td>faces</td>
<td>(horizontal:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>transverse)</td>
<td></td>
</tr>
<tr>
<td>The two longitudinal side</td>
<td>X-direction</td>
<td>For the longitudinal continuity</td>
</tr>
<tr>
<td>faces</td>
<td>(horizontal:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>longitudinal)</td>
<td></td>
</tr>
</tbody>
</table>

G. Bondslip

In order to model bondslip between the concrete and the reinforcement bars, a multilinear graph is inserted where the friction stress is represented in function of the slip. This is shown in Graph 2 [4].

![Friction stress-slip diagram Graph]

Graph 2 Friction stress-slip diagram to model bondslip

In addition to the friction stress-slip diagram, DIANA also demands some values for certain relations: the shear stiffness DSSX and the normal stiffness DSNY, separately for the longitudinal and for the transverse reinforcement bars. These values are defined according to M. Eriksen [5] and can be seen in Table 5.

Table 5 Values for DSSX and DSNY

<table>
<thead>
<tr>
<th></th>
<th>DSSX [N/mm²]</th>
<th>DSNY [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal</td>
<td>47.80</td>
<td>1269910.00</td>
</tr>
<tr>
<td>Transverse</td>
<td>47.80</td>
<td>2116516.67</td>
</tr>
</tbody>
</table>

H. Mesh

For the generation of the mesh, some settings can be defined. For the default mesher type, quadratic/hexagonal is selected and for the default mesher order, quadratic is selected, along with the option of linear interpolation for the
definition of the mid-side location. For the case without active crack control, a desired element size of 100 mm is defined, while for the case with active crack control, a desired element size of 90 mm is defined. For the case with active crack control, the mesh can be seen in Figure 2.

Figure 2 Mesh for the model with active crack control

I. Analysis settings

To obtain the results, two analyses will be performed: a transient heat transfer analysis and a structural nonlinear analysis. In DIANA, several settings can be defined, such as the maximum number of iterations, the convergence tolerance, the solution method or the iteration method. The solution method is set to be the parallel direct sparse and the iteration method the Regular Newton-Raphson method. For the structural nonlinear analysis, the option of line search is selected. The maximum number of iterations is defined to be 100 and the convergence tolerance 0.01. Also the time steps for which DIANA calculates the model must be defined. These can be seen in Table 6.

Table 6 Time steps used in the analysis

<table>
<thead>
<tr>
<th>Time step [day]</th>
<th>Number of steps</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>40</td>
</tr>
<tr>
<td>0.2</td>
<td>10</td>
</tr>
<tr>
<td>0.5</td>
<td>12</td>
</tr>
<tr>
<td>1</td>
<td>11</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
</tr>
</tbody>
</table>

It can be seen that the total duration of the period which will be calculated is 31 days, which is exactly the duration of one month. For the temperature profile of April, having a duration of 30 days, the last time step of 2 days will be replaced with a time step of 1 day, so that the total duration corresponds to this month.

IV. RESULTS AND DISCUSSION

In this chapter, the results as calculated by DIANA will be investigated and discussed. This will be done by studying the stresses in the concrete and in the reinforcement steel, the crack pattern and the crack strains and the temperature distribution. Some of these results will be highlighted and discussed more in detail. The analysis will be split into three parts, with each part examining the influence of one particular aspect.

The results of the temperatures are obtained by performing a transient heat transfer analysis in DIANA, while all the other results concerning crack strain, crack width, stress or tensile strength, are obtained by performing a structural nonlinear analysis.

A. Part I – active crack control and temperature

In the first part, the influence of a different temperature profile will be checked. In total, six different temperature profiles will be considered as mentioned before. All these profiles will be checked with and without the modelling of active crack control, so that the influence of active crack control can be investigated.

1) No active crack control

To check the influence of the temperature profile, first the temperature distribution throughout the depth of the slab is examined. This is done for the middle of the slab for the temperature profile of 17°C and can be seen in Graph 3.

Graph 3 Temperature along the depth for different time steps – 17°C

In this graph, the development of the temperature distribution is shown just after the removal of the sheeting. It can be seen that in the first hours after removal of the sheeting, the temperature in the lower part of the slab still increases, while the temperature at the surface starts to decrease immediately. The reason for this is that the influence of the adiabatic heat development is still present in the lower parts of the slab. It can be said that the ambient temperature has not fully permeated through the slab. However, when also the lower part of the slab begins to feel the influence of the ambient temperature, the temperature throughout the whole depth starts to decrease towards 17°C.

In Graph 4, the temperature in the middle of the slab on the surface can be seen in function of time for two temperature profiles. In this graph, the influence of the temperature profile is clear. Where the surface temperature for the temperature profile of 17°C converges towards 17°C, keeps the temperature in January adapting to the external temperature. Also the difference in temperature is clear.

Graph 4 Temperature in the middle of the slab at the surface – January vs. 17°C
The difference in cooling velocity between the different temperature profiles, also has an effect on the stresses within the slab. This can be seen in Graph 5, where the stresses are shown along the slab 4 hours after removal of the sheeting, when there is no cracking yet for any of the temperature profiles.

Graph 5 Stresses $\sigma_{xx}$ at the surface at location L4 – 1 day 4 hours

It can be seen that the stresses of the constant temperature profile of -17°C are the highest, and decrease in the following order: April, January, October, July, 17°C. The overall observation is that, the lower the temperature, the higher the stresses. The faster the tensile stress (positive) increases, the faster the tensile strength of the concrete will be exceeded and the faster cracking will appear.

When studying the mechanism for the month of January in particular, it can be seen that the cracks initiate at the location of transverse reinforcement bars. In Figure 3, the cracks after one month can be seen.

Figure 3 Crack strain pattern after one month – January

The reason why the cracks are formed there, is because there are stress peaks at the location of the transverse bars. In concrete, stress concentrations typically form at the location of edges or imperfections. Therefore, the locations of the transverse reinforcement bars are typical locations along the free transverse side of the slab where such stress concentrations appear. After the propagation of the cracks, the stress at the location of the crack is relieved and decreases to zero. Now, tensile stresses can be noticed in the longitudinal steel bars, which pull at both sides of the crack in order to keep it tightly closed, while the bars are in compression in between the cracks.

2) Active crack control

When active crack control is modelled, the temperatures are only slightly lower, because of the presence of extra surface subject to convection, within the notches. The similarity of the temperatures leads to stresses which are very similar to the ones in Graph 5: the effect of the temperature profile remains.

To check the difference with the case without active crack control, also the month of January is considered. The crack pattern after one month is shown in Figure 4. In this case, it can be seen that the cracks are located at the transverse position of the sawcuts.

Figure 4 Crack strain pattern after one month – January – active crack control

This is the case because of the sawcuts. By applying these, stress concentrations are formed near the tips of these sawcuts. This can be seen in Graph 6, where the stress along the slab is shown near the tips of the sawcuts, just before the crack propagates in that location.

Graph 6 Stress $\sigma_{xx}$ at the surface at $Y = 500$ mm – active crack control vs. no active crack control

From this graph, the reason for the location of the cracks is very clear. The behavior of the reinforcement bars after the cracking is the same as in the case without active crack control: high tensile stresses at the location of the cracks and compression stresses in between.

B. Part 2 – convection coefficient

In the second part, the influence of a different convection coefficient is examined. This is a very important parameter in the field of the early-age behavior of CRCP, so a clear influence is expected. For this part, two temperature profiles are considered: January and July. In these models, active crack control is modelled.

In Graph 7, the stresses in function of time can be seen for the different convection coefficients, at the location of one of the cracks and far enough from the sawcut. The three models are identical during the first day, which is reflected in the graph. After this first day, there is a difference in the models: a different convection coefficient. A higher convection coefficient means that the heat which is developed during the first day due to the hydration in the concrete, will faster be exchanged with the ambient air and that the cooling of the concrete will thus happen faster. Because of this, the tensile
stresses within the concrete because of the cooling, which develop because of the concrete contraction, will develop faster as can be seen in the graph. As the tensile strength in the concrete is practically the same for the three cases, the cracks will propagate faster when the convection coefficient is higher.

For the month of July, similar findings are made, with the only difference that the cracks propagate later on and slower as the temperature decrease in the concrete is less steep.

C. Part 3 – bondslip

In the third and final part, the modelling of bondslip will be discussed. This is done in models without active crack control, as otherwise it was impossible for DIANA to calculate the model. For this part, a model in which bondslip is modelled is made for four temperature profiles: January, April, July and October.

When bondslip is modelled, it can be seen that the cracks initiate slightly later. The crack pattern for the month of January at the last calculated time step can be seen in Figure 5. In this figure, it can be seen that the main cracks also propagate at the location of the transverse reinforcement bars, but that there have also formed smaller cracks between them.

In Graph 8, the stresses along the slab can be seen for the cases with and without bondslip, right at the edge and at the level of the reinforcement. In this graph, the mechanism of bondslip is clear. At the time step of 4 days and 14 hours, the stress in the concrete is nearly the same for both cases. However, near the transverse reinforcement bars, the stress peak is much smaller. This is because bondslip between the concrete and the reinforcement steel bars is allowed, so a part of the stress is relieved in this way. Because of this, the cracking will be initiated later as the stress peaks will not yet exceed the tensile strength of the concrete. The stress after a time of 5 days does exceed this tensile strength, so from that time step, cracks are formed.

The stresses in the reinforcement after the cracking are similar in this case as discussed before. However, when bondslip is modelled, both the tensile and the compressive stresses are clearly smaller. This is also a direct effect of the bondslip: because this is allowed between the reinforcement and the concrete, the stresses are not only relieved in the concrete, but also in the steel.

V. CONCLUSIONS

Altogether, a clear and explainable influence was noticed for each of the investigated aspects: higher tensile stresses and thus more cracks in the early stages for a colder external temperature profile, faster crack propagation for a higher convection coefficient because of the faster cooling and smaller stress peaks and thus a delayed crack initiation when bondslip was modelled. These are the main conclusions of this research.

In future work, it is suggested to perform site tests on actual CRCP sections. This is necessary in order to fully understand the actual cracking behavior of CRCP, as a simulation using software always has its limitations. In future researches, it is recommended that software models are used in combination with field tests, and that some refinements are made regarding the modelling of CRCP.

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<th>Meaning</th>
</tr>
</thead>
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<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>AC</td>
<td>Active Crack (Control)</td>
</tr>
<tr>
<td>ATB</td>
<td>Asphalt-Treated Base</td>
</tr>
<tr>
<td>BS</td>
<td>Bondslip</td>
</tr>
<tr>
<td>CEB-FIP</td>
<td>Comité Européen du Béton – Fédération Internationale de la Précontrainte (European Committee for Concrete – International Federation for Prestressing)</td>
</tr>
<tr>
<td>CRCP</td>
<td>Continuously reinforced concrete pavement</td>
</tr>
<tr>
<td>CTB</td>
<td>Cement-Treated Base</td>
</tr>
<tr>
<td>DIANA</td>
<td>Displacement Analyser</td>
</tr>
<tr>
<td>EICM</td>
<td>Enhanced Integrated Climatic Model</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite element method</td>
</tr>
<tr>
<td>FEA</td>
<td>Finite element analysis</td>
</tr>
<tr>
<td>FWD</td>
<td>Falling Weight Deflectometer</td>
</tr>
<tr>
<td>GPR</td>
<td>Ground Penetrating Radar</td>
</tr>
<tr>
<td>HIPERPAV</td>
<td>High Performance Paving (Software)</td>
</tr>
<tr>
<td>HMA</td>
<td>Hot Mix Asphalt</td>
</tr>
<tr>
<td>JPCP</td>
<td>Jointed plain concrete pavement</td>
</tr>
<tr>
<td>JRCP</td>
<td>Jointed reinforced concrete pavement</td>
</tr>
<tr>
<td>LCB</td>
<td>Lean Concrete Base</td>
</tr>
<tr>
<td>MEPDG</td>
<td>Mechanistic-Empirical Pavement Design Guide</td>
</tr>
<tr>
<td>NAC</td>
<td>No Active Crack (Control)</td>
</tr>
<tr>
<td>NTNU</td>
<td>Norges Teknisk-Naturvitenskapelige Universitet (Norwegian University of Science and Technology)</td>
</tr>
<tr>
<td>PCC</td>
<td>Portland Cement Concrete</td>
</tr>
<tr>
<td>Symbol</td>
<td>Meaning</td>
</tr>
<tr>
<td>--------</td>
<td>-------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$A_c$</td>
<td>cross-sectional area of the concrete</td>
</tr>
<tr>
<td>$A_s$</td>
<td>cross-sectional area of the longitudinal reinforcement</td>
</tr>
<tr>
<td>DSSX = $D_{11}$</td>
<td>linear stiffness modulus (shear stiffness)</td>
</tr>
<tr>
<td>DSNY = $D_{22}$</td>
<td>linear stiffness modulus (normal stiffness)</td>
</tr>
<tr>
<td>$E_{cm}$</td>
<td>Young’s modulus of the concrete</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>characteristic cylinder-compressive strength (concrete)</td>
</tr>
<tr>
<td>$f'_{ck}$</td>
<td>characteristic cube-compressive strength (concrete)</td>
</tr>
<tr>
<td>$f_{cm}$</td>
<td>mean compressive strength (concrete)</td>
</tr>
<tr>
<td>$f_{cem}$</td>
<td>mean uniaxial tensile strength (concrete)</td>
</tr>
<tr>
<td>$f_t$</td>
<td>tensile strength (concrete)</td>
</tr>
<tr>
<td>$f_y$</td>
<td>yield strength (steel)</td>
</tr>
<tr>
<td>$h$</td>
<td>notional size of the member</td>
</tr>
<tr>
<td>$h_{conv}$</td>
<td>heat transfer coefficient</td>
</tr>
<tr>
<td>$q_{conv}$</td>
<td>heat flux due to convection</td>
</tr>
<tr>
<td>R</td>
<td>radius of the reinforcement bar</td>
</tr>
<tr>
<td>$R_b$</td>
<td>ratio of the reinforcement surface to the volume of the concrete</td>
</tr>
<tr>
<td>$S_{XX} = \sigma_{XX}$</td>
<td>global Cauchy stress in the X-direction</td>
</tr>
<tr>
<td>$T_c$</td>
<td>temperature of the concrete</td>
</tr>
<tr>
<td>$T_e$</td>
<td>external temperature</td>
</tr>
<tr>
<td>$u$</td>
<td>perimeter in contact with the atmosphere</td>
</tr>
<tr>
<td>$\alpha_t = CTE$</td>
<td>coefficient of thermal expansion</td>
</tr>
<tr>
<td>$\delta u$</td>
<td>iterative displacement increment</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson ratio</td>
</tr>
<tr>
<td>$\Pi$</td>
<td>energy potential</td>
</tr>
<tr>
<td>$\rho$</td>
<td>density</td>
</tr>
</tbody>
</table>
Chapter 1. Introduction

1.1. Introduction on CRCP

To start, a short introduction about CRCP will be given. This is done in order to get a better understanding of the problem statement. There will also be focused on the history of CRCP in Belgium. Later on, in the next chapter, CRCP will be discussed more in detail.

1.1.1. CRCP – basics

Continuously reinforced concrete pavement (CRCP) is an excellent pavement solution for heavily loaded roads with a long life performance and minimum maintenance. Continuous longitudinal steel reinforcement is provided without any transverse joints except the construction joints where required for the construction purposes, in order to control the longitudinal shrinkage of concrete. It overcomes the defects caused by transverse joints in a common concrete pavement, improves the stability of vehicles, and increases overall capacity of pavement. The change in temperature and moisture conditions causes volumetric changes in the concrete, leading to the formation of cracks. Those cracks appear in the form of a random crack pattern all over the width of pavement which lead to structural distress in the pavement such as punchouts. These early age cracks are held tightly closed by the longitudinal reinforcement. It is believed that the early age cracking is affected by concrete material properties, environmental conditions and longitudinal steel layouts.

A technique to control the crack pattern and to limit premature crack deterioration, is active crack control, also called induced cracking. The technique has been used widely in recent years, however originally mainly for JPCP (jointed plain concrete pavements) are JRCP (jointed reinforced concrete pavements). Since CRCP is meant to allow the occurrence of cracks in a natural way and there is no further control over the origination of these cracks, it turned out to be necessary to have a uniform crack pattern. For this reason, active crack control was also introduced at CRCP sections. It was found that this technique achieved transverse cracks sooner and straighter and at a more regular interval. The technique can thus reduce the probability of a non-uniform crack pattern and even prevent punchouts.
The influence of this active crack control technique, as well as the influence of the concrete material properties, longitudinal steel lay-outs and environmental conditions such as the temperature profile, are matters which need to be investigated further. It is desirable that more studies are developed to perform more research concerning these fields.

1.1.2. CRCP – history in Belgium

In Belgium, there are a lot of examples of concrete roads which served traffic for more than fifty years. An example is the Avenue de Lorraine in Brussels, which served traffic from 1925 until 2003. It can thus be stated that Belgium has a long history of concrete road construction. From all roads in Belgium, not less than 17% consists of a concrete pavement and from the Belgian motorway network, the share of concrete pavements amounts 40%. From these concrete pavements, the most are from the type of CRCP. [1]

CRCP is used in Belgium on a large scale for more than forty years. It was first applied in the 1950s on some experimental sites, but it was only in the late 1960s that the technique was becoming more largely applied and in 1970, the first large-scale construction was started. Along the years since then, there have been used different concepts, which were called Concept 1, Concept 2 and Concept 3.

Concept 1 was used between 1970 and 1977. In this concept, the structure consists of 0.85% longitudinal reinforcement which was placed at a depth of 60 mm and a concrete slab with a thickness of 200 mm. Between the lean concrete base and the concrete slab, an interlayer with a thickness of 60 mm was placed. In this concept, it was found that the average crack spacing was very low. [2]

Concept 2 was then used between 1981 and 1991. In this concept, the structure consists of 0.67% longitudinal reinforcement which was placed at a depth of 90 mm and a concrete slab with a thickness of 200 mm, placed on a lean concrete base with a thickness of 200 mm. The asphalt interlayer was dropped. It was found that the crack distribution was more regular than with concept 1, with an average crack spacing from 1.4 m to 2.4 m and almost 70% of the crack spacing being from 0.8 m to 3.0 m. Apart from this observations, it was found that the CRCP using Concept 1
still behaved perfectly, while the CRCP using Concept 2 showed erosion of the base layer resulting in punchout problems. [3]

Because of the problems concerning Concept 2, Concept 3 was introduced, also with the eye on increasing traffic loads. This Concept 3 was introduced in the 1990s and consists of 0.75% longitudinal reinforcement which was placed at a depth of 80 mm and a concrete slab with a thickness of 230 mm. In this concept the asphalt interlayer with a thickness of 60 mm from concept 1 which was absent in Concept 2 was reintroduced. In 2010, the slab thickness was increased from 230 mm to 250 mm, which should be laid with concrete containing air entrainer for the highest traffic loads. In Figure 1, a typical cross section can be seen of CRCP according to Concept 3, using a lean concrete base (see next paragraph).

![Figure 1: A CRCP cross section of Concept 3](image)

In Table 1, an overview is given of the characteristics of the three concepts. In the models used in this Master Thesis, the Design Concept 3 will be followed. This will be reflected in the slab thickness, the reinforcement depth and the steel lay-out of the longitudinal and the transverse reinforcement.
Table 1: Overview of the CRCP characteristics for the three concepts [5]

<table>
<thead>
<tr>
<th></th>
<th>Concept 1</th>
<th>Concept 2</th>
<th>Concept 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab thickness</td>
<td>20 cm</td>
<td>20 cm</td>
<td>23 cm</td>
</tr>
<tr>
<td>Reinforcement depth</td>
<td>6 cm</td>
<td>9 cm</td>
<td>8 cm</td>
</tr>
<tr>
<td>Interlayer</td>
<td>Asphalt interlayer</td>
<td>No asphalt interlayer</td>
<td>Asphalt interlayer</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>Longitudinal</td>
<td>0.85 %</td>
<td>0.67 %</td>
<td>0.75 %</td>
</tr>
<tr>
<td>reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal bars</td>
<td>ø 18 mm every 15 cm</td>
<td>ø 16 mm every 15 cm</td>
<td>ø 20 mm every 18 cm</td>
</tr>
<tr>
<td>Transverse bars</td>
<td>ø 12 mm every 60 cm</td>
<td>ø 12 mm every 60 cm</td>
<td>ø 12 mm every 60 cm</td>
</tr>
<tr>
<td>Air-entraining agent</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

In order to summarize the different concepts, a schematic representation of the cross section can be seen in Figure 2 to Figure 5, for each of the different design concepts. For this, half the width of a typical slab is shown, so the width in these figures is 1.8 m. The left side is the free transverse side and the distance between this side and the middle of the first reinforcement bar is for every concept 80 mm.

Figure 2: Cross-section of the concrete slab for Design Concept 1
1.2. Problem statement

As stated before, the influence of this active crack control technique, as well as the influence of the early-age effects and environmental conditions such as the temperature profile on the transverse cracking during the early ages of the concrete pavement, are matters which need to be investigated further.

Also, so far no 3D early-age analysis of transverse cracking with the modelling of active crack control has been performed using the software DIANA. DIANA is an extensive multi-purpose finite element software package that can be used for a wide range of problems arising in civil engineering and thus also in road engineering. From that point of view, it is desirable that more
studies are developed using 3D finite element software to perform more research concerning these
fields.

1.3. Objectives of the Master Thesis

The objectives of this Master Thesis are the following:

- To develop a comprehensive 3D numerical model in order to simulate the early age cracking in CRCP under environmental loading. In DIANA, the material models of the concrete and the reinforcement steel will be set, together with well-chosen temperature profiles and boundary conditions. On this model, a FE analysis is performed in order to study the displacements, stresses and crack patterns in a CRCP segment.

- To check the influence of some important aspects in the field of CRCP. The influence of active crack control on the crack pattern in CRCP will be studied, as well as the influence of the temperature profile, the boundary condition regarding convection and the modelling of bond-slip between the reinforcement steel bars and the concrete.

1.4. Approach and structure of the Master Thesis

First, an introduction is given about the concept of CRCP. The basics and a brief history have already been discussed in this chapter, but the principle will be discussed more in detail. Also, the different base types and reinforcement patterns are discussed more in detail, as well as the concept of active crack control. To end the chapter, a literature review is given discussing some findings of earlier researches about this subject.

Then, there is focused on the modelling itself. The method of finite element analysis is explained and the use of the software DIANA is discussed in detail. The choice of every characteristic of the model, such as the geometry, material properties, structural constraints, environmental loading and finite element meshing, is explained. For the collection of the results, the calculation phase will be divided into several parts. The first part studies the influence of the chosen temperature profile along with the influence of active crack control. This is done by making models using six different temperature profiles, making a variant with and a variant without active crack control for each
temperature profile. The second part studies the influence of a different convection coefficient of the concrete. Therefore, this convection coefficient is modified in two well-chosen models of the first part. The third and final part studies the influence of the modelling of bond-slip. This is done by comparing two identical models with only a difference concerning this bond-slip modelling, and this for four different temperature profiles. In Figure 6, a flowchart can be seen with all the different models made in DIANA. It should be noted that in this flowchart, only the most important aspects are mentioned that distinguish the different models for clarity reasons.

After the calculation of the models, the results can be analysed. The results of the models are compared to each other for each of the different parts. The stresses, displacements, temperatures within the concrete and the crack strains and crack patterns are investigated at different locations along the CRCP section. With the help of these results, the influence of the chosen aspects of CRCP can be studied in detail and the underlying mechanisms can be determined.

Finally, some comments can be made on the advantages or disadvantages of the different temperature profiles, convection coefficients or other aspects. The conclusions can be made about the influence of the chosen aspects, which is the main goal of this Master Thesis.
Figure 6: Flowchart with all the models made in DIANA
Chapter 2. Literature review

In this literature research, firstly there will be focused on the basis of CRCP. A short introduction has already been given, but in this chapter the basic principles will be explained more in detail, as well as the difference between CRCP and ordinary concrete pavements. Also, the different base types and reinforcement patterns are discussed more in detail, as well as the concept of active crack control. To end this chapter, a literature review is given discussing some findings of earlier researches about this subject.

2.1. Basic principle

CRCP consists of longitudinal steel reinforcement and transverse bars. The concrete slab does not contain transverse joints unless it is really necessary for construction purposes, such as end-of-day header joints, at bridge approaches and at transitions to other pavement structures. This is the main difference with other concrete pavements such as JRCP (Jointed Reinforced Concrete Pavement) or JPCP (Jointed Plain Concrete Pavements, where all necessary cracking occurs at joints and not elsewhere in the slabs.). Because of the absence of transverse joints, the transverse cracks occur in a random crack pattern. These cracks are a result of stresses mainly induced by variations in temperature or moisture: the volume changes that go with these temperature and moisture variations, are restrained by the longitudinal reinforcement and the pavement base layer, with tensile stresses in the concrete as a result. These tensile stresses, which are referred to as restraint stresses, increase more rapidly than the strength of the concrete at the early age of the pavement, so eventually, full-depth transverse cracks propagate, dividing the pavement into shorter, individual slabs. As thus, it can be said that the continuous longitudinal reinforcement does not provide a structural reinforcement function, but a shrinkage reinforcement function. [5]

The cracks are held closed by the longitudinal reinforcement: this reinforcement results in internal restraint and produces transverse cracks that are closely spaced with small crack widths. This helps to maximize the interlocking of the aggregates between adjacent CRCP panels. The shorter panel sizes and high load transfer between adjacent panels reduce the bending stresses from traffic loads and temperature and moisture curling. In Figure 7, a close-up can be seen of the reinforcement steel used in a CRCP.
This kind of concrete pavement is well suited for roads which are heavily loaded. Other advantages are the long life performance and the low maintenance. This long-term performance largely depends on the early-age behavior, which is thus a very important field to investigate. Especially the optimizing of the crack spacing pattern, called active crack control, is important with respect to this early-age behavior and will be studied in detail further on. Figure 8 shows a very basic scheme of a CRCP, in which the absence of transverse joints can be seen, as well as the cracks which occur in a controlled pattern.
In summary, a well-designed and constructed CRCP accomplishes the following objectives:

- Eliminate joint-maintenance costs for the life of the pavement, resulting in a reduced number of work zones and related travel days.
- Provide long-term, high load transfer across the transverse cracks, resulting in a smooth and quiet ride with less distress development at the cracks than is the case with jointed pavements.

The following aspects should be kept in mind while designing a CRCP project. In this list, design aspects as well as construction aspects are being considered. [7]

- Structural design, concrete mixture and construction decisions and practices should maximize load-transfer efficiency across cracks and minimize slab flexural stresses.
- Cracks that are closely spaced (0.9 m to 1.2 m) and tight (less than 0.5 mm at the depth of the reinforcement) help maximize load-transfer efficiency and minimize flexural stresses, while keeping the steel stress well below the yield strength.
- Closely spaced, tight cracks can be obtained by including the following measures in the project:
  - Proper longitudinal steel content (typical minimum: 0.7% of the slab cross-section area).
  - Good rebar diameter and spacing.
  - Good overlapping of reinforcement splices.
  - Proper depth of reinforcement placement.
- Reinforcement design has to consider plastic deformation. Stress in the reinforcement should be limited to a reasonable percentage of the yield strength in order to limit the amount of plastic deformation and to avoid fracture.
- Larger and abrasion-resistant aggregates promote proper interlocking of the aggregates and thus a better load-transfer efficiency.
- Good consolidation of concrete around the reinforcement to improve long-term bonding.
- Sufficient slab thickness is required in order to manage transverse tensile stresses due to truck loading and curling.
- Uniform and stable foundation layers, providing good drainage and extending beyond the slab edge through the shoulder area and through transitions at bridge approaches etc.
- The base layer below the CRCP should be resistant to erosion.
- Edge support provided by a widened lane or tied concrete shoulders can improve the performance, reducing bending stresses from heavily-loaded axles.
- Longitudinal construction joints should be tied to adjacent lanes or shoulder slabs.
- For each different CRCP application or weather condition, curing should be actively managed in order to achieve the desired transverse crack spacing and crack width as well as adequate concrete strength.

In Belgium, high strength concrete is widely used for the construction of pavements: the mean compressive strength at 28 days is more than 50 MPa. The cement used for this is either Portland cement or a blast furnace slag cement of the strength class 42.5, having a low alkali content. On all Belgian motorways which are constructed nowadays, exposed aggregate surfaces are being used. For the history of CRCP in Belgium along with a table summarizing the three design concepts which were used through the years, a reference is made to section 1.1.2.

2.2. Base type

2.2.1. Different base layer types

In order to have a good performance of the CRCP, it is important to select a proper base layer. The base layer should be selected keeping an eye on the following considerations:

- The base should provide a smooth and uniform platform for construction.
- The surface should be non-deforming for an accurate placement of the reinforcement and a uniform slab thickness.
- The base should provide sufficient friction with the slab in order to help in the formation of a proper crack spacing.
- The base should not be erodible during the design life of the CRCP.

These considerations in mind, some different base layer types can be used. [7]

- ATB (Asphalt-Treated Base). This kind of base layer is non-erodible and gives a proper friction with the slab. A possible problem is stripping of the asphalt binder from aggregates, so a good mixture design with enough asphalt is needed. The advantages of ATB are the
small loss of support related to moisture content, a smooth platform for construction, a
reduced moisture and temperature curling and a proper amount of friction with the CRCP
slab.

- **CTB (Cement-Treated Base).** This kind of base layer contains crushed aggregates or
granular soil and an amount of cement, in order to provide a 7-day unconfined compressive
strength of 3.5 MPa and a water content of circa 1% below the optimal moisture. CTB is
strong and non-erodible, but under repeated loading it can be possible that there is some
erosion, which can lead to a loss of support and punchouts. Therefore, also for this type of
base layer a proper mixture design is needed.

As it is not desired to have a complete bonding between the CTB and the slab due to the
increase in effective slab thickness and thus the increase in needed amount of steel
reinforcement, it can be useful to use an asphalt interlayer between the slab and the CTB.
This layer is called an asphalt bond breaker and is a layer with a thickness of 25 mm to 50
mm of densely graded HMA (hot mix asphalt), placed on top of the CTB. This layer
decreases the erosion potential and provides a stress relief in the CRCP.

- **LCB (Lean Concrete Base).** This type of base consists of aggregates mixed with a sufficient
amount of cement and water, in order to reach a higher strength than a CTB layer. This
kind of layer reduces base erosion and a loss of support and provides a smooth surface for
construction. Also with this type of base layer, it can be useful to use an asphalt layer on
top of the LCB layer in order to reduce erosion to a minimum and to provide a stress relief.

- **A Dense-Graded Granular Base and Subbase.** This type of base layer, using dense-graded
unbound granular materials with low plasticity, can be used for roads with lower traffic
loads. A relative density of at least 95% is needed in order to have no consolidation and
settlement issues. The construction should be performed very careful in order to have a
minimum loss of density and thus slab support, leading to punchouts.

Since CRCP is mainly used for roads with higher traffic loads, it is more opted to use a
stabilized base such as ATB directly beneath the CRCP and to use a granular subbase layer
between this stabilized base layer and the subgrade.

- **A permeable base.** The function of this type of base layer is mainly collecting the water
that comes into the pavement and leading it away to drains at the edges of the pavement,
quickly enough. An important disadvantage of a permeable base is the infiltration of
concrete mortar, which results in an additional interlocking between the slab and the base and thus increasing the effective thickness and reducing the effective steel percentage. This can lead to wider cracks and early punchouts. Also infiltration in the permeable base by the layer beneath it (such as lime-treated subgrades) is possible, which can lead to differential settlements. Hence, it is useful to use for example a geotextile or a densely graded asphalt layer beneath the permeable base. Also, this base should have a sufficient resistance against erosion, as the stability is very important in the application of CRCP. Altogether, it can be seen that the use of permeable bases in combination with CRCP is rather limited.

2.2.2. Influence of the base type

In the following, there will be focused on the influence of the base type on the behavior of CRCP. Two base-types will be considered: an asphalt bond breaker and a lean concrete base. The longitudinal displacements of the CRCP under short- and long-term temperature loadings was studied by Han et al. [8]. The CRCP with the asphalt bond breaker will be referred to in the Figures as A-CRCP and the CRCP with the lean concrete base as L-CRCP.

It should be noted that the A-CRCP section and the L-CRCP section both have different characteristics apart from the base type. These are the location, the slab thickness, the traffic conditions and the subgrade soil. However, the slab thickness does not vary too much and the other characteristics do not significantly affect the longitudinal behavior of the CRCP.

The short-term measurements lead to the following observations. In Figure 9, the hourly behavior of A-CRCP can be seen. The numbers (A-0, A-10, …) refer to the number of meters away from the free end of the slab. Also the daily maximum expansion at A-0 is indicated in the figure, referred to as DME.
In Figure 10, the hourly slab behavior in L-CRCP can be seen, with analogous notations as used in Figure 9.

From Figure 9 and Figure 10, it can be seen that the DME decreases as the location of measurement is further away from the free end. This is intuitively logical with respect to boundary conditions. However, there is also a difference for the different types of base layer. For A-CRCP, using the asphalt bond breaker, time lags can be noticed for the peak displacements at different longitudinal locations. This means that the responses of a CRCP slab might be affected by previous temperature variations and magnitudes if a viscous layer is used beneath the concrete slab. This implies that slab response predictions can be misleading if they are only based on current temperature data.
For L-CRCP, the longitudinal displacement was clearly smaller when moving further away from the free end compared with A-CRCP. This means that the lean concrete base is more effective at confining the maximum expansions than the asphalt bond breaker, particularly in the zone until 10 m away from the free end.

From the long-term measurements, some considerations about the optimal expansion joint width are made. For A-CRCP, it was found that the annual maximum expansions at the free end could become more than the current criterion for expansion joint width, which is 20 mm. For L-CRCP however, the criterion is acceptable and could even be lower to optimize the structural performance. From the study, which estimated the annual maximum expansions along with the daily maximum expansions, the optimal expansion joint width for CRCP was defined for each kind of base layer type. For A-CRCP, the expansion joint width was suggested to be in the range of 10 mm to 30 mm, depending on the slab length, the design conditions and the construction season. For L-CRCP, this was suggested to be 10 mm.

In Figure 11, the longitudinal displacement at the location of the free end of an A-CRCP section is shown, along with a mid-depth temperature band defined from collected data (shown by A-Mid). In this figure, L stands for long subsection and S for short subsection. It can be seen that the free-end displacements become greater for the long subsection. Altogether, one can see that the maximum free-end expansion and contraction are circa 22 mm and 6 mm, which gives a difference that is more than 20 mm, as stated earlier.

![Figure 11: Longitudinal behavior of the free end of A-CRCP](image)

*Figure 11: Longitudinal behavior of the free end of A-CRCP [8]*
2.3. Reinforcement

In this section, the details and the design of the reinforcement will be discussed more in detail. Before discussing the longitudinal and the transverse reinforcement separately, first the main types and properties will be considered. [7]

The material that is used most frequently in CRCP is deformed steel bars. It is also possible to use other materials such as solid stainless steel or fiber reinforced polymer bars, which provide a higher durability when considering the corrosion issues with deformed steel bars. However, the cost of these materials is higher, which explains the popularity of deformed steel bars.

As explained before, the steel bars are very important in CRCP as the different change in volume between the steel and the concrete generates stresses in these materials. Factors that also influence these stresses are the surface area of the steel and the shape of the deformations of the surface of the reinforcement bars. For this reason, the diameter and the form of the bars must be closely studied.

The yield strength that is required in the use of CRCP is 420 MPa, but in some countries, also higher steel grades are being used. However, a higher yield strength does not always go along with the need of fewer steel when the Young’s modulus of the steel is the same. This Young’s modulus is typically in the order of 200 GPa. The coefficient of thermal expansion of the steel is typically in the range of 10 to 12 x 10^{-6}/°C. This parameter is very important as the difference between the CTE of the steel and the CTE of the concrete, decides the restraints and thus the crack patterns.

In what follows, some guidelines will be discussed as found in the CRCP Manual of the U.S. Department of Transportation [7]. These will be linked to the Belgian guidelines for reinforcement as used in the Belgian Concept 3 which has been shortly discussed above. In Figure 12, a general configuration of longitudinal and transverse reinforcement can be seen, placed upon an asphalt-treated base.
2.3.1. Longitudinal reinforcement

A main purpose of the longitudinal reinforcement is to generate transverse cracks by the restraint of the steel and to hold these cracks tightly closed. The characteristics of the longitudinal reinforcement are the steel percentage, the bar diameter, the bar spacing and the depth of the reinforcement. An optimal reinforcement content means that the crack spacing and width is optimal and that the stress in the steel is within acceptable limits.

2.3.1.1. Reinforcement ratio

The reinforcement ratio is the ratio of the area of longitudinal reinforcement to the area of the concrete: $A_s/A_c$. When this ratio increases, the crack spacing will be smaller, the crack widths will be smaller and the stresses in the steel will be smaller. When keeping the spacing between cracks in a range of 1 m to 2 m, the chance of punchouts or spalling is minimized. This will be further discussed in the next section. However, when there is a proper base support and the cracks do not intersect, a crack spacing from 0.6 m on also shows a good performance. The crack width should be kept under 0.5 mm in order to minimize the infiltration of water and the intrusion of incompressible materials.
The importance of the reinforcement ratio also has to do with the thickness of the concrete pavement. When selecting a higher thickness of the concrete pavement in order to increase the structural capacity, the reinforcement ratio will decrease and thus this will result in a larger crack spacing, larger crack widths and higher stresses in the reinforcement steel. Also an increase in concrete strength can have this effect.

Altogether, the guidelines suggest reinforcement ratios within the range of 0.7% to 0.8%. When exceeding this range, intersecting cracks can occur, leading to possible punchouts. This range suits very well with the Belgian concept 3, which defines a reinforcement ratio of 0.75%.

2.3.1.2. Bar size and spacing

There are also some guidelines concerning bar spacing. There is a minimum spacing in order to ensure a good consolidation of the concrete during the placement. The maximum spacing ensures an adequate bonding of the concrete and the steel and thus the necessary restraint which is needed for the required crack width and spacing. The range which is given to the bar spacing is 100 mm to 230 mm, which again suits very well with the Belgian concept 3 which defines a bar spacing of 180 mm. The bar size depends on the reinforcement ratio and the bar spacing. Typical bar diameters are given to be 13 mm to 22 mm (Concept 3: 20 mm).

When choosing the bar size, one should also take into account the surface area of the reinforcement. Important in this case is the ratio of the reinforcement surface to the volume of the concrete, denoted as $R_b$. When this ratio increases, the average crack spacing will decrease. Also, a greater reinforcement surface implies a greater bond area and thus more restraint to the concrete by the steel. Because of this, the cracks will be tighter. A higher $R_b$ can be reached for a given reinforcement content, by using smaller bar diameters. To take into account these considerations, $R_b$ is controlled and it is recommended to have a minimum $R_b$ of 1.2 $m^2/m^3$ for construction during the summer and a minimum $R_b$ of 1.6 $m^2/m^3$ for construction during the spring or fall.

2.3.1.3. Reinforcement depth

When the reinforcement is depth is kept low, the crack width will be smaller. The drying shrinkage and variations in temperature are more present at the surface, which leads to wider cracks, so it
seems good to keep the reinforcement depth low. However, this increases the chances of corrosion as the chances on exposure to chlorides from de-icing salts increase. It is advised to have a concrete cover of between one third and one half of the slab thickness. When comparing this guideline with the Belgian Concept 3, which defines a concrete cover of 8 cm for a slab thickness of 25 cm, it can be concluded that these guidelines correspond approximately. As the depth of the reinforcement has a great influence on the performance of the CRCP, it is important to handle this reinforcement depth very carefully.

2.3.1.4. Lap splices
At splices, it is important that the longitudinal steel bars are properly lapped, to ensure the continuity of the CRCP. When not properly lapped, this is a main source of failures in the CRCP. In Figure 13, an example of some of these so-called lap splices is shown.

![Figure 13: Example of lap splices](image)

The splicing length should be around 33 times the bar diameter and the bars should be secured well enough so that the two bars stay firmly together. Therefore, it is recommended to use two ties per lap. In Figure 14, a schematic image can be seen for respectively a skewed and a staggered lap-splice pattern.
With respect to these patterns, there are some guidelines. For the skewed pattern, the skew angle must be more than 30° from the perpendicular to the centerline. This can be done in practice by skewing the reinforcement by half the pavement width. For the staggered pattern, no more than one third of the bars can end in the same transverse plane and the minimum distance between staggers should be 1.2 m (4 ft using the units in Figure 14).
2.3.2. Transverse reinforcement

The purposes of transverse reinforcement are the following:

- Supporting the longitudinal steel and maintaining the adequate bar spacing and reinforcement depth.
- Keeping longitudinal cracks held tightly. These cracks can occur because of differential settlements, heave or shallow saw cuts.
- Serving as tie bars across longitudinal joints.

2.3.2.1. Bar size and bar spacing

The bar size and the bar spacing of the transverse reinforcement follows the same reasoning as goes for the longitudinal reinforcement. A typical range for the bar diameter is given to be 13 to 19 mm, which is very similar to the longitudinal reinforcement. However, a bar diameter of 13 mm is most frequently used, which corresponds well with the bar diameter of 12 mm as used in the concept 3 in Belgium.

For the bar spacing, typical increments of 60 cm, 90 cm or 120 cm are given. In concept 3 in Belgium, the bar spacing for the transverse reinforcement was 60 cm until 2010. Since 2010, a bar spacing of 70 cm is being used.

2.3.2.2. Tie bars

A last aspect about the reinforcement that will be discussed, is the use of tie bars. Tie bars are used along longitudinal joints (both construction joints and contraction joints), in order to maintain a tight joint. Keeping the joint tight is important as for contraction joints, this maintains an adequate load transfer, while for construction joints, this keeps two lanes together. Two types of tie bar configurations can be used: a traditional configuration, which is shown in Figure 15, or a two-piece configuration, as shown in Figure 16. The tie bars are usually placed at mid-depth of the CRCP slab.
Concerning the placement of those tie bars, it is a good practice to place them perpendicular to the longitudinal joint at a specified spacing. An example of a common arrangement is the following: tie bars with a length of 760 mm and a diameter of 13 mm to 16 mm, spaced at 760 mm center to center, with half the length on each side of the joint.

### 2.3.3. Conclusions

To conclude the chapter about the reinforcement, it can be said that the guidelines as found in the CRCP Manual of the U.S. Department of Transportation correspond very well to the Belgian Concept 3 which is currently used in Belgium. This implies that the underlying concepts of the use of reinforcement in this Concept 3 have been studied and understood very well, resulting in logical values for important aspects as the bar size and spacing and the reinforcement depth.
2.4. Active crack control

The crack pattern in CRCP can be characterized by a low mean crack spacing as well as a high percentage of clusters of closely spaced cracks. Also, there is a probability of a non-uniform crack pattern such as closely spaced cracks, divided cracks, meandering or Y-cracks, which are schematically shown in Figure 17. However, it is difficult to reduce this probability by just adjusting the amount of longitudinal steel. This is a problem, as a non-uniform crack pattern may lead to the development of punchouts.

![Figure 17: Schematic representation of different kinds of crack formation [5]](image)

A technique to control the crack pattern and to limit premature crack deterioration, is active crack control, also called induced cracking. The technique has been used widely in recent years, however originally mainly for JPCP (jointed plain concrete pavements) are JRCP (jointed reinforced concrete pavements). Since CRCP is meant to allow the occurrence of cracks in a natural way and there is no further control over the origination of these cracks, it turned out to be necessary to have a uniform crack pattern. For this reason, active crack control was also introduced at CRCP sections. It was found that this technique achieved transverse cracks sooner and straighter and at a more regular interval. The technique can thus reduce the probability of a non-uniform crack pattern and even prevent punchouts. In Figure 18, a first representation of the effect of active crack control can be seen. The method used in this example will be discussed in the next section.
2.4.1. Methods

A first method to apply active crack control, is the early entry sawcutting method. Early entry saws are lightweight devices that allow the sawing operation to begin 1 h to 4 h after the casting of the concrete. Most of these saws use a dry-cutting operation with blades that do not require water for cooling. The purpose of this method is that the probability increases that the transverse cracks will be induced at the location of the sawcuts. Another advantage of this method is that the depth of the sawcuts can be reduced because the pavement is sawed in an early stage. In Figure 19, an example of a typical sawcutting machine of the manufacturer Soff-Cut can be seen. It should be noted that this method originally intended to make a cut along the whole width of the slab, but this would change later on, as will explained later in this section.
Another method to apply active crack control, is the metallic and plastic insertion method. With this method, single and double metallic layers are inserted into fresh concrete in order to act as crack inducers. These crack inducers intend to induce bottom-up cracks in the slab. It was found that the early entry sawcutting method was more effective, but when plastic tape was inserted in the top part of the CRCP, it was found that the cracks developed earlier than with the early entry sawcutting method. These findings are due to the location of the crack inducer: for the same pavement surface reduction area, the crack inducer inserted at the top of the slab is more effective than the crack inducer inserted in the interior of the slab. This is because the changes in temperature and moisture at the surface are higher, which helps to initiate cracking. In Figure 20, an example of a plastic insertion method can be seen: the automated tape insertion method. This method consists of creating a weakened plane by inserting a plastic strip in the fresh concrete up to a depth of approximately 75 mm.
With respect to these methods, an important limitation was experienced. This limitation is that a transverse sawcut along the whole width of the slab could decrease the aggregate interlocking and the load transfer efficiency. This leads to a reduction of the pavement lifetime and to the formation of surface defects such as spalling.

To overcome this limitation, a new active crack control method was introduced by Rens et al. in 2012. This method aimed to reduce the number of clustered, closely spaced cracks by means of a partial surface saw cut method. This method was first applied in the reconstruction project of a section of the motorway E313. In this section, active crack control was applied by performing sawcutting at one side of the concrete slab, perpendicular to the road, with a length of 40 cm, a depth of 3 cm and a sawcutting spacing of 1.20 m. The cuts were applied immediately after the exposed aggregate surfacing and within 16 h after the paving of the concrete. An overview of this method can be seen in Figure 21.
Figure 21: Schematic overview of this method of (a) the joints and lanes and (b)-(c) the sawcut geometry (in mm) [9]
2.4.2. Sawcutting parameters

When focusing on the last early entry sawcutting method, it can be seen that different parameters of this method are important, such as the sawcutting timing and the sawcutting depth. These two parameters will be discussed in this section.

The sawcutting timing is very important, as when the operation is done too early, raveling of the concrete will occur because the concrete strength is not high enough in order to resist the saw. When the operation is done too late, the buildup of residual stresses can result in random cracking. The longer the delay of the sawcutting, the greater the probability becomes of random cracking. For this reason, an ideal sawcutting window exists, as can be seen in Graph 1.

As thus, it was found that the notches should be made between the initial and the final setting of the concrete. A general time frame is found to be between 4 hours and 12 hours after the concrete placement, but this depends on the materials, mix properties, environmental factors and external restraints. Another consideration is the use of an exposed aggregate surface in Belgium. As explained earlier, a sheeting is done to prevent the exposed aggregate surface from drying out. Because of this sheeting, the sawcutting can only be applied after the removal of the sheet, which is around 10 hours to 24 hours after the concrete placement, so that the conventional construction execution is not affected.
Also concerning the sawcutting depth, some remarks can be made. Since for CRCP the concrete cover should be sufficient to prevent corrosion of the reinforcement, the sawcutting cannot be done as deep as is the case for JPCP pavements. However, a shallower sawcut will be sufficient as it takes advantage of the greater changes in moisture and temperature at the slab surface in order for the cracks to initiate at the sawcut tips. Since the standard concrete cover for CRCP in Belgium is 80 mm, a proper sawcutting depth would be between 30 mm and 60 mm.

2.4.3. Conclusions

Based on the methods and test sections discussed in the previous sections, the following conclusions can be made.

- The sawcutting method with partial surface sawcuts instead of a sawcut along the whole width provides a crack initiation beneath those sawcuts: most of the cracks initiate at the tip of the sawcuts. The effect of this method is especially efficient within the first two months after concrete placement.
- This method can decrease the percentage of short-spaced cracks and clusters of cracks. Also, the cracks become straighter and occur in a more regular pattern.
- The sawcutting timing and sawcutting depth have a great influence on the effectiveness of this crack induction: a larger depth and earlier sawcutting can help the crack induction at the sawcuts.
- The crack width is slightly smaller when active crack control is used in comparison with a situation without active crack control.

2.5. Literature – previous research

In this section, some different papers will be studied. There will be one paragraph about each paper, in which will be discussed what was exactly researched, what kind of software or research program was being used and what the conclusions were. At the end of this chapter, a conclusion will follow mentioning the things that should be investigated further.
2.5.1. Study of concrete thermal properties for sustainable pavement design

In the first research that is reviewed, performed by Hu et al. [11], the thermal properties of Portland cement concrete (PCC) were examined. First, a literature study was performed to define the major factors affecting the thermal properties of PCC as well as the influence of the thermal properties such as the coefficient of thermal expansion on the pavement behavior. Afterwards, some experiments were performed, more specifically CTE tests were performed on some laboratory and field samples. Also, the thermal conductivity of two kinds of concrete were tested: a typical Iowa PCC mix and an asphalt cement concrete mix.

The conclusions were the following:

- The thermal properties of concrete are affected by the concrete materials, mix proportion, moisture condition and age. This was also clear from the test with the two kinds of concrete, which resulted in different thermal properties for the different concrete types.
- The CTE of the concrete is clearly influenced by the type of coarse aggregate.

2.5.2. Effect of plastic sheet curing in concrete pavements

In this research by Ren and Houben [12], a numerical early-age concrete pavement temperature prediction model is presented. In this model, the effects of several parameters on the early-age performance of the pavement can be checked, such as curing method, ambient temperature, paving time and wind velocity. Some existing heat flux models are reviewed and an extension on these models is proposed, based on the energy balance method. This prediction model was verified using the measured data of two field sections and showed a match with these field data. Using this model, the effect of polyethylene sheeting on the development of the pavement temperature was analysed.

The conclusions were the following:

- Polyethylene sheeting is very effective in capturing moisture of freshly poured concrete and thus minimizes the plastic shrinkage and reduces drying shrinkage.
- When used improperly, polyethylene sheeting can result in too high concrete temperatures in summer conditions, causing damages. To solve this issue, an adequate type of polyethylene sheeting should be selected.
2.5.3. Mechanism and modelling of transverse cracking in CRCP
This research is done by Al-Qadi and Elseifi and is about the relationship between the transverse cracking in CRCP and the transverse steel reinforcement [13]. First, field tests were performed including surface condition inspection with digital video, ground penetrating radar survey (GPR) to determine the location of the cracks relative to the bars, falling weight deflectometer (FWD) to check the load transfer efficiency and ground-truth coring. Then, a 3D finite element (FE) model was made to check which factors contribute to the transverse cracking. This was done using the software ABAQUS. It should be noted that creep and the effects of early behavior of concrete were not considered.

The conclusions were the following:

- From the tests, it could be concluded that the mean crack spacing was equal to the design spacing of the transverse steel reinforcement bars.
- From the FE model, it was found that the temperature gradient in the concrete was non-linear.
- Also from this model, it was found that two mechanisms contribute to the formation of transverse cracks:
  - Compressive longitudinal stress at the surface between transverse bars
  - Tensile stress concentrations near the transverse bars

2.5.4. Crack spacing and crack width investigation from CRCP sections
In this research by Kohler and Roesler [14], some full-scale CRCP sections were investigated and the crack width and crack spacing were examined. The crack width model as proposed by the MEPDG (the Mechanistic-Empirical Pavement Design Guide) was used to standardize the measurements and was compared to the measured data.

The conclusions were the following:

- The crack spacing as predicted by the model matched the test results, based on the material properties, geometry and environmental conditions.
- The crack width as predicted by the model overestimated these test results.
- The measured crack width was smaller for a higher reinforcement ratio.

2.5.5. Mid-depth punch-out research of CRCP

The punch-out phenomenon was researched by Fan et al. [15], using a non-linear finite element (FE) analysis. This model was made using the software ABAQUS. In this model, the thermal sensitivity of the concrete and the steel and the bonding between the concrete and the reinforcement steel were taken into account, as well as bottom friction slip. In this research, the influence of different factors on the transverse cracks was examined. These factors were the coefficient of thermal expansion, the Young’s modulus of the concrete and the steel, the bondslip between the concrete and the steel and the location of the reinforcement bars.

The conclusions were the following:

- The main reason of mid-depth punch-outs is the appearance of horizontal cracks in the reinforced layer.
- The higher the Young’s modulus and the CTE of the concrete, the greater the concrete vertical stress becomes. Choosing an aggregate with a low Young’s modulus and CTE is therefore helpful in controlling the transverse cracking.
- The concrete stress is clearly higher when no bondslip is modelled.
- The higher the reinforcement depth, the smaller the concrete stress becomes.
- The maximum of the temperature shrinkage stress was found to be near the transverse cracks and near the longitudinal steel bars.

2.5.6. Forensic investigation of CRCP in fair and poor condition

In this research, done by Chorzepa et al. [16], two CRCP sections that showed closely spaced cluster cracks are investigated. One section was in poor condition as several punchouts showed in this section, and the other section was in fair condition. In order to investigate these sections, non-destructive as well as destructive tests were performed. The non-destructive testing methods used were a ground penetration radar (GPR) and a falling weight deflectometer (FWD), while the destructive testing methods used were coring and both on-site and laboratory testing.
The conclusions were the following:

- The cause of the punch-outs in the section in poor condition, was found to be a poor material composition (aggregate segregation and soft paste) and environmental conditions (thermal expansion/contraction and weather cycles).
- The distress in both sections generally resulted from early-age cracks such as shrinkage cracks, which were spaced too closely: the crack spacing is recommended to be above 0.6 m. Cluster cracking is a result of thermal expansion/contraction and concrete shrinkage in combination with increased traffic or fatigue loading. When longitudinal cracks appear, bounded by transverse cluster cracks, there is a risk of punch-outs.

2.5.7. Cracking behavior of CRCP in Belgium

In this research by Ren et al. [17], some previous field investigations are summarized and reviewed. Also some recently constructed CRCP sections are discussed. The influence of the adaptations in the design concepts are reviewed and discussed. The previous investigations that are discussed are done by Verhoeven and Van Audenhove, Holler, Winner et al. and Van Avermaet and Van Weyenberg, all on CRCP sections in Belgium. The crack spacing and crack width were investigated in detail, including cluster cracking and crack width movement.

The conclusions were the following:

- For the recently constructed CRCP sections, the crack spacing is low and the percentage of clusters of closely spaced cracks is high. The behavior of CRCP sections of the current design concept is good and the deterioration is rather small.
- The steel percentage ranging from 0.70 % to 0.75 % does not influence the crack spacing or the cluster cracking significantly, while decreasing this percentage to 0.65 % seems to improve the crack pattern.
- The higher the percentage of longitudinal reinforcement, the smaller the crack width on the pavement surface. The addition of steel fibres controls this crack width.
2.5.8. Active crack control for CRCP in Belgium

This research, done by the same researchers as the previous one, is about active crack control through partial surface notches [9]. Some previous methods and tests are reviewed, done by McCullough, by Kohler and Roesler and by Lim. Afterwards, a new partial surface notch for active crack control is proposed and explained. This is adopted in the motorway E313 in Herentals, Belgium in 2012. The effectiveness of this new practice is discussed. This new practice has been discussed earlier and can be seen in Figure 21.

The conclusions of this research have also been discussed before and can be found in section 2.4.3.

2.5.9. Experimental analysis of curling behavior of CRCP

In this study by Han et al. [18], the vertical deformation of a CRCP section, called curling, was examined. To this end, some in-situ tests were performed at a full scale test section. Two sections were examined: one with a length of 0.75 m and one with a length of 2.0 m, both with a free end expansion joint.

The conclusions were the following:

- The slab sections curled corresponding to the temperature gradient.
- The maximum deformations were found to be located at the slab edges and at the transverse cracks. The deformations located at the longitudinal joint were found to be smaller.
- The curling deformation at the expansion joint was found to be independent of the length of the test section.

2.5.10. Conclusions

In this section, some previous researches were reviewed and summarized. Not every research on the subject of CRCP was reviewed, but a considerable part of the researches is considered. Some researches were left out because they were very close to a research that was already reviewed, or because another similar research from the same person was already reviewed. In the past, there have also been some Master Theses about this subject, for example two Master Theses on the
optimization of active crack control in CRCP by S. Pascarella (2018) and S. Depuydt (2016). In these Theses, a 2D model of a CRCP section was made using the software program Samcef Field. Altogether, it can be concluded that there has certainly been some research on this subject, but that not every aspect of it is already studied in every detail. The thermal properties were investigated through some tests, as well as the effect of sheeting. Also the mechanism of the transverse cracks themselves was studied by means of testing. Different aspects of these cracks were examined, such as the crack width, crack spacing, punchouts and active crack control. Also the curling behavior in CRCP is examined through testing. Furthermore, some FE models have been made in different software programs such as ABAQUS (3D models) and Samcef Field (2D models), in order to investigate aspects as active crack control, punchouts and the relation between transverse cracking and the transverse reinforcement bars.

The previous investigations on the cracking itself were numerous, but researches about the influence of some aspects such as boundary conditions – ambient temperature as well as convection between the concrete and the air – or the modelling of bondslip, which will be studied in this research, were rather scarce. In addition, a FE study of CRCP through a 3D model made using DIANA, seems not to be done yet. Therefore, it can be said that the subjects investigated in this research are a valuable addition to the existing knowledge of (cracking in) CRCP.
Chapter 3. Methodology

3.1. Finite Element Modelling

The finite element method (FEM) is a numerical method used for solving engineering or mathematical problems. Typical problem areas can be fluid flow, mass transport, electromagnetic potential or, as is the case in this research, structural analysis and heat analysis. These problems are solved by dividing a large system into a finite amount of smaller, simpler parts: the finite elements. The equations used to model these finite elements are then put together into a large system of equations describing the entire problem. The division into smaller parts has several advantages: a complex geometry can be represented accurately, different material properties can be included, the total solution can be represented easily and the local effects are considered and can be investigated.

The practical application of FEM is called finite element analysis (FEA). It is a computational tool for performing engineering analysis. It includes techniques of mesh generation for dividing a model into smaller elements, as well as a software containing a FEM algorithm. FEA is a good choice for the analysis of problems where the material properties or environmental conditions differ in function of time (such as the adiabatic heat curve, the convection coefficient or the temperature profile, which will be discussed later on) or where the desired precision varies over the domain. In the subject of this Master Thesis, the area near the notches used for active crack control will be more of interest than the rest of the CRCP section.

The modelling of the CRCP section in DIANA in order to perform the FEA consists in short of the following steps:

- Defining the geometry of the model
- Defining the materials and their properties
- Defining the structural constraints by the modelling of supports
- Defining the temperature profile by the modelling of a boundary condition
- Defining the mesh

After the modelling of the CRCP section, the results can be computed by defining the settings of the analysis. A detailed discussion of the different settings in DIANA will be given in the next
section. For the investigation of the results, it is possible to see the results directly on the model by means of colours, but it is also possible to view the results in tabular form or to compose different kinds of graphs.

### 3.2. Software description

As mentioned before, the software used for this Master Thesis is DIANA (DIspacement ANAlyzer). It is a finite element code, based on the displacement method, which is under development since 1972. It is a program with extensive material, element and procedure libraries and pre- and postprocessors to define the model and to view the results in a graphical interactive environment [19]. The most extended capabilities of DIANA lay in the field of (reinforced) concrete and soil. A wide range of material aspects can be considered, such as cracking of concrete, yielding of steel, creep and shrinkage or aging. For the casting of the concrete, heat generation and maturing of young hardening concrete can be simulated.

For the concrete, several material models are available, such as the total strain crack model or the multi-directional fixed crack model, or more special material models such as the Maekawa-Fukuura model. However, the concrete can also be defined with reference to several international design codes, such as the European CEB-FIP Model codes 1990 or 2010, the American Association of State Highway and Transportation Officials (AASHTO) or the Eurocode 2 EN 1992-1-1 [19], which will be used in this research. For the steel, the simple Von Mises plasticity model can be used, possibly with kinematic or isotropic hardening. Also the steel can be defined with reference to design codes such as the Eurocode 3 EN 1993-1-1, but it is chosen to use the Von Mises plasticity model in this research.

The analysis that will be performed is a transient heat transfer analysis to compute the temperature throughout the model for each different time step, as well as a structural nonlinear analysis with a physical nonlinearity. For a physical nonlinear analysis, different material models are available such as plasticity, creep and cracking. Also time dependent development of temperature can be defined. The load can be applied in load steps, but in this Master Thesis, the model will be calculated using time steps. By defining this, the software will perform the analysis for each defined time step.
In the nonlinear analysis, the nonlinear system of equations must be solved iteratively until equilibrium has been reached. For this purpose, there are several iteration schemes programmed in DIANA. These are the Constant and Linear Stiffness method, Regular and Modified Newton-Raphson method and the Quasi Newton methods Broyden, BFGS and Crisfield [19]. In this research, it is chosen to use the Regular Newton-Raphson method. To stabilize the convergence or to increase the convergence speed, it is possible to apply a Line Search algorithm. In this research, in which convergence is an important issue, it is chosen to use this option.

After the results have been calculated, they can be printed in tables, processed in graphs or viewed in several view modes. It is also possible to visualize the crack patterns and to view the results in well-chosen cross sections of the model or in selected points. These options are very important for the investigation of the results and for the comparison of the different models and will thus be used to the fullest.

3.3. Input

In this section, all the different parameters and characteristics which should be defined in DIANA are being discussed. The choice of every input value is being explained and some very important aspects in the field of early-age behavior, such as the adiabatic heat curve, the convection coefficient and the definition of creep and shrinkage in DIANA and the associated parameters, are extensively discussed.

3.3.1. Geometry

The geometry of the model is based on the design Concept 3 for CRCP in Belgium. This is discussed before, but in this section, the precise input parameters will be explained.

3.3.1.1. Concrete slab

In Concept 3, the thickness of the concrete slab is 250 mm, so that is also the thickness used in the model. The model width is taken as 1.8 m, as this is half the typical width of concrete slabs used in CRCP. It is chosen to take only half of the width in order to limit the computation time. Because
of considerations concerning symmetry, the other half of the slab is expected to behave in the same way. The length of the model is 2.4 m. This is chosen with the eye on active crack control: when this is applied with a typical sawcutting spacing of 1.2 m, the model will be long enough to see the full effect of two notches of the active crack control.

When active crack control is modelled, the dimensions will be the following: a width of 4 mm, a depth of 40 mm and a length of 400 mm [9]. The cut spacing is 1.2 m, so the centre lines of the cuts are located at a length of 0.6 m and 1.8 m.

### 3.3.1.2. Reinforcement

Also the reinforcement is modelled following the guidelines as found in Concept 3. This means that the reinforcement depth is 80 mm. The longitudinal bars have a diameter of 20 mm and a bar spacing of 180 mm. However, this implies a reinforcement ratio of 0.70 %, which differs from the prescribed value in Concept 3 of 0.75 %. The reason for this is that the prescribed slab thickness has changed in 2010 from 230 mm to 250 mm. When a slab thickness of 230 mm was used, the reinforcement ratio would be 0.75 %. In the current situation, it is rather unclear whether the bar spacing should be decreased to reach the reinforcement ratio of 0.75 %, or a reinforcement ratio of 0.70 % is accepted. Intuitively, one would argue that the reinforcement ratio of 0.75 % should be respected, but in this model, it is chosen to maintain the bar spacing of 180 mm. The concrete cover is 80 mm, so there are 10 longitudinal bars modelled.

The transverse bars have a diameter of 12 mm and a bar spacing of 700 mm. It is chosen to model a bar right in the middle of the length of the slab, in order to maintain the symmetry. A total of 3 transverse bars is modelled. For the majority of the different parts, the reinforcement type is embedded. For the bond-slip reinforcement, this setting will be different. In Figure 22, the main geometry as modelled in DIANA can be seen for the models including active crack control.
3.3.2. Materials

In this section, all the input parameters of the concrete and the reinforcement steel will be discussed.

3.3.2.1. Concrete

When the concrete material properties are defined, first all the aspects must be selected which have to be included. These aspects are the following:

- Total Strain crack model
- Shrinkage
- Creep
- Heat flow
- Young hardening concrete

Each of these aspects implies extra parameters that should be defined.

3.3.2.1.1. Model code

To model the concrete, it is chosen to use a concrete model code from the library, for which some parameters are fixed according to the specific code. In this model, the Eurocode 2 EN 1992-1-1 is selected, as it is the main code used in Flanders. The Eurocode 2 EN 1992-1-1 model code gives
both the characteristic cylinder-compressive strength $f_{ck}$ and the characteristic cube-compressive strength $f'_{ck}$. Both values are part of the name of the normal weight concrete class. A normal weight concrete is selected, along with a concrete class of C45/55, which means that $f_{ck} = 45$ MPa and $f'_{ck} = 55$ MPa. DIANA uses this input to set the stress–strain diagram in the compressive and tensile regime. The stress-strain diagram according to Eurocode 2 EN 1992-1-1 can be seen in Graph 2.

![Stress-strain diagram](image)

**Graph 2: Stress–strain diagram according to Eurocode 2 EN 1992-1-1 [20]**

The aggregate type Sandstone is selected and the cement type Class N is chosen, which is the normal cement type used in concrete roads. It is chosen to use a relatively strong aggregate type as this is common in concrete road construction. The aggregate type only affects the Young’s modulus in DIANA. The choice of sandstone leads to a lower Young’s modulus in comparison with quartzite, so the concrete will be less stiff. In Table 2, the parameters can be seen which are fixed in DIANA for the defined concrete type.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus $E_{cm}$</td>
<td>25.3982 GPa</td>
</tr>
<tr>
<td>Poisson ratio $\nu$</td>
<td>0.2</td>
</tr>
<tr>
<td>Thermal expansion coefficient $\alpha_t$</td>
<td>$1 \times 10^{-5}/^\circ\text{C}$</td>
</tr>
<tr>
<td>Density $\rho$</td>
<td>2400 kg/m³</td>
</tr>
<tr>
<td>Mean uniaxial tensile strength $f_{ctm}$</td>
<td>3.79545 MPa</td>
</tr>
<tr>
<td>Mean compressive strength $f_{cm}$</td>
<td>53 MPa</td>
</tr>
</tbody>
</table>
3.3.2.1.2. *Creep and shrinkage*

Creep and shrinkage are both physical properties of concrete which are related to a change in shape or in volume of the concrete structure. In DIANA, these two properties can be selected separately to be taken into account in the analysis, but the parameters that are needed in DIANA are in the same window as creep during drying and shrinkage are inseparable. In this chapter, the both physical properties are explained and the parameters that are demanded in DIANA are discussed.

**Creep**

Creep can be defined as follows. It is the deformation of a structure under a sustained load. When a concrete structure is exposed to a long term pressure or stress, its shape can change. This deformation is normally in the direction the force is applied. Examples are a concrete column getting more compressed, or a beam that is bending. Creep does not necessarily cause concrete to fail or break apart. When a load is applied to concrete, it experiences an instantaneous elastic strain which develops into creep strain if the load is sustained. [21]

Creep of concrete is a viscoelastic problem. Viscoelasticity is a material behavior with memory: the strain history affects the current stresses. For this type of behavior, creep experiments are used, where a stress is applied at time zero and the strains are recorded as a function of time: the creep curve. The Eurocode uses creep curves to define this viscoelastic relation. For the implementation of the viscoelastic model, it is important to find an algorithm in which it is not necessary to remember the complete strain history in order to limit the required memory. In DIANA, the Kelvin chain model is implemented for the creep function. DIANA uses curve fitting on this Kelvin chain. [19]

**Shrinkage**

Shrinkage is the change in volume of concrete after its placement and is a detrimental property of concrete, affecting the long-term strength and durability. Shrinkage is the main cause of the initiation of cracks, especially around the reinforcement which does not move along. There are different kinds of shrinkage which can be distinguished: [22]

- Plastic shrinkage: due to water movement out of the concrete while still in plastic state. This can be during the process of hydration or due to environmental conditions leading to evaporation. This type of shrinkage can be reduced by preventing the rapid loss of water
from the surface, for example by covering the surface with a sheeting immediately after the concrete placement. This will also be done in this model, as will be explained later.

- Drying shrinkage: due to the evaporation of water held in the pores, causing the pores to disappear because of capillary forces.
- Autogenous shrinkage: due to the chemical reaction of the cement with the water.
- Carbonation shrinkage: due to the reaction of CO₂ in the presence of water with hydrated cement. Crystals of Ca(OH)₂ are dissolved and CaCO₃ is formed. This new product is less in volume, which causes shrinkage.
- Cooling shrinkage: due to the cooling of the concrete after the hydration process.
- Thermal shrinkage: due to the temperature variations from the ambient air.

Parameters

The parameters that are required for definition of creep and shrinkage in the concrete, are the following: ambient temperature, notional size of the member and relative ambient humidity and the concrete age at the end of the curing period.

The ambient temperature can be defined to model the influence of the environmental conditions. This ambient temperature is thus different for each different temperature profile. When a constant temperature profile is applied, then this constant temperature will be applied as ambient temperature. When a realistic temperature profile is applied, then the average temperature throughout this profile will be applied as the ambient temperature. To give more detail to the environmental conditions, also the ambient humidity can be defined. It is chosen to leave this at the default value of 80%.

In the Eurocode, a notional size is used in the formulation for creep and shrinkage. For that reason, this parameter is required. This notional size can be found using the following formula:

\[
h = \frac{2 \cdot A_c}{u}
\]

In this formula, \(A_c\) is the cross-sectional area of the concrete and \(u\) is the perimeter in contact with the atmosphere. In this situation, \(A_c = 1.8 \text{ m} \cdot 0.25 \text{ m} = 0.45 \text{ m}^2\) and \(u = 0.25 \text{ m} + 1.8 \text{ m} = 2.05 \text{ m}\), which leads to a notional size of the member of \(h = 0.439 \text{ m}\).
The concrete age at the end of the curing period is 1 day: this is the time after which the sheeting will be removed from the concrete. The sheeting is done in order to prevent drying out of the fresh concrete.

3.3.2.1.3. Heat flow

In order to model the heat flow, the following data are required: the heat conductivity of the concrete, the heat capacity of the concrete and the definition of the hydration heat via an adiabatic curve.

The value that is applied for the heat conductivity is 3 W/m°C, as found in the PhD thesis from D. Ren [5]. This value is also supported by Hu et al., who propose a value between 1.5 and 3.5 W/m°C for the heat conductivity. They also proposed a value of $0.74 \times 10^{-5}$°C and $1.3 \times 10^{-5}$°C for the thermal expansion coefficient, so the value which is fixed in DIANA, $1 \times 10^{-5}$°C, is acceptable [11]. For the specific heat capacity, in this same source a value of 1000 J/kg°C is found. Since the concrete used in this model has a density of 2400 kg/m³, the heat capacity that should be applied is $2.4 \cdot 10^6$ J/m³°C.

A very important aspect in the field of the early-age behavior of CRCP is the heat development due to hydration. This is the heat generated when water and cement react. It is influenced by the type, the amount and the fineness of cement, but also by the curing temperature. The heat development is very important for the strength and stresses in the concrete during the early age. Due to a higher rate of heat loss at the surface than at the inside of the slab, there is a thermal gradient which causes cracking in the concrete because of the non-uniform expansion within the slab. Important in this aspect is the rate of heat hydration, which is calculated with the equivalent-age maturity method. The degree of hydration must be determined in this method. This is defined as the cement fraction that has reacted and can be determined experimentally. Two methods are commonly used for this purpose: the determination of the amount of chemically bound water and the determination of the amount of heat generated during hydration.

In DIANA, the heat of hydration must be modelled by defining the adiabatic heat curve. This curve gives the adiabatic heat development in function of time, and is an increasing function. It depends on the heat production rate $q$. In Graph 3, the heat production rate in function of time can be seen.
for the cement type CEM III/B 32.5 for three different testing temperatures [23]. It can be seen that the most heat is produced in the first few days, after which the heat production goes to zero, which means that the adiabatic curve will converge to a maximum value after a few days.

As mentioned, Graph 3 shows the heat production rate for the cement type CEM III/B 32.5. This is to give an example of how the heat production goes with the time. However, the cement type that is very commonly used in concrete pavements in Belgium is CEM III/A 42.5N/LA (Ren, 2015). This is a cement type with a higher amount of clinker and a higher strength, and a limited alkali content (LA: Low Alkali). This implies a different heat production rate, so it is desired to apply an adiabatic curve for this cement type or very similar. This is found in a paper from G. De Schutter [24] and can be seen in Graph 4.

Graph 3: Heat production rate in function of time of CEM III/B 32.5

Graph 4: Adiabatic temperature rise in function of time of concrete with CEM III/A 42.5
The adiabatic heat goes up to 50.97 °C and reaches this maximum after 4 days. This corresponds well to the heat production rate as seen in Graph 3.

Along with the adiabatic heat development goes the definition of the initial temperature of the concrete. This is set to be 20 °C, as this is the initial temperature seen on Graph 4.

3.3.2.2. Steel

For the steel, only the Young’s modulus and the yield strength should be selected. It is chosen to model no plastic hardening for the steel. This is decided because the yield strength will definitely never be reached for this type of application. The Young’s modulus of steel is set to be 200 GPa. For the reinforcement, the steel that is used is BE500S. This means that the yield strength is 500 MPa.

3.3.3. Convection

Another very important aspect in the field of early-age cracking in CRCP is convection. Convection is the heat transfer due to the movement of molecules within fluids such as gases or liquids. In this case, the fluid is the ambient air. The heat transfer takes place because the heated fluid is forced to move away from the heat source, carrying energy with it. To express the heat flux due to convection, Newton’s law of cooling can be used:

\[ q_{\text{conv}} = h_{\text{conv}} \cdot (T_e - T_c) \]

Where:

- \( q_{\text{conv}} \) = heat flux due to convection [W/m²]
- \( h_{\text{conv}} \) = heat transfer coefficient [W/m²/°C]
- \( T_e \) = external temperature (temperature of the air) [°C]
- \( T_c \) = temperature of the concrete [°C]

The heat exchange due to convection can be 3 to 4 times higher than the heat exchange due to longwave radiation [25]. Therefore, the heat transfer coefficient is very important. It is influenced by several factors, such as the surface in contact with the air, the pavement roughness and the
geometry of the CRCP section. However, since convection occurs due to the movement of the ambient air, also the wind plays an important part in this process. In order to obtain the heat transfer coefficient, several empirical prediction models are developed and adopted in concrete temperature prediction models, such as McAdams, EICM and HIPERPAV. In Graph 5, the heat transfer coefficient can be seen in function of the wind velocity according to those prediction models. [5]

![Graph 5: Heat transfer coefficient vs. wind velocity for different prediction models](image)

The wind velocity is very small at the ground level, especially at the location of motorways in Belgium, which are commonly flanked by some rows of trees. Therefore, it is chosen to take the value for the heat transfer coefficient corresponding to a wind velocity of 0-1 m/s. In the model, a heat transfer coefficient of 6 W/m²/°C is applied for the first part. In the second part, where the influence of the convection coefficient will be studied, this parameter will be raised to respectively 8 W/m²/°C and 10 W/m²/°C.

In order to prevent drying out of the fresh concrete, a sheet is applied after concrete placement. This is modelled by setting the heat transfer coefficient to 0 W/m²/°C during this period of sheeting. This is a simplification, as also the sheet has a heat transfer coefficient. However, this value is very small compared with the value 6 W/m²/°C, so the simplification is justified. For the duration of the sheeting, some specifications can be found in guidelines. For example, the AASHTO Guide
Specifications for Highway Construction prescribes a curing period of three days for concrete pavements [26]. Also in other guidelines, curing periods of three or more days are found. In this model, it is chosen to apply a curing period of only one day. In that way, the effect of sheeting can be checked, but the early age behavior of the concrete, which is the main scope of this Master Thesis, is not delayed too much as the heat convection with the ambient air and thus the influence of the external temperature is in that case already present after one day. In Graph 6, the heat transfer coefficient that is modelled in DIANA can be seen as a function of time.

Graph 6: Heat transfer coefficient in function of time as modelled in DIANA

3.3.4. Loads

The only loads that are modelled for this research, are the self-weight of the model and the environmental temperature conditions as an external loading. The self-weight, which acts as a result of gravity, is simply modelled by checking the option ‘global load’ in DIANA. To apply an external temperature profile, an external temperature of 1°C is attached as a boundary condition to the top surface of the slab and one of the two transverse faces. This boundary condition is connected to the convection setting which is defined earlier. Then, the temperature profile is defined by applying a time dependent function to the external temperature of 1°C. For the constant temperature profiles, this is simply done by applying a factor equal to the constant temperature for the whole period of 31 days. The other, realistic profiles are inserted as a table which results in a
multilinear temperature profile. In order to get a detailed profile, a factor must be assigned to a lot of different time steps.

For the temperature profiles, it is chosen to use the profiles from four months in order to cover several parts of the year and thus several temperature conditions: the temperature profiles from January, April, July and October of the year 2018 are applied. The graphs with the temperature profile are found in an online database [27]. However, a tabular input is required, so tables must be generated from these temperature profiles. This is done by using an online tool that reads data from a graph to generate a table [28]. In Graph 7 to Graph 10, the temperature profiles of the four chosen months can be seen which are inserted as a table in DIANA.

Graph 7: Temperature profile in January 2018

Graph 8: Temperature profile in April 2018
As mentioned earlier, an external temperature of 1°C is attached as a boundary condition to the top surface of the slab and one of the two transverse faces. In Figure 23, the locations of the boundary conditions in the model can be seen. The case is shown where active crack control is modelled.
3.3.5. Boundary conditions – supports

In DIANA, a fixed rotation and/or translation in one or more directions can be assigned to a face of the slab. This can be done by using the support settings. In Table 3, an overview can be seen of the faces which have an imposed fixed translation, along with the direction of this fixed translation and the reasoning behind it.

<table>
<thead>
<tr>
<th>Faces</th>
<th>Fixed translation</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom face</td>
<td>Z-direction (vertical)</td>
<td>The slab is supported at the bottom by the base</td>
</tr>
<tr>
<td>One of the two transverse</td>
<td>Y-direction (horizontal:</td>
<td>For the transverse continuity: symmetry</td>
</tr>
<tr>
<td>side faces</td>
<td>transverse)</td>
<td></td>
</tr>
<tr>
<td>The two longitudinal side</td>
<td>X-direction (horizontal:</td>
<td>For the longitudinal continuity</td>
</tr>
<tr>
<td>faces</td>
<td>longitudinal)</td>
<td></td>
</tr>
</tbody>
</table>

In Figure 24 and Figure 25, the modelling of these supports in DIANA can be seen. The case is shown where active crack control is modelled.
3.3.6. Bondslip

In order to model the bondslip behavior between the reinforcement and the concrete in DIANA, another reinforcement property is made. Here, a new material should be made for the reinforcement where bondslip reinforcement can be selected. In this new material, the mass density is demanded by DIANA and is set to be 7850 kg/m³, a common mass density of reinforcement steel. For the failure model, a multilinear graph can be inserted where the friction stress is represented in function of the slip. This graph is shown in Graph 11. The graph is found in a publication on horizontal cracking in CRCP by Kim et al., and is thus very well suited for this research. [29]
This bondslip curve is defined by the values that can be seen in Table 4.

Table 4: Tabulated values for the bondslip diagram

<table>
<thead>
<tr>
<th>Slip [mm]</th>
<th>Friction Stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.203</td>
<td>0</td>
</tr>
<tr>
<td>-0.102</td>
<td>-1.86</td>
</tr>
<tr>
<td>-0.051</td>
<td>-5.31</td>
</tr>
<tr>
<td>-0.025</td>
<td>-4.83</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.025</td>
<td>4.83</td>
</tr>
<tr>
<td>0.051</td>
<td>5.31</td>
</tr>
<tr>
<td>0.102</td>
<td>1.86</td>
</tr>
<tr>
<td>0.203</td>
<td>0</td>
</tr>
</tbody>
</table>

In addition to the friction stress-slip diagram, DIANA also demands some values for certain relations: the shear stiffness DSSX and the normal stiffness DSNY. DSSX is equal to the linear stiffness modulus $D_{11}$, which defines the relation between the shear traction and the relative shear displacement in the X-direction of the reinforcement and which has the unit N/mm$^3$ (stress per
length). DSNY is equal to \( D_{22} \), which defines the relation between the normal traction and the relative normal displacement in the Y-direction of the reinforcement and which also has the unit N/mm\(^3\). DSNY can be interpreted as the stiffness related to the crushing of the concrete by the reinforcing bar. DSSX can be interpreted as the slope of the bondslip curve at the point of zero slip. The value for DSSX is calculated as the slope between zero and the first sampling point in the bondslip curve. This is the reason why DSNY is much larger than DSSX. [30]

According to a Master Thesis from M. Eriksen (NTNU, 2016), the values for DSSX and DSNY are defined. These values can be found in Table 5. [30]

<table>
<thead>
<tr>
<th></th>
<th>DSSX [N/mm(^3)]</th>
<th>DSNY [N/mm(^3)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal</td>
<td>47.80</td>
<td>1269910.00</td>
</tr>
<tr>
<td>Transverse</td>
<td>47.80</td>
<td>2116516.67</td>
</tr>
</tbody>
</table>

Table 5: Values for DSSX and DSNY

For the values for DSSX, a reference is made to the fib Model Code 2010. For the values for DSNY, the following formula is used: [30]

\[
DSNY = \frac{E_{cm}}{2 \cdot R} \cdot 10^3 \frac{N}{mm^3}
\]

Where:
- \( E_{cm} \) = the Young’s modulus of the concrete = 25.3982 GPa
- \( R \) = radius of a reinforcement bar = 10 mm (longitudinal) and 6 mm (transverse)

Also the geometry should be adapted. Here, there is a choice to select ‘circular beam bondslip’ or ‘truss bondslip’. However, after checking the results of both options in one of the models, it could be seen that these were identical in this case. It is chosen to select ‘circular beam bondslip’. Finally, the following element data is assigned to the reinforcement bars: ‘Bondslip Interface Reinforcement – BEAM’. The option ‘BEAM’ is chosen instead of ‘TRUSS’ because also in the definition of the geometry, the option of ‘circular beam bondslip’ was selected.
3.3.7. Mesh

For the mesh generation, several default settings for the whole model can be selected. In this section, the choice of these settings will be explained, together with the element-specific options. [19]

The first setting that can be chosen, is the default mesher type. This can be quadratic/hexagonal or triangle/tetrahedron, referring to the shape of the elements. DIANA will try to use the selected type of element as much as possible. When selecting triangle/tetrahedron, the mesh will only contain elements of this type, but when selecting quadratic/hexagonal, some triangle/tetrahedron elements might be needed in order to fill the geometry. For this model, it is chosen to select quadratic/hexagonal.

The second setting that can be chosen, is the mesher order. This can be linear or quadratic. For the linear option, the mesh edges are straight segments. Here, the mapping from the abstract to the realized element is linear. When selecting quadratic, the edges of the elements can be curved. In this way, the domain boundary can in general be represented more accurately, and there are accuracy benefits in the interior of the mesh. However in this model, as the edges and side faces of the model are straight lines and planes, it is chosen to select the quadratic option.

When selecting the quadratic mesh type, two options can be chosen for the definition of the mid-side location: linear interpolation or on shape. For straight shapes, there is no difference between these two options. However, in concave or convex shapes, the mid-side node location determined by linear interpolation means that this mid-side node can be out of the defined geometry, and the mid-side node location on shape means that this mid-side node is coincident with the defined geometry. This choice is relevant for complex geometries with concave or convex shapes, but in this case, this choice is not so relevant. The option of linear interpolation is selected.

Now, the mesh settings of the different shapes within the model can be defined, being only the concrete slab in this case. By defining the mesh of the concrete slab, automatically a mesh of convection will be generated in accordance with the mesh of the slab. The reinforcement bars are regarded as one reinforcement element and should not be further defined. This defining of the mesh settings of the slab is in order to divide the shapes into the desired number of elements or into elements having the desired element size. Two options can be selected to this end: divisions
or element size. When selecting divisions, a fixed number of edge divisions can be assigned to all the edges and a mesh is generated starting from this division. This option can also be assigned separately to different edges. When selecting element size, the desired element size can be inserted. DIANA will try to adapt as much as possible, but it is of course not guaranteed that all the elements have this same element size. It is chosen to assign a desired element size to the mesh. It is a matter of selecting an element size that is small enough to obtain adequate results, but not too small as this would increase the computation time exponentially.

For the model where no active crack control was modelled, the desired element size is set to be 100 mm. This did not cause any problems and a clean and well-arranged mesh was achieved. This mesh can be seen in Figure 26. In total, the mesh of the concrete slab contains 1296 elements and the mesh of the convection 504 elements.

![Figure 26: Mesh for the model without active crack control](image)

For the model where active crack control was being modelled, the configuration of the elements located near the notches have a great influence on the total number of elements and thus on the computation time. Therefore, selecting a slightly different desired element size can lead to great changes in the total number of elements. It is found that the optimal desired element size is 90 mm. For this desired element size, the mesh of the concrete slab contains 5832 elements and the mesh of the convection 1090 elements, which is acceptable in terms of computation time as well as in terms of adequate results. This mesh can be seen in Figure 27.
3.3.8. **Analysis settings**

To obtain the results, two analyses will be performed: a transient heat transfer analysis and a structural nonlinear analysis. In this section, the different settings of these analyses will be explained.

### 3.3.8.1. Transient heat transfer analysis

For the transient heat transfer analysis, it is important to select the initial temperature of 20°C as an initial temperature field. The solution method is kept at the default method which is parallel direct sparse. This solver is developed by Intel Math Kernel Library and shows a high performance and memory efficient usage for solving large sparse symmetric and asymmetric systems of equations. The analysis type is set to be nonlinear, so the hydration heat analysis and the equivalent age options can be selected. [19]

For the execution of the calculation, the time steps should be defined. An overview of the time steps is given in Table 6. It can be seen that the total duration of the period which will be calculated is 31 days, which is exactly the duration of one month. For the temperature profile of April, having a duration of 30 days, the last time step of 2 days will be replaced with a time step of 1 day, so that the total duration corresponds to this duration.
Table 6: Time steps used in the analysis

<table>
<thead>
<tr>
<th>Time step [day]</th>
<th>Number of steps</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>40</td>
</tr>
<tr>
<td>0.2</td>
<td>10</td>
</tr>
<tr>
<td>0.5</td>
<td>12</td>
</tr>
<tr>
<td>1</td>
<td>11</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
</tr>
</tbody>
</table>

The maximum number of iterations is set on 5 and the convergence tolerance on $1 \times 10^{-6}$. The iteration method is set to be Newton regular. These are the default settings and will be explained in the next section. For the transient heat analysis, typically no convergence problems occur. The output of this analysis contains the temperature, the heat flux, the degrees of reaction and the equivalent age. For this research, only the temperature will be investigated in detail.

### 3.3.8.2. Structural nonlinear analysis

For this analysis, first the types of nonlinearity should be selected. For this analysis, only the option of physical nonlinearity is selected: this takes into account the nonlinear elasticity, plasticity and creep. Geometrical nonlinearity is not to be selected, as the deformations are not too large. The solution method is also for this analysis the parallel direct sparse method. For the start step, an additional load should be selected, being the self-weight of the structure. For the time steps, the same time steps as for the transient heat analysis are selected and can be seen in Table 6.

The iteration method that is selected is also in this analysis the Regular Newton-Raphson method. In this method, the stiffness relation between the tangential stiffness and the displacement difference is evaluated after every iteration. For the Modified Newton-Raphson method, this is only done at the start of the increment. The Regular Newton-Raphson method provides quadratic convergence, so the method only needs few iterations to converge towards a solution. The number of iterations is thus relatively small, but on the other hand, the iterations are time-consuming. [19]

Apart from the iteration method, it can be chosen to select the option ‘line search’. This is very well suited for situations involving rapid changes in tangent stiffness, for example for an analysis...
of reinforced concrete where cracking occurs, as is the case in this analysis. The concept of line search is the following, as described in the DIANA Manual: [19]

“The Line Search algorithm uses a prediction of the iterative displacement increment $\delta u$ as obtained by one of the ordinary iteration algorithms and scales this vector by a value to minimize the energy potential $\Pi$. Although ‘energy potential’ may not be the correct terminology for the physical behavior, e.g. in case of plasticity, this poses no problem for the algorithmic implementation within an increment. While the local minimum of the energy potential represents the equilibrium, the minimum in the line search direction can be regarded as the best solution in the predicted direction.”

This method accelerates the process and in addition provides convergence where this would be impossible without line search. Therefore, it is chosen to select this option.

The maximum number of iterations is set to be 100, in order to obtain convergence in as much as possible time steps. For the convergence norm, the displacement and the force norms are selected. The convergence tolerance is set to be 0.01 for both cases, which is acceptable. When no convergence would be reached, the option is selected that the analysis continues.

Because not all the output variables are needed for this analysis, it is chosen to select some. The following results are selected to be calculated:

- Total displacement
- Green total principal strains
- Green crack strain
- Green principal crack width
- Cauchy total stresses (global and principal)
- Parameters Young’s modulus and tensile strength

For the stresses, the global stresses will be calculated for both the concrete and the steel reinforcement bars.
Chapter 4. Results and discussion

In this chapter, the results as calculated by DIANA will be investigated and discussed. This will be done by discussing the stresses in the concrete and in the reinforcement steel, the crack pattern and the crack strains and the temperature distribution. As mentioned before, this analysis will be split into three parts. In the first part, the influence of a different temperature profile will be checked. In total, six different temperature profiles will be considered: a constant temperature of 17°C, a constant temperature of -17°C and four realistic temperature profiles from the months January, April, July and October 2018 at the location of Brussels. All these profiles will be checked with and without the modelling of active crack control, so that the influence of active crack control can be investigated. This will be the second discussion point of this first part.

In the second part, the influence of a different convection coefficient is examined. This is a very important parameter in the field of the early-age behavior of CRCP, so a clear influence is expected. For this part, two temperature profiles are considered: January and July. In these models, active crack control is modelled.

In the third and final part, the modelling of bondslip will be discussed. This is done in models without active crack control, as will be discussed later on. For this part, a model in which bondslip is modelled is made for four temperature profiles: January, April, July and October.

The results of the temperatures are obtained by performing a transient heat transfer analysis in DIANA, while all the other results concerning crack strain, crack width, stress or tensile strength, are obtained by performing a structural nonlinear analysis.

4.1. Part 1 – external temperature and active crack control

4.1.1. Influence of the temperature profile – no active crack control

4.1.1.1. Temperature

To check the influence of the temperature profile, first the temperature distribution throughout the depth of the slab is examined. This is done for the middle of the slab, so for the point X = 1200 mm and Y = 900 mm. In Graph 12, the temperature profile along the depth at this location is shown
for different time steps. First, this is examined for the constant temperature profile of 17°C and afterwards the other profiles will be examined.

In this graph, it can be seen that after 1 day, there is no influence from the extern temperature profile yet. This is because of the sheeting. As during the first day the convection coefficient is modelled as 0 W/m²/°C, no heat exchange takes place with the surroundings. During this first day, the temperature in the concrete depends on the adiabatic heat development. The temperature raises from 20°C, which is the initial temperature in the concrete, to more than 34°C, due to the heat development because of hydration. After this day, there is influence from the ambient temperature. The temperature in the concrete will now decrease to 17°C. At the surface, the temperature is lower than at the bottom of the concrete slab because of the direct contact with the air. It can be seen that the change in temperature is the highest in the first few days after removing the sheeting. After this, the temperature converges until it reaches a temperature of 17°C in the whole slab.

In Graph 13, the temperature along the depth is shown for the same temperature profile of 17°C, but for more detailed time steps between 1 and 2 days. In this graph, the development of the temperature distribution is shown just after the removal of the sheeting. It can be seen that in the first hours after removal of the sheeting, the temperature in the lower part of the slab still increases, while the temperature at the surface starts to decrease immediately. The reason for this is that the influence of the adiabatic heat development is still present. In the first hours after the sheeting is
removed, the adiabatic heat development still has more influence in the lower parts of the slab than the ambient temperature. It can be said that the ambient temperature has ‘not fully permeated through the slab’. However, when also the lower part of the slab begins to feel the influence of the ambient temperature, the temperature throughout the whole depth starts to decrease towards 17°C.

Graph 13: Temperature along the depth for different time steps – 17°C (2)

That the temperature is not the same along the width of the slab, can be seen in Figure 28. Here, the temperature in the slab can be seen after 2 days.

It can be seen that the lowest temperatures are located at the edge of the slab which connects the two faces of the slab that are in contact with the air. Because of this reason, this location is the one
where the ambient temperature will have the highest influence and thus were the concrete will cool down first.

In Graph 14, the temperature along the depth of the slab can be seen for the constant temperature profile of -17°C, at the same location as before: in the middle of the slab. It is clear that this follows the same reasoning as for the other constant temperature profile. As expected, the temperature after one day is exactly the same as the temperature does not depend on the external temperature yet but only on the adiabatic heat development within the concrete. However, since the external temperature is much lower, the temperature in the concrete decreases a lot faster than was the case with the other constant temperature profile. Another remark that can be made concerning these graphs, is that the difference in temperature between the top and the bottom of the slab, decreases with time. This is the case because in the first few days, the influence of the ambient air is not yet fully permeated to the bottom of the slab.

The temperature along the depth for different time steps for the month January can be seen in Graph 15. The reasoning behind this graph is similar to the reasoning behind the previous graphs. This is the case until 8 days. Then, it can be seen that there is a rise in temperature. This is because of the fluctuating temperature profile: when the temperature in the slab has decreased enough to be around the ambient temperature, the slab will adapt to the ambient temperature that on its turn also increases and decreases. This adaption of the surface can be seen in Graph 16, where the
temperature in the middle of the slab on the surface can be seen in function of time. It should be noted that at the end of the month, the time steps are taken bigger in order for the computational time to decrease, so this temperature is not very detailed then. However, the fluctuating character can clearly be seen on the graph. To illustrate the difference, the temperature is also shown for the constant temperature profile of 17°C. In that case, it can be seen that the temperature converges to 17°C without fluctuating.

Graph 15: Temperature along the depth for different time steps – January

Graph 16: Temperature in the middle of the slab at the surface – January vs. 17°C
For the other months, the temperature in the concrete slab shows the same characteristics as for the month January, but with different temperature ranges. These temperatures can be seen in Graph 17 to Graph 19. In these graphs, the last time step is shown where the temperature at the surface decreases. It can be seen that the colder the temperature range is, the later this time step occurs, which is logical because the temperature must decrease more in that case.
Altogether, it can be said that the temperature within the concrete is clearly influenced by the external temperature profile. This can be seen in the extent to which the temperature decreases, and the velocity of doing so.

4.1.1.2. Stresses and crack patterns

Another very important aspect regarding the initiation of cracks, is the development of the stresses in the concrete and in the reinforcement. These stresses will normally be checked at the location of reinforcement bars. This is logical for the stresses in the reinforcement bars itself, but also the stresses in the concrete will be checked at the location of these bars from top view. The stresses in the concrete will then be checked at a height of 5 mm beneath the surface, or at a height of 5 mm above or below the reinforcement bar. In order to be clear, the numbering that will be used in what follows can be seen in Figure 29. In this figure, the top view is shown. The longitudinal bar closest to the edge is called L1 and so on until the bar located closest to the middle of the slab which is called L10.
In this section, first a general overview will be given of some of the stresses and more specifically on the differences between the different temperature profiles. Afterwards, the stress mechanism and the relation with the cracking will be more extensively discussed for one specific temperature profile. In order to have a perfect understanding in the stresses as shown in the figures, the following should be noted: negative stresses mean compressive stresses and positive stresses mean tensile stresses. Since the tensile strength of concrete is much lower than the compressive strength, it is important to examine the tensile stresses more in detail. These will be more critical and will eventually cause the cracking.

In Graph 20, the stresses can be seen at the location of L4 at the surface of the slab, for the six different temperature profiles, and this after a time of 1 day and 4 hours, which is 4 hours after the removal of the sheeting. This time step is chosen as there is in this time step no cracking yet for each of the different temperature profiles. When cracking is initiated, this would mean a relief of stresses and thus a big influence. In the graph, a clear influence can be seen of the temperature profile. The overall observation is that, the lower the temperature, the higher the stresses.
It can be seen that the stresses of the constant temperature profile of -17°C are the highest, and decrease in the following order: April, January, October, July, 17°C. A first remark is that the stresses for April are higher and thus closer to a tensile stress than January, which means that April is more critical. When looking at the temperature profile, this becomes logical: the temperature until 1 day and 4 hours is lower for April than for January. Overall, when checking the time step when the first cracking shows, the most critical temperature profile can be defined. The moment of the first cracking is shown in Table 7.

**Table 7: Time of first cracking for each temperature profile**

<table>
<thead>
<tr>
<th>Temperature profile</th>
<th>Time of first cracking</th>
</tr>
</thead>
<tbody>
<tr>
<td>17°C</td>
<td>No cracking after 31 days</td>
</tr>
<tr>
<td>-17°C</td>
<td>1 day 7 hours</td>
</tr>
<tr>
<td>January</td>
<td>4 days 19 hours</td>
</tr>
<tr>
<td>April</td>
<td>4 days 19 hours</td>
</tr>
<tr>
<td>July</td>
<td>No cracking after 31 days</td>
</tr>
<tr>
<td>October</td>
<td>20 days</td>
</tr>
</tbody>
</table>
From this table, it can be concluded that the constant temperature profile of -17°C is the most critical, followed by January and April which are equally critical and October. July and 17°C seem not critical in this case as no cracks have shown in one month. The fact that April is equally critical as January is due to the rather low temperatures in the first days of April’s temperature profile of 2018. However, this is realistic as the winter has only just ended when April begins.

Going back to Graph 20, the stresses seem to be constant along the length for each different temperature profile. However, this is not the case, as can be seen in Graph 21.

![Graph 21: Stresses $\sigma_{XX}$ at the surface at location L4 – 1 day 4 hours (January)](image)

In this graph, the stresses are higher and thus more critical at some particular locations along the length: at 500 mm, 1200 mm and 1900 mm. These locations are exactly the locations of the transverse reinforcement bars, so a clear influence of these bars can be concluded. Later on, this will be discussed more in detail. When checking the stresses at another location in the slab, similarly done as in Graph 20, also the composition of this graph is expected to be similar. It will just be a graph with straight lines without being able to notice the deflections for each different temperature profile like in Graph 21, because the differences between them are too high. Therefore, those graphs will not be composed as the only thing that can be learned from it, is which temperature profile is more critical, and this has already been discussed.
What will be checked, is the crack pattern after this is fully propagated. It is chosen to check the crack pattern for each temperature profile – except from 17°C and July as no cracks are shown here – after one month. These can be seen in Figure 30 to Figure 33.

Figure 30: Crack strain pattern after one month – 17°C

Figure 31: Crack strain pattern after one month – January

Figure 32: Crack strain pattern after one month – April
From these figures, the following remarks can be made. At the transverse side of the slab which is in contact with the air, it can be seen that the cracks show at the location of the transverse reinforcement bars at 500 mm and at 1900 mm. In January and April, the cracks propagate at this transverse location throughout the whole width. The similar crack pattern is due to the similar temperatures in the first part of this month. However, it can be seen that the cracks in January are slightly wider than in April, which is due to the colder temperatures in January in general.

For the constant temperature of -17°C, the cracks are the widest. Also, a formation can be seen of two extra cracks along the width in comparison with January and April. These cracks are formed because the stresses between the two other cracks became too high for the concrete to withstand: the distance between these two side cracks is too large for this temperature profile. For the month of October, the opposite can be seen. For this month, the cracks only show after 20 days. This is due to its temperature profile: the temperatures are at first not as low as in January or April, but after 18 to 19 days, the temperature goes below 10°C and stays relatively cold afterwards. This can indicate towards a critical temperature that must be reached in order for the concrete to show cracks, and which is not reached for the month of July and the constant temperature of 17°C.

Altogether, it can be said that the crack patterns are not necessarily random, but by all means very difficult to predict. Regarding this, it is expected that active crack control can help to control the location of the crack initiation and propagation. This will be checked in the following section of this part.
Now, more will be focused on the mechanism behind the formation and the location of these cracks and the link with the stresses. For this, it is chosen to focus on one temperature profile. The temperature profile that is considered is January, as in this month the cracks are showing rather early, but not too early – the first cracks show after 4 days and 19 hours – in order to be able to check the development of the stresses before the cracking for a sufficient amount of time. Also, the crack pattern is clear and not too spread out.

In Graph 22, the stress in function of time can be seen at the location of the right crack, so for X = 1900 mm, right at the surface and at a distance of 500 mm from the transverse free side of the slab. Regarding this graph, the following remarks can be made. First, the stress decreases. This is during the first day, when the sheet is still present. Because of the heat development due to hydration, the temperature in the concrete will increase according to the adiabatic curve. Because of this temperature rise, the concrete will try to expand, but as this is restrained, compression stresses are developed in the X-direction. After one day, the sheeting is removed and the opposite will show. Because of the drop in temperature in the concrete, the compression stress decreases and morphs into tension stress. This tension stress is what is critical in the field of concrete, as the concrete’s tensile strength is rather low. After nearly 5 days, a sudden relief in the tensile stress can be seen. This is the moment when the crack appears: because of this crack, the stresses in the concrete are relieved.

Graph 22: Stress vs. time at the crack location (surface)
When examining the precise moment of the crack, a reference is made to Graph 23, where also the tensile strength of the concrete is shown for this location. It can be seen that this tensile strength is developed during the first days after concrete placement, so during the hardening of the concrete. However, it can be seen on this graph that the stress in the concrete does not exceed this tensile stress. The reason for this is that the crack did not initiate at the location of this graph. The crack initiated somewhere else, which caused the stress to decrease for other positions along the width of the slab too.

![Graph 23: Stress and tensile strength vs. time at the crack location (surface)](image)

To know the precise location of the crack initiation, the stress along the width is studied, and this at the location and at the height of the transverse reinforcement bar T3. It can be seen in Graph 24 that right before the crack initiation, so after 4 days and 14 hours, the stress is maximum at the edge of the concrete slab. For this reason, the cracks initiate here, at the precise location of the transverse steel reinforcement, as the stresses in the concrete are higher at the location of these bars. The crack initiation can be seen clearly in Figure 34. The time step right after this crack initiation, a stress relief can be noticed right at the free edge of the concrete slab. However, as these first cracks are very small, the stress is not fully relieved and a stress peak can be seen at a distance of circa 100 mm from this edge, going over the tensile strength at that time step. This exceedance of the tensile strength causes the crack to propagate and to widen.
The only question left, is why the cracking starts at the precise location of the transverse steel reinforcement. For this, the stress is checked at the location where the first cracks initiate, and this for the time step right before this crack initiation: after 4 days and 14 hours. This can be seen in Graph 25. It is clear that the stress peaks are located right at the location of the transverse steel bars. In concrete, stress concentrations typically form at the location of edges or imperfections. Therefore, the locations of the transverse reinforcement bars are typical locations along the free transverse side of the slab where such stress concentrations appear.
An overall illustration of the stresses in the concrete slab in comparison with the tensile strength of the concrete, is given in Figure 35 and Figure 36. In these figures, it can be seen that, considering the whole slab, the stresses increase towards the free transverse side of the slab, with the maximum stresses located at the transverse bars, while the tensile strength decreases towards the free transverse side of the slab and towards the top surface. This is another indication of the reason why the stresses initiate at these precise locations.

Figure 35: Stress $\sigma_{xx}$ in the slab after 4 days and 19 hours
Another interesting aspect to examine, is the behavior of the longitudinal steel bars in this process. For this, the stresses in a longitudinal reinforcement bar in the X-direction are checked. This can be seen in Graph 26, right before and after the initiation of the cracks.

In this graph, it can be seen that right before the cracking, the stresses are practically equal to zero. At this moment, the reinforcement is not yet fully used. However, right after the crack initiation, high stresses can be seen in the reinforcement. It can be remarked that the highest – positive – stresses are present at the location of the cracks. This is the case because at the location of these
cracks, the reinforcement ‘pulls’ at the both sides of the crack at the concrete, causing the tension in the reinforcement bars. As between the cracks the bar pulls at two different cracks at two sides, there is compression in the intermediate area. At the location of the transverse bar, a stress relief can be noticed: the stress goes to zero. However, the stresses stay widely below the yield strength of the reinforcement steel, being 500 MPa, so no problems are expected regarding the yielding of the reinforcement bars.

4.1.1.3. Crack width

For the investigation of the crack width, again only the month of January is considered. In terms of crack width, the most interesting thing is the evolution of the crack width in time and the difference in this evolution. In Graph 27, the crack width is shown in function of time for different locations in the CRCP slab: at the top for \( Y = 100 \) mm and \( Y = 300 \) mm and at the bottom for \( Y = 100 \) mm, and this at the longitudinal location of the crack (\( X = 450 \) mm). In this graph, it can clearly be seen that for the same transverse location, the crack width at the bottom is lower than the crack width at the surface. The crack width for the other transverse location is a little lower compared with the other location at the top, but the difference is very small. It can also be remarked that the course of the graph is similar for all the locations. This indicates that the crack width evolves in a similar way along the slab.

![Graph 27: Crack width vs. time for different locations – January](image-url)
To check the reason why the crack width is higher at location 2 than at location 1 at the surface and why it is higher at the surface than at the bottom for a fixed transverse location, the crack width across the slab is checked. This is done for several heights: $Z = 250$ mm (top of the slab), $Z = 170$ mm (level of the reinforcement), $Z = 125$ mm (middle of the slab) and $Z = 0$ mm (bottom of the slab), and this at the longitudinal location of the crack ($X = 450$ mm). This is checked after a period of 7 days and 12 hours, when the crack has already propagated over the whole width of the slabs for a few days. These crack widths can be seen in Graph 28. From this graph, it can be seen that the crack width at the surface is quite random. However, at the level of the reinforcement it can be seen that there are drops in the crack width near the locations of the longitudinal steel bars. This is the case because the longitudinal reinforcement bars hold these cracks tightly closed. This influence of the longitudinal steel bars is also noticeable below the level of the reinforcement, but this influence fades away towards the bottom.

Altogether, it can be seen from the graph that in general, the crack width is minimal at the level of the reinforcement, because of the reason that is mentioned before (because the longitudinal reinforcement bars hold these cracks tightly closed), and that the further away from this reinforcement, the higher the crack width becomes. However, the crack width at the surface seems to be higher than the crack width at the bottom until a transverse location of about half the model, even though the surface is closer to the reinforcement. This is probably the case because the influence of the external temperature is higher at this location (near the surface and near the free transverse side), and thus also the stress variation is higher. Finally, it can be seen that the range of the crack width does not vary too much across the width of the slab. It will be interesting to check if this is also the case when active crack control is modelled.
4.1.2. Influence of the temperature profile – active crack control

In this section, the influence of the different temperature profiles on the early-age behavior will be checked, by examining models in which active crack control is modelled. Therefore, the focus in this section will also lie on the influence of active crack control on the stresses and the formation of cracks.

4.1.2.1. Temperature

In order to check the difference in temperature with the case where no active crack control was modelled, it is chosen to check the difference in temperature for the model with a constant external temperature of -17°C, because the temperature differences are largest for this model.

The temperatures for both cases (no active crack control: NAC vs. active crack control: AC) can be seen in Table 8, along the depth and at the middle of the model of the slab (X = 1200 mm, Y = 900 mm), for different time steps. In this table, it can be seen that the temperature differences are very small. After one day, the temperatures are the same for both cases, because until that time, the temperature only depends on the adiabatic heat development due to hydration in the concrete. Also after the sheeting period, the differences in temperature between both cases are very small. This is due to the only difference in the model between both cases: the modelling of the notches.
Due to these notches, there is an additional area which is in contact with the external air and which is thus subject to convection. Because of this, the temperature for the case of active crack control is slightly lower. However, because of the converging character of the temperature, the temperature of \(-17^\circ\text{C}\) will be reached after approximately the same time, which is illustrated by the temperatures for both cases after 15 days.

Table 8: Temperatures for both cases along the depth – middle of the slab

<table>
<thead>
<tr>
<th>Depth [mm]</th>
<th>1 day [(^\circ\text{C})]</th>
<th>2 days [(^\circ\text{C})]</th>
<th>3 days [(^\circ\text{C})]</th>
<th>5 days [(^\circ\text{C})]</th>
<th>15 days [(^\circ\text{C})]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>NAC</td>
<td>AC</td>
<td>NAC</td>
<td>AC</td>
<td>NAC</td>
</tr>
<tr>
<td>50</td>
<td>34.40</td>
<td>34.40</td>
<td>10.33</td>
<td>10.24</td>
<td>-3.22</td>
</tr>
<tr>
<td>100</td>
<td>34.40</td>
<td>34.40</td>
<td>12.21</td>
<td>12.19</td>
<td>-2.28</td>
</tr>
<tr>
<td>150</td>
<td>34.40</td>
<td>34.40</td>
<td>13.54</td>
<td>13.50</td>
<td>-1.62</td>
</tr>
<tr>
<td>200</td>
<td>34.40</td>
<td>34.40</td>
<td>14.28</td>
<td>14.24</td>
<td>-1.25</td>
</tr>
<tr>
<td>250</td>
<td>34.40</td>
<td>34.40</td>
<td>14.74</td>
<td>14.69</td>
<td>-1.02</td>
</tr>
</tbody>
</table>

It can be concluded that the modelling of active crack control only has a minor influence on the temperature inside the concrete, and that the influence of the temperature profile will thus be the same for the case where active crack control is modelled. The most important difference between the case of active crack control and the case without active crack control, will be the stresses in the concrete and the crack pattern, which will be discussed in the next section.

4.1.2.2. Stresses and crack patterns

The stresses and cracks in the case of active crack control will be examined in a similar way as in section 4.1.1.2. However, first the different crack patterns are checked. It can be seen that in all the models, the first cracks initiate after 12 hours. That this is the case for all the different temperature profiles, is normal because during the first 24 hours all the models behave in the same way as the sheeting is still present. However, the cracks are only right at the tip in the early stages after crack initiation, so the stresses can be checked anyway, and this at the same location and at the same time as in section 4.1.1.2. : after 1 day and 4 hours, so 4 hours after the removal of the
sheeting, at the location of reinforcement bar L4, right on the surface of the slab. These stresses can be seen in Graph 29.

![Graph 29: Stresses $\sigma_{XX}$ at the surface at location L4 – 1 day 4 hours – active crack control](image)

From the graph, it can be seen that the stresses are almost identical to the stresses in the case without active crack control. This is because the only cracks that are present at this moment are located at the very tip of the notches. Therefore, one can conclude that the external temperature profile influences the temperature inside the profile and the stresses (cf. previous sections), and that this influence in itself is not affected by whether active crack control is modelled or not. Regarding this, the most important scope of this section is examining the influence of active crack control on the formation and the location of cracks in comparison with the situation without active crack control. After all, the conclusion from section 4.1.1., that the constant temperature profile of -17°C is most critical and the one of 17°C is least critical, can be extended to this section.

In Figure 37 to Figure 42, the crack pattern can be seen for the different temperature profiles after a period of one month, as at that moment, the crack pattern is certainly fully propagated for all of the different temperature profiles. By comparing these crack patterns, some first differences with the case without active crack control can be checked and a proper choice can be made about which temperature profile will be examined further in order to check the difference with the case without active crack control in the field of stresses.
Figure 37: Crack strain pattern after one month – 17°C – active crack control

Figure 38: Crack strain pattern after one month – -17°C – active crack control

Figure 39: Crack strain pattern after one month – January – active crack control
Figure 40: Crack strain pattern after one month – April – active crack control

Figure 41: Crack strain pattern after one month – July – active crack control

Figure 42: Crack strain pattern after one month – October – active crack control
Looking at these figures of the crack strain pattern after one month, the following remarks can be made. While in the case without active crack control, no cracks are initiated in one month for the temperature profiles of 17°C and of July, there are some cracks showing in the case with active crack control. This is only due to the presence of the notches. As is remarked before regarding Graph 29, the stresses in the concrete are nearly identical in the early stages. The initiation of cracks at the tip of the notches, is thus only due to the stress concentrations that typically form at the location of edges and imperfections in the concrete. The fact that the crack patterns are not symmetrical in these cases, can be due to the following: when a crack propagates at one of the two notches, the stress is relieved for that longitudinal location for the rest of the slab. Due to the asymmetrical meshing, it can be possible that the crack propagates initially one time step earlier at the one notch than at the other notch. Because of the relief of stress, it is not necessary for the other crack to propagate further, so only one crack will fully propagate across the slab.

This is also the case for the other temperature profiles. For all those profiles, it is clear that the cracks initiate at the location of the notches. After initiation, it is only a matter of time for the cracks to propagate further across width of the slab. The influence of active crack control is very clear. Another remark can be made regarding January and April. In previous sections, the similarity of those two months was highlighted, and also in this case, a clear resemblance can be observed. The propagation of the cracks can be seen to be very similar – the left crack on the figures even has almost the exact same shape for the two months. Finally, a remark can be made about the constant temperature profile of -17°C. For this temperature profile, a rather irregular crack pattern can be observed. While the initiation of the cracks is also for this temperature profile right at the tips of the notches, some other cracks start to propagate at different locations. The reason for this are the considerably higher tensile stresses for this case due to the cooling of the concrete in comparison with the other temperature profiles. It can be concluded that these stresses are not fully relieved by the two cracks that were formed at the transverse location of the notches. However, looking at the magnitude of the crack strains, it can be seen that the highest crack strains are located at the tips of the notches.

For a further investigation of the stresses within the slab, the temperature profile of January is selected, because of the clarity of the crack pattern. Also, it will be more interesting to compare the case with active crack control with the case without it for one and the same month.
First, to get a view on the stresses throughout the whole slab, the stresses are checked at the location of reinforcement bar L10 right at the surface, so at the location of the bar which is the furthest from the transverse free side. This is checked after a time of 2 days and 16 hours, as on that moment the crack is not fully developed across the whole width of the slab, but there are already considerable tensile stresses in the concrete. These stresses are shown in Graph 30. In this graph, a clear difference can be seen. In the case of no active crack control, the fluctuations of the stress along the length of the slab are very small. However, there are peaks which are not clearly visible in the graph, located at the position of the transvers reinforcement bars, similar to Graph 25. For the case with active crack control, it can be seen that there is a drop of the stresses at the location of the notches. Apparently, these notches are the cause of a relief of stress at these transverse locations which is noticeable throughout the whole width of the slab.

![Graph 30: Stress σxx at the surface at bar L10 – AC vs. NAC](image)

If the stresses are lower at the transverse position of the notches than between those notches, one could argue that it is strange that the cracks propagate at these positions. To get a better understanding of this, the stresses are checked very close to the tips of the notches, at the surface along the length of the slab. This is done for the location Y = 500 mm and after a period of 2 days and 7 hours: right before the propagation of the crack in that location. These stresses are shown in Graph 31.
From this graph, the reason why the cracks initiate at the notches is very clear. The stress peaks are exactly at these locations, so it is only logical that the cracks propagate there. It should be noted that for the case of no active crack control, there are also peaks which are very small in comparison with the peaks for active crack control. These smaller peaks are located at the transverse reinforcement bars, cf. Graph 30. However, it can be seen that the stress peaks are still below the tensile strength in the concrete at this moment, which is circa 2.5 MPa as visible in Graph 23.

To check this, the stresses in function of time are checked very close to the tips of the notches. These stresses are shown in Graph 32, for both cases at the surface at the location of the crack and at a position of $Y = 500$ mm, together with the tensile strength of the concrete, which is practically the same for both cases. From this graph, the following can be remarked concerning the differences between both cases. During the first day and even during the first day after the removal of the sheeting, the stresses are the same in both cases due to the equal distance from the transverse free side. After this period, a peak in the stress can be seen for the active crack control, around the moment of Graph 31. It is at that time that the cracks propagate at that location. After this peak, when the cracks are formed here, there is a relief of tensile stress, because of the mechanism which is discussed before: the tensile stresses that built up because of the contraction of the concrete...
which causes the concrete to ‘pull’ at the constrained edges, are relieved when the cracking propagates at this location.

![Graph 32: Stress and tensile strength vs. time at the crack locations (surface) – AC vs. NAC](image)

To end this section, the stresses in the reinforcement are checked in the X-direction. These stresses can be seen in Graph 33. From this graph, the same remarks can be made as in section 4.1.1.2. with Graph 26. Before cracking, the stresses are practically zero and right after the cracking, higher stresses can be seen in the reinforcement, exactly at the locations of these cracks. That is the reason why there is a shift in stress peaks in comparison with the case without active crack control: simply because there is a shift in position of the cracks. These tensile stress peaks are present because after the cracks are formed, the reinforcement ‘pulls’ at the concrete at both sides of the crack in order to keep these parts close to each other. The compression stresses in between the cracks are found to be smaller for the case with active crack control. This is because the cracks lie closer to each other: the compression area is smaller, so a lower compression stress is reached. At the location of the transverse bar in the middle, also in the case of active crack control a stress relief can be noticed.
4.1.2.3. Crack width

In order to check the influence of active crack control on the crack width, the same graphs are made as in section 4.1.1.3. This is again done for the month of January.

For the first graph, the crack width in function of time is checked. This is done for two locations: at the top right at the tip of the notch (X = 600 mm, Y = 400 mm) and at the bottom as close to these X- and Y-coordinates as possible, regarding the mesh. This can be seen in Graph 34. With respect to this graph, the following remarks can be made. At the top, right at the location of the tip of the sawcut, the maximum crack width is reached within the first days after the crack initiation. Afterwards, the crack width decreases and stabilizes. This is the case because of the presence of the sawcut which is wide in comparison to the crack (4 mm), and because after the crack initiation in this point, the stress is relieved. Because of this stress relief, together with the action of the longitudinal reinforcement, the crack width decreases after reaching this crack width peak. For the bottom of the slab, there is no peak in the crack width. In this point, the crack width increases towards a width which is maintained afterwards, following an evolution similar to the location at the top after the peak. In this case, the crack width is higher at the bottom. This can be due to the distance to the longitudinal reinforcement, but also to the shift in X- and Y-coordinates regarding the mesh.
To make the second graph, the crack width across the slab is checked. This is done for several heights: Z = 250 mm (top of the slab), Z = 170 mm (level of the reinforcement), Z = 125 mm (middle of the slab) and Z = 0 mm (bottom of the slab), and this at the longitudinal location of the crack (X = 1800 mm). This is checked after a period of 7 days and 12 hours, when the crack has already propagated over the whole width of the slabs for a few days and after the same time as checked for the case without active crack control. These crack widths can be seen Graph 35.

From this graph, the same remark can be made as before regarding the reinforcement level: in general, the crack width is minimal at this location. Furthermore, the influence of the active crack control is very clear. For the surface level, the cracking only starts at Y = 400 mm, as here the sawcut ends. For the other levels, the maximum crack widths are located before Y = 400 mm, because of the presence of the sawcut: this sawcut is very wide in comparison with the normal width of cracks (namely 4 mm), which facilitates the formation of wider cracks at this transverse location. The crack width at these levels drops at Y = 400 mm and increases slightly after this point. Only at the bottom level, the crack width stays practically equal after Y = 400 mm, just because it is at the bottom and thus easily for cracks to propagate. In DIANA, the bottom is supported by means of a fixed translation in Z-direction, but not by the modelling of any base.
layer, causing the cracks to be able to propagate easily. For the top surface, the remark can be made that the crack width returns to zero at some distance from the sawcut. This is only the case because the crack is not perfectly straight, and that at this position the crack makes a ‘bend’. This is probably also the reason for the increase for the levels within the slab at some distance from the sawcut.

![Graph 35: Crack width across the slab for different positions after 7 days 12 hours – ACC](image)

**4.1.3. Comparison of the crack propagation**

To end this section, a comparison is given of the crack propagation for both cases (no active crack control vs. active crack control). For the case without active crack control, this is done for the constant temperature profile of -17°C. This is done because with this profile, the cracks initiate after 1 day and 7 hours and at this time, the time steps used in the model are still smaller, so the crack propagation can be seen more detailed. In Figure 43 to Figure 51, the crack propagation can be seen for this temperature profile for the case without active crack control.
Figure 43: Crack strain pattern after 1 day and 7 hours – -17°C – no active crack control

Figure 44: Crack strain pattern after 1 day and 9 hours – -17°C – no active crack control

Figure 45: Crack strain pattern after 1 day and 12 hours – -17°C – no active crack control
Figure 46: Crack strain pattern after 1 day and 14 hours – -17°C – no active crack control

Figure 47: Crack strain pattern after 1 day and 16 hours – -17°C – no active crack control

Figure 48: Crack strain pattern after 2 days and 2 hours – -17°C – no active crack control
Figure 49: Crack strain pattern after 3 days – -17°C – no active crack control

Figure 50: Crack strain pattern after 4 days – -17°C – no active crack control

Figure 51: Crack strain pattern after 10 days – -17°C – no active crack control
From these figures, it can be seen that the cracks propagate along the whole width during the first day after the sheeting is removed. Especially between 14 and 16 hours after the removal of the sheeting, the crack propagates fast. During the second day after the removal of the sheeting, some smaller cracks are formed between the main transverse cracks. After this second day, the crack pattern does not change much anymore.

In Figure 52 to Figure 58, the crack propagation can be seen for the case with active crack control. This is shown for the temperature profile of January, as the crack pattern is clear in that case so a clear view on the crack propagation can be obtained.

Figure 52: Crack strain pattern after 1 day – January – active crack control

Figure 53: Crack strain pattern after 1 day and 12 hours – January – active crack control
Figure 54: Crack strain pattern after 2 days – January – active crack control

Figure 55: Crack strain pattern after 2 days and 12 hours – January – active crack control

Figure 56: Crack strain pattern after 2 days and 19 hours – January – active crack control
From these figures, it can be seen that there are already cracks after 1 day (actually even after 12 hours, as discussed earlier). However, the propagation of the cracks becomes only clear after 2 days, so 1 day after the sheeting is removed. During this second day and the first half of the third day after the sheeting is removed, the crack propagates along the whole width of the slab. Afterwards, the crack pattern does not change much anymore.

When comparing both cases, it can be concluded that the crack propagates faster after the crack initiation in the case without active crack control. The cracks propagate directly across the slab. In the case with active crack control, there seems to be a retardation of about 1 day after the crack initiation. Once the crack begins to propagate faster, the time span to propagate fully across the
slab is similar to that time span in the case without active crack control. When the crack is fully propagated across the slab, the crack pattern does not change too much anymore.

### 4.1.4. Conclusions

About the influence of a different temperature profile on the one hand, and the influence of modelling active crack control on the other hand, the following conclusions can be made.

The temperature profile that is modelled has a clear influence on the temperature within the slab. Roughly, it can be said that the lower the ambient temperature, the faster the cooling of the concrete appears and the colder the concrete gets. Because of this faster cooling, the stresses increase more rapidly and exceed the tensile strength earlier. Therefore, the cracking will start earlier when the ambient temperature is lower. Also, more and wider cracks can be seen for the colder temperature profiles in comparison with the warmer ones. In general, the tensile stresses are highest at the location of the transverse steel bars, so this is where the cracking initiates.

When active crack control was modelled, the same influence of the temperature could be noticed. The difference with the case without active crack control is that now, the cracks initiate at the tips of the notches instead of at the location of the transverse reinforcement bars. This is the case because there is a peak in the tensile stresses right at the tip of these notches. Therefore, one can say that it is easier to control the crack pattern using active crack control, as the cracks always initiate and propagate at the locations of the notches, while without active crack control, the cracks initiate at the location of the transverse reinforcement, but possibly propagate unpredictably.

### 4.2. Part 2 – convection coefficient

In this part, the influence of a different convection coefficient will be checked. As discussed before, the convection coefficient is modelled to be zero during the first day in order to model the presence of sheeting. After this first day, the convection coefficient is modelled to have a constant value of 6 W/m²/°C. In this part, the convection coefficient after the first day will be increased to respectively 8 W/m²/°C and 10 W/m²/°C. This will be done for two of the temperature profiles:
for the month of January and for the month of July, as these months have the most extreme, though realistic temperatures. In this part, active crack control is modelled.

### 4.2.1. Temperature profile: January

First, the temperature profile of January is checked. As the crack patterns after a whole month are very similar, it is more useful to check the crack strain pattern at the early age. In Figure 59 to Figure 61, the crack strains can be seen after 2 days and 2 hours for the three different values for the convection coefficient.

**Figure 59: Crack strain pattern after 2 days and 2 hours – January – 6 W/m²°C**

**Figure 60: Crack strain pattern after 2 days and 2 hours – January – 8 W/m²°C**
Keeping an eye on these figures, some expectations and predictions can be made with respect to the behavior inside the slab. Since the model is identical during the first day for the three situations, the cracking will also for the increased convection coefficient initiate after 12 hours, at the tip of the notches. After this first day, there is a difference in the models: a different convection coefficient. A higher convection coefficient means that the heat which is developed during the first day due to the hydration in the concrete, will faster be exchanged with the ambient air and that the cooling of the concrete will thus happen faster. Because of this, the tensile stresses within the concrete because of the cooling, which develop because of the concrete contraction as explained before, will develop faster. As the tensile strength in the concrete is practically the same for the three cases, the cracks will propagate across the slab faster when the convection coefficient is higher.

To check this prediction, first the temperature in the absolute middle of the slab is checked in function of time. This can be seen in Graph 36. The following remarks can be made: in the first day, the increase in temperature is identical for the different convection coefficients, because in the first day, the three models are identical. After this, the decrease in temperature starts because of the contact with the ambient air. It can be seen that the higher the convection coefficient, the faster this decrease goes. After this cooling process, the concrete continuously adapts to the ambient air. Because of the quickly varying temperature profile, the differences for the different convection coefficients are barely noticeable. It can be concluded that the temperature in the concrete is influenced by increasing the convection coefficient as expected.
After this, also the stresses in function of time are checked, and this at the location of one of the cracks. It is chosen to check this at a location of $Y = 1100$ mm, as at this location the influence of the capricious character of the stresses near the notch is low enough. These stresses can be seen in Graph 37. A similar remark can be made as with the temperature: an identical course during the first day, and after that a faster development of the tensile stresses for a higher convection coefficient, due to a faster cooling. It can be seen that the peaks of the stresses occur at exact the moment when the crack propagates to this location: 2 days and 16 hours for $6 \text{ W/m}^2\text{/°C}$, 2 days and 7 hours for $8 \text{ W/m}^2\text{/°C}$ and 2 days for $10 \text{ W/m}^2\text{/°C}$. Another remark that can be made is that the peak stresses in the graph do not exceed the tensile strength of the concrete. The explanation for the cracks is that the highest stress peaks, that are higher than the tensile strength, are located at the very tip of the crack at that particular moment. This means that a small deflection in X- or Y-direction, which has quickly happened because a node of the mesh should be selected and not a precise point, leads to the missing out of this stress peak on the graph. However, in this case that is not a problem as the goal was to show the difference between the different convection coefficient, which is certainly clear in the graph.

![Graph 36: Temperature in the middle for the different convection coefficients – January](image.png)
Altogether, it can be said that the influence of the convection coefficient is as expected and that the predictions about it as stated in the beginning of this sections, are validated and approved.

4.2.2. Temperature profile: July

For the month of July, it is of course expected that the convection coefficient has a similar influence as for the month of January. To support this prediction, the same graphs are made as in section 4.2.1., at exact the same locations. These can be seen in Graph 38 and Graph 39. Indeed, again the graphs have a similar shape for the three different convection coefficients, with a faster reaction when a higher convection coefficient is modelled. It can be concluded that the influence of the convection coefficient is the same as discussed in the previous section.

Next to this, another remark can be made about this month. Where the temperature in the month of January decreases rather fast after the sheeting is removed, this is not the case in this month. Here, the decrease of the temperature is less steep. The result of this is that the increase of the tensile stress in the concrete is also less steep, which is clearly visible when comparing Graph 37 to Graph 39. Because of this, the cracking at this point only occurs after 8 to 9 days. However, looking at Figure 41, it can be noticed that there is no cracking at X = 1900 mm and Y = 1100 mm.
Indeed, on Graph 39 it can be seen that the stress is not fully relieved to zero. The partial relief in this point is because of the formation of the crack at the other notch at that time. Apparently, when the convection coefficient is increased, the cracks do develop at least to the location of $Y = 1100$ mm. This can be due to the faster development of the tensile stress, but this should be said with some caution. Because of the asymmetrical mesh, it is possible that the crack develops one step faster at the one notch to a particular $Y$-location than at the other notch. When the tensile stresses in the concrete are not that critical, as is the case for July, it is possible that the one crack propagates, while the other crack loses the need to propagate because of the partial relief of stresses. It can thus be said that the fact that the crack propagates to at least $Y = 1100$ mm for the higher convection coefficients, can be attributed to the asymmetrical mesh and its random character rather than to this higher convection coefficient, although this convection coefficient has an influence on the stresses.

To check the propagation of the cracks, the crack strain pattern is checked for the different convection coefficients after one month. These can be seen in Figure 62 to Figure 64. Figure 62 was already given before (as Figure 41), but is given again in order to be clear and complete.

Graph 38: Temperature in the middle for the different convection coefficients – July
Location: surface crack, Y = 1100 mm

Graph 39: Stress $\sigma_{XX}$ at a surface crack for the different convection coefficients – July

Figure 62: Crack strain pattern after one month – July – 6 W/m²°C
It can be seen that for the right notch, the crack only fully propagates for the convection coefficient of 8 W/m²/°C. For the convection coefficient of 10 W/m²/°C, the cracks first propagates, past the location of Y = 1100 mm causing the full relief of stress in Graph 39, but then it stops to propagate. As stated before, this can be attributed to the asymmetrical mesh and its random character, together with the tensile stresses which are not so critical for the temperature profile of July and which make the need disappear for both cracks to fully develop.

4.2.3. Conclusions

About the influence of a different convection coefficient, the following conclusions can be made.
A higher convection coefficient means that the heat which is developed during the first day due to the hydration in the concrete, will faster be exchanged with the ambient air and that the cooling of the concrete will thus happen faster. Because of this, the tensile stresses within the concrete because of the cooling will develop faster. As the tensile strength in the concrete is practically the same for the three cases, the cracks will propagate across the slab faster when the convection coefficient is higher.

Because of this different cooling velocity, also the fully propagated crack pattern itself can differ for a higher convection coefficient. A clear relation cannot be noticed, but it can be said that due to a different cooling velocity, the mechanism of crack propagation is also different, which can result in a different fully propagated crack pattern.

4.3. Part 3 – bondslip

4.3.1. General

In this part, the influence of bondslip modelling will be investigated. This is done according to section 3.3.6., where the modelling of the bondslip between the concrete and the reinforcement steel is explained. Because of the irregular and asymmetrical mesh, it was not possible to model bondslip along with active crack control. Therefore, the bondslip is modelled in the model where active crack control is not considered. For this case without active crack control, the model with bondslip will as thus be compared to the model without bondslip.

Bondslip was modelled for the four temperature profiles of January, April, July and October. It could be seen that the cracks initiated around the same time for both cases. However, when bondslip was modelled, the model was not able to run until the last time step (except for the month of July where no cracks showed). The error message given by DIANA, was the following:

```
SEVERITY : ABORT
ERROR CODE: /DIANA/NL/LB41/0130
ERRORMSG.A: Relative displacement -0.20430 in element 1547 is outside the range for the DUSTS diagram.
Correct your input.
DIANA-JOB ABORTED
```
It says that the relative displacement is outside the range of the friction stress-slip diagram. Indeed: the slip in this diagram goes from -0.203 mm to 0.203 mm. Apparently, the model does not allow further calculations when the relative displacement is out of this range. A solution could be to extend the graph, but it is chosen to respect the reference regarding this graph and not to change it. The most important stresses, those before and just after the crack initiation, can be checked anyway.

In Table 9, an overview can be seen of the moment of first cracking and the moment for which DIANA stopped running. It can indeed be seen that the cracks initiate around the same time for a given temperature profile.

*Table 9: Comparison crack formation no bondslip vs. bondslip*

<table>
<thead>
<tr>
<th>Month</th>
<th>No bondslip</th>
<th>Bondslip</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cracks from time step</td>
<td>Model stopped after</td>
</tr>
<tr>
<td>January</td>
<td>4 days 19 hours</td>
<td>-</td>
</tr>
<tr>
<td>April</td>
<td>4 days 19 hours</td>
<td>-</td>
</tr>
<tr>
<td>July</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>October</td>
<td>20 days</td>
<td>-</td>
</tr>
</tbody>
</table>

In Figure 65 to Figure 67, the final crack strain patterns can be seen. For January, now a single crack can be seen at the location of the reinforcement, but in this case also some other cracks are formed in between the transverse reinforcement bars T1 and T3. Above T2, again no cracks can be seen. April now shows a crack pattern similar to the one of October without bondslip. The crack pattern of October is in this case very simple, showing only one crack right above the transverse steel bars T1 and T3.
Figure 65: Crack strain pattern after 8 days – January

Figure 66: Crack strain pattern after 5 days and 9 hours – April

Figure 67: Crack strain pattern after 27 days – October
4.3.2. Temperature profile: January

In this section, again the temperature profile of January will be discussed more in detail. The first thing that is noticed, is that the temperature is identical to the case without bondslip. This is logical, as the temperature does not depend on how the reinforcement steel and the concrete interact. Also the tensile strength in the concrete is identical for both cases. The difference lies in the stress within the concrete. First, the stress in function of time is checked for the same location: on a place where a crack shows in both cases (at the surface, for X = 1900 mm and Y = 500 mm, so above bar T3). This can be seen in Graph 40.

![Graph 40: Stress and tensile strength vs. time at the crack location (surface) – no BS vs. BS](image)

In this figure, one can see that the stresses are nearly identical in both cases. The stress in the case of bondslip is relieved a little later, as also the cracking shows a few time steps later in that case.

Now, the stress along the length at the location of the free transverse side is checked at the level of the reinforcement. This is done for the same time step as in the case without bondslip right before cracking appeared in that model without bondslip, but also for the time step right before cracking in this case. These stresses can be seen in Graph 41. In this graph, the mechanism of bondslip is clearer. At the time step of 4 days and 14 hours, the stress in the concrete is nearly the same for both cases. However, near the transverse reinforcement bars, the stress peak is much smaller. This is because bondslip between the concrete and the reinforcement steel bars is allowed,
so a part of the stress is relieved in this way. Because of this, the cracking will be initiated later as the stress peaks will not yet exceed the tensile strength of the concrete. The stress after a time of 5 days does exceed this tensile strength, so from that time step, cracks are formed.

![Graph 41: Stress along length at the location of the free transverse side – no BS vs. BS](image)

Finally, the reinforcement stress $\sigma_{XX}$ is checked and compared for both cases. The bar that is checked, is the longitudinal steel bar L3 ($Y = 440$ mm). The same can be remarked as before. On the locations were cracks show, there are peaks in the stress (cf. Figure 65). In between those cracks, the stress is negative, which indicates at compression in the concrete in these parts. These remarks are not new. However, a clear difference can be remarked between the stresses in the case of bondslip and these in the case without bondslip. When bondslip is modelled, both the tensile and the compressive stresses are clearly smaller. This is also a direct effect of the bondslip: because this is allowed between the reinforcement steel and the concrete, the stresses are not only relieved in the concrete, but also in the steel. One could argue that the lower stresses are a result from the presence of more cracks in comparison with the case without bondslip, but this is not the case as can also be seen in the graph. Also the stresses in bar L8 ($Y = 1340$ mm) are shown, so at a transverse location where only two cracks appear, and also these stresses are very low in comparison with the case without bondslip. A similarity between the two cases, is that the compression stress between the cracks is relieved to zero at the location of the middle transverse bar T2, above which no transverse cracks appear in both cases.
4.3.3. Conclusions

About the influence of the modelling of bondslip, the following conclusions can be made.

It could be seen that the temperature inside the concrete and the tensile strength of the concrete are not influenced by bondslip. The difference lies in the stress within the concrete and in the cracking pattern. In the case of bondslip, the cracking shows a few time steps later. This is because bondslip between the concrete and the reinforcement steel bars is allowed, so a part of the stress is relieved in this way. Because of this, the cracking will be initiated later as the stress peaks will not yet exceed the tensile strength of the concrete at the specific time when this happens in the case without bondslip. The stress does exceed this tensile strength some time steps afterwards, so only from that time step, cracks are formed.

Also in the stresses in the longitudinal reinforcement bars, a clear difference can be noticed. When bondslip is modelled, both the tensile and the compressive stresses are clearly smaller. This is also a direct effect of the bondslip: because this is allowed between the reinforcement steel and the concrete, the stresses are not only relieved in the concrete, but also in the steel.

To end, it can be noticed that the crack pattern for a specific month is clearly different in the case where bondslip is modelled. This is due to the different stress relations between the concrete and the reinforcement bars, on which the propagation of the cracks strongly depends.
Chapter 5. Conclusions and recommendations

In this chapter, first the conclusions will be summarized. A summary of the Master Thesis will be given as well as an overview of the most important findings. Afterwards, some recommendations will be given regarding future work on the subject and some aspects that could be investigated further will be highlighted.

5.1. Summary – conclusions

The first objective of this Master Thesis was to develop a comprehensive 3D numerical model in order to simulate the early age cracking in CRCP under environmental loading. In DIANA, the material models of the concrete and the reinforcement steel were set, together with well-chosen temperature profiles and boundary conditions. On this model, a FE analysis was performed in order to study the displacements, stresses and crack patterns in a CRCP segment. The second objective was to check the influence of some important aspects in the field of CRCP. The influence of active crack control on the crack pattern in CRCP was studied, as well as the influence of the temperature profile, the boundary condition regarding convection and the modelling of bond-slip between the reinforcement steel bars and the concrete.

First, an introduction was given about CRCP. The very basics were explained and the history of the use of CRCP in Belgium was discussed. Through the years, three concepts have been used. The current concept is Design Concept 3, which underwent an adaptation in 2010. This current concept is the one that was considered during the research. In the literature study that followed after this introduction, first some different themes were highlighted. These themes were the different base layer types and the influence of the base type, some important considerations on the longitudinal and transverse reinforcement such as the depth, bar size and bar spacing, and the concept of active crack control and the methods to apply this technique.

Afterwards, some previous researches were reviewed and the conclusions obtained in these researches were summarized. Altogether, it could be concluded that there has certainly been some research on this subject, but that not every aspect of it is already studied in every detail. Some researches that have been done included tests on the mechanism of transverse cracking, curling behavior, thermal properties and the effect of sheeting. Different aspects of transverse cracks were
examined, such as the crack width, crack spacing, punchouts and active crack control. Furthermore, some FE models have been made in different software programs, in order to investigate aspects as active crack control, punchouts and the relation between transverse cracking and the transverse reinforcement bars. The previous investigations on the cracking itself were numerous, but researches about the influence of some aspects such as boundary conditions or the modelling of bondslip, which will be studied in this research, were rather scarce. In addition, a FE study of CRCP through a 3D model made using DIANA, seemed not to be done yet. Therefore, it can be said that the subjects investigated in this research are a valuable addition to the existing knowledge of CRCP.

In the next part, the methodology was discussed. The concept of finite element modelling in general and the use of the software program DIANA in particular were explained in detail. Also the choice for every input parameter in DIANA was explained and motivated. These input parameters included the geometry of the concrete slab and the reinforcement bars, the material aspects of the steel (yielding) and of the concrete (concrete model code, shrinkage, creep and heat flow), the convection coefficient, the external temperature profile and the boundary conditions to model the supports. Also the modelling of bondslip was explained, which was done in some of the models. Afterwards, the meshing was discussed and the settings for the analysis, including a transient heat transfer analysis and a structural nonlinear analysis.

Next, the results could be studied and investigated. This was done by discussing the stresses in the concrete and in the reinforcement steel, the crack pattern and the crack strains and the temperature distribution.

In the first part the influence of a different temperature profile was checked. Some different profiles were checked with and without the modelling of active crack control, so that the influence of active crack control could also be investigated. It was found that the temperature profile has a clear influence on the temperature within the slab. Roughly, it can be said that the lower the ambient temperature, the faster the cooling of the concrete appears and the colder the concrete gets. Because of this faster cooling, the stresses increase more rapidly and exceed the tensile strength earlier. Therefore, the cracking will start earlier when the ambient temperature is lower. Also, more cracks can be seen for the colder temperature profiles in comparison with the warmer ones. When active crack control was modelled, the same influence of the temperature could be noticed. The difference
with the case without active crack control is that now, the cracks initiate at the tips of the notches instead of at the location of the transverse reinforcement bars. This is the case because there is a peak in the tensile stresses right at the tip of these notches. Therefore, one can say that it is easier to control the crack pattern using active crack control, as the cracks always initiate and propagate at the locations of the notches, while without active crack control, the cracks initiate at the location of the transverse reinforcement, but possibly propagate unpredictably.

In the second part, the influence of a different convection coefficient was examined. For this part, two temperature profiles were considered: January and July. In these models, active crack control was modelled. It could be concluded that a higher convection coefficient results in a faster propagation of cracks. This is the case because a higher convection coefficient means that the heat which is developed during the first day due to the hydration in the concrete, will be faster exchanged with the ambient air and that the cooling of the concrete will thus happen faster. Because of this, the tensile stresses within the concrete because of the cooling will develop faster.

In the third and final part, the modelling of bondslip was discussed. This was done in models without active crack control, because of the irregular and asymmetrical mesh, which made it impossible to model bondslip along with active crack control. It could be concluded that in the case of bondslip, the cracking shows a few time steps later. This is the case because bondslip between the concrete and the reinforcement steel bars is allowed, so a part of the stress is relieved in this way. Because of this, the cracking will be initiated later as the stress peaks will not yet exceed the tensile strength of the concrete at the specific time when this happens in the case without bondslip. The stress does exceed this tensile strength some time steps afterwards, so only from that time step, cracks are formed.

Altogether, a clear and explainable influence was noticed for each of the investigated aspects: higher tensile stresses and thus more cracks in the early stages for a colder external temperature profile, faster crack propagation for a higher convection coefficient because of the faster cooling and smaller stress peaks and thus a delayed crack initiation when bondslip was modelled. These are the main conclusions of this Master Thesis and as thus, it can be said that the initial objectives as discussed in the first chapter have been achieved.
5.2. Recommendations – future work

In the final section of this Master Thesis, some recommendations are given regarding future work in the field of CRCP. Some suggestions are given for future researches on CRCP, as well as things that could be adapted in the model.

To start, it is suggested to perform site tests on actual CRCP sections. This is necessary in order to fully understand the actual cracking behavior of CRCP, as a simulation using software always has its limitations. In future researches, it is recommended that software models are used in combination with field tests.

Also regarding the model itself, some recommendations can be made. In this research, the temperature profiles that were considered were temperature profiles of one specific month. It goes without saying that a temperature profile of one specific month can differ much for different years. Therefore, it is suggested to combine the temperature profiles of different months in order to consider this from a more general point of view, by simply taking the average of three consecutive months. In this research, this combination of temperature profiles has been done (for January until March, April until June, July until September and October until December), but it was decided not to use these as the influence of the temperature was already made clear. Another combination that can be made is a combination of the temperature profiles of the same month over several years.

Furthermore, in this model, the bottom of the slab is supported by applying a boundary condition, in order to model the base layer. However, it can be suggested for future researches that the frictional slip between the concrete slab and the base layer is modelled, by applying an extra connection property assignment to the bottom face. With respect to this aspect, the curling up behavior can be investigated.

Another recommendation for future researches, is to focus on the active crack control. As this is a very important aspect in the field of CRCP, it is desirable to investigate this more detailed. Examples of aspects that can be studied, are the optimal timing, spacing, length and width of the sawcuts, or the influence of applying the sawcuts under an angle with the edge of the slab. Another remark regarding active crack control, is that the sawcut normally has a rounded shape at its end, while in this research, the sawcuts were modelled having a rectangular shape. It is recommended to take this into account in future researches.
Last but not least it is emphasized that the aspects related to heat flow and restraints within the concrete slab are very important in the field of the early-age behavior of CRCP. Therefore, it is crucial to get a good understanding of these aspects in order to be able to describe the early-age behavior as detailed as possible. These aspects include the heat development within the concrete due to hydration, the pavement moisture and the related evaporation cooling, the convection between the concrete and the ambient air and the mechanisms of creep and shrinkage within the concrete.

Even though concrete is sometimes considered to have a capricious nature, and even though it is argued that one will never fully understand the mechanisms that are behind its behavior, it is of vital importance that the knowledge of this material is expanded as much as possible. Concrete is everywhere and concrete roads can take us everywhere. When placed properly.
References


Early-age behavior of continuously reinforced concrete pavements (CRCP)

Freek Speleman
Student number: 01300156

Supervisors: Prof. dr. ir. Hans De Backer, Prof. ir. Pieter De Winne

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