Temperature fluctuations and concrete pavements

The influence of temperature fluctuations on the design of continuously reinforced concrete pavements

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Summary

Pavements are used everywhere, and are of a major importance in the infrastructure. However, conditions for which these pavements are designed for and built in are rapidly changing due to climate change. The trends observed in the Netherlands suggest a maximum temperature increase of 3 degrees Celsius and an increase of the average difference between day and night temperatures of 0,6 degrees Celsius in the coming 60 years.

Rigid (concrete) pavements will be influenced by this change, since temperature gradients, caused by temperature variations, exert a normal force. This normal force can manifest itself through tensile stresses in the pavement construction, due to the pavements shrinking while cooling down and expanding while warming up. These temperature related tensile stresses combined with the tensile stresses exerted by the traffic on the pavement are dealt with in different ways depending on the type of rigid pavement that is used. Jointed concrete pavements give room for pavements to marginally expand and shrink in the joints between the different plates, while reinforced concrete pavements use reinforcement to counteract the tensile stresses. For highways a continuous pavement is often used for highways, since it eliminates joints which improves riding comfort and it almost does not require any maintenance, avoiding construction works over almost the entirety of its designed lifetime. These longer stretches of pavement however do need reinforcement to restrain the tensile stresses, which yields the name Continuously Reinforced Concrete Pavement. During construction, the development of the tensile stresses and tensile strength of the pavement is crucial in forming the cracking pattern that arises when the tensile stresses exceed the tensile strength of the pavement. Lower temperatures can result in a slower development of the tensile strength, resulting in more cracks since the tensile stresses will more often exceed the tensile strength, while higher temperatures could result in the opposite effect. The higher the tensile strength and stiffness of the concrete is when cracking, the bigger the crack width will be. Controlling this crack width is of high importance, because otherwise the pavement could become vulnerable to for example chemicals penetrating into the pavement construction.

Several methods are used to design continuously reinforced concrete pavements. FLOOR 3.0 is an example of a design method that is mostly focused on concrete floors inside buildings, while VENCON 2.0 is an example for the construction of highways. In these methods, all the variables used to calculate the pavement thickness can be categorized in five different categories:

- Traffic loads
- Traffic stresses
- Temperature stresses
- Materials properties
- Foundation

The traffic loads and foundation are used to calculate the *expected* axle load repetitions that a pavement needs to be designed for. The traffic stresses, temperature stresses, materials used and foundation are used to calculate the *allowed* axle load repetitions. These are then combined to make sure the pavement meets the requirements.
Temperatures are accounted for in the design using temperature gradients, from the top of the pavement to the bottom and cause a tensile stress. Several temperature gradients, which all occur at different frequencies, are used to represent reality. In other words, small gradients occur more often than large gradients, and both are considered in the design.

The relationship between temperature gradients, caused by daily temperature fluctuations, and pavement thickness is analyzed using a developed model based on VENCON 2.0, which is deemed to be most applicable in this thesis. Both an increase in temperature gradient as an adjustment for the frequency distribution, making bigger gradients occur more often and smaller gradients less, cause

the pavement thickness to increase. A direct relationship is not found between daily temperature fluctuations and temperature gradients, but if the established trend of the temperature fluctuations occurs for the used standard temperature gradient as well, the pavement thickness will increase by 1-2 millimeter. However, if the standard temperature gradient, used in the model, is not representative, then the increase of thickness could be bigger. Additionally, extreme combinations of traffic and temperature stresses could increase the minimum thickness of pavement constructions as well, since extreme conditions are bound by a boundary condition. This condition requires the pavement thickness to be increased if the ratio between the maximum stresses and bending tensile strength gets too large. This minimum thickness primarily occurs for pavements with less than $1*10^7$ axle load repetitions.

Samenvatting

Verhardingen zijn overall, en zijn van groot belang in de infrastructuur. Echter, de omstandigheden waar deze verhardingen voor zijn ontworpen en waarin ze zijn gemaakt veranderen snel door de klimaatveranderingen. De tendens van deze ontwikkelingen in Nederland suggereert een toename van de dagelijkse maximum temperaturen van 3 graden Celsius en een toename van het gemiddelde verschil tussen dag en nacht van 0,6 graden Celsius in de komende 60 jaar.

Rigide (betonnen) verhardingen zullen worden beinvloed door deze verandering, doordat een temperatuur gradienten in het beton, veroorzaakt door temperatuur variaties, kan ontstaan. Deze temperatuur gradient zorgt voor een normaalkracht in de verharding, die zich kan uiten in een trekkracht, doordat de verharding uitzet tijdens het opwarmen en krimpt tijdens het afkoelen. Met deze trekkrachten door temperatuur en trekkrachten door een verkeersbelasting wordt op verschillende manieren mee rekening gehouden, afhangend van de soort verharding. Ongewapende verhardingen maken gebruik van ofwel betonstenen of betonplaten en zijn verbonden door middel van een voegconstructie die ruimte geeft voor uitzetten en krimpen. Gewapende verhardingen gebruiken wapening om de trekkrachten door uitzetten en krimpen op te vangen. Voor snelwegen is een doorgaand gewapende betonverharding vaak gebruikt, doordat er geen voegen zijn, waardoor het rijcomfort verbeterd en er weinig tot geen onderhoud nodig is, waardoor er geen wegwerkzaamheden zijn. Deze lange stukken verharding hebben echter wel wapening nodig om de trekkrachten tegen te gaan, waardoor ze aan hun naam “doorgaand gewapend beton” komen. Tijdens de uitvoering is de ontwikkeling van de trekkrachten en treksterkte van een verharding cruciaal omdat deze het scheurpatroon bepalen van de verharding. Een scheur komt voor wanneer de trekkrachten de treksterkte overschrijden. Wanneer er lagere temperaturen zijn tijdens de uitvoering zal dit ervoor zorgen dat de treksterkte minder snel toeneemt en er dus meer scheurvorming is, terwijl bij een hoge temperatuur de treksterkte sneller zal toenemen en er dus minder scheurvorming zal zijn. De combinatie van hogere treksterkte en stijfheid van de betonverhardingen tijdens het scheuren zorgt ervoor dat de scheurwijdte toeneemt. Het beheersen van het scheurpatroon is van groot belang omdat bij een te grote scheurwijdte de verharding kwetsbaar kan worden voor bijvoorbeeld binnendringende schadelijke stoffen.

Verschillende methodes worden gebruikt voor het ontwerpen van doorgaand gewapende verhardingen. FLOOR 3.0 is een voorbeeld van een ontwerpmethode die zich voornamelijk toespitst op (gewapende) betonnen werkvloeren in gebouwen, terwijl VENCON 2.0 een voorbeeld is van een programma dat wordt gebruikt bij het ontwerp van snelwegen. In deze ontwerpmethodes kunnen de (input) variabelen worden ingedeeld in vijf verschillende categoriën:

- Verkeersbelastingen
- Verkeersspanningen
- Temperatuurspanningen
- Materiaal eigenschappen
- Fundering

De verkeerbelasting en fundering worden gebruikt voor het berekeningen van de verwachte aantal aslastherhalingen, waar de verharding voor moet worden ontworpen. De verkeersspanningen, temperatuurspanningen, materiaal eigenschappen en fundering worden gebruikt voor het berekenen van het toegestane aantal aslastherhalingen. De toegestane en verwachte aantal herhalingen worden dan gecombineerd in een functie om ervoor te zorgen dat de verharding voldoet aan de vereisten.

Temperatuursvariaties worden meegenomen in het ontwerp door middel van temperatuurgradiënten, van de bovenkant naar de onderkant van de verharding. Verschillende temperatuurgradiënten worden gebruikt en komen elk in verschillende frequenties voor. In andere

woorden, kleine temperatuurgradiënten komen vaker voor dan grote temperatuurgradiënten en beiden worden ze meegenomen in het ontwerp.

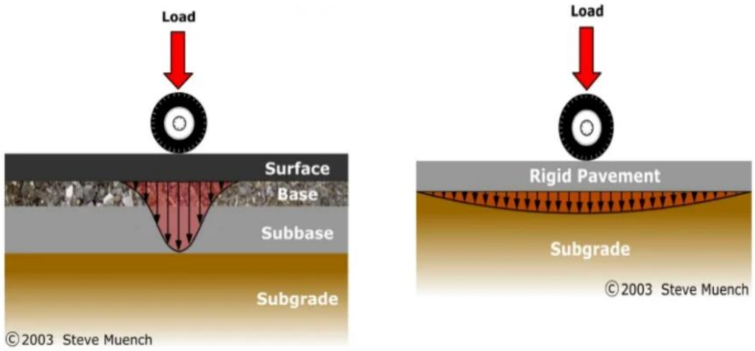
Het verband tussen temperatuur gradienten, veroorzaakt door temperatuurvariaties, en verhardingsdikte is geanalyseerd door middel van een ontworpen model gebasseerd op VENCON 2.0, het model dat het meest toepasbaar was voor dit onderzoeksrapport. Zowel een toename van de temperatuurgradiënt als een aanpassing van de gradiënt frequentie, waardoor grotere gradienten vaker voorkomen en kleinere gradienten minder vaak voorkomen, zorgen ervoor dat de verhardingsdikte toeneemt. Een direct verband is niet gevonden tussen de dagelijkse temperatuurveranderingen en temperatuurgradienten in verhardingen, maar als de waargenomen ontwikkeling van de temperatuursvariaties wordt gebruikt voor het aanpassen van de standaardgradiënten, dan is de verwachten dat de verhardingsdikte toeneemt met 1 à 2 millimeter. Echter, de standaargradiënten, die gebruikt worden in het model, hoeven niet representatief te zijn voor de werkelijkheid, wanneer dit niet het geval is en de gradiënten in werkelijkheid groter zijn, dan kan de toename van de verhardingsdikte hoger uitvallen. Extreme combinaties van verkeersspanningen en temperatuurspanningen kunnen tevens zorgen voor een toename van de minimumdikte van betonverhardingen, doordat er met deze extreme combinaties rekening wordt gehouden door middel van een randvoorwaarde. Deze randvoorwaarde zorgt ervoor dat wanneer de verhouding tussen de maximale spanningen en de buigtreksterkte te groot wordt, de verhardingsdikte moet worden verhoogd. Dit is voornamelijk van toepassing bij wegen met minder dan $1*10^7$ aslastherhalingen.

Contents

1. Introduction

1.1 Pavements

Pavements are used everywhere, whether it's a pavement for walking, riding bikes, or driving motor vehicles, pavements are there to make travel more comfortable. Understanding how pavements work is of vital importance when the perfect balance is needed between factors like costs, sustainability, users ('s comfortability), or usage (types of traffic).
There are three general types of roadway pavements, flexible pavements, rigid pavements, and composite pavements. Flexible pavements typically consist of asphalt concrete, rigid pavements of Portland cement concrete, and composite pavements are a mix of the two, mostly occurring from rehabilitation works with a different type of pavement than was initially used during construction. The terms flexible and rigid relate to the way asphalt and concrete pavements, respectively, transmit stress and deflection to the underlying layers. The stress and deflection distributions in asphalt concrete and Portland concrete pavements depend on the relative stiffness of these layers with respect to those of the underlying granular layers. This ratio is much lower for asphalt concrete than Portland concrete, which justifies their generic designation as flexible and rigid, respectively. (Papagiannakis & Masad, 2008)



Flexible pavements Rigid pavements

Figure 1 Typical cross section of flexible and rigid pavements and the differences in transmitting stress/deflections to underlying layers (Kant, 2016)

During the design of a pavement structure, several factors are being taken into account, such as: traffic loads, pavement performance, materials, costs, and the environment. One of those factors, the environment, has been changing over the last few decades, which might be the cause for complications in the design of pavements.

1.2 Climate change

In the last 650.000 years there have been seven cycles of glacial advance and retreat, with the abrupt end of the last ice age about 11.700 years ago marking the beginning of the modern climate area. However due to the increased output of greenhouse gasses in the atmosphere, this natural cycle has been interrupted. (Figure 2) As has been demonstrated in the mid-19th century, greenhouse gasses have a heat-trapping nature, preventing heat radiated from Earth's surface from escaping into space as freely as it used to. (Climate Change: Ocean Heat Content, 2018)

Figure 2 CO2 in the atmosphere over the past 800.000 years (What is Climate Change?, 2022)

The effects of climate change have already had observable effects on the environment. Glaciers have shrunk, ice on rivers and lakes is breaking up earlier, plant and animal ranges have shifted and trees are flowering sooner. Climate change is resulting in the planet's surface temperature rising about 1 degree Celsius since the late 19th century. And with most of the warming occurring in the past 40 years, it is expected that these temperatures will continue to rise for the foreseeable future. These changes in global surface temperature are illustrated in Figure 3, where the temperatures over the last 140 years are shown relative to the 1951-1980 average temperatures. (What is Climate Change?, 2022)

Additionally, there also appears to be a consensus emerging that local variability in certain regions (amongst which is western Europe) has been increasing in the past 40 years as well. Meaning that extreme temperatures will occur more often than in the past. (Vincze, Borcia, & Harlander, 2017)

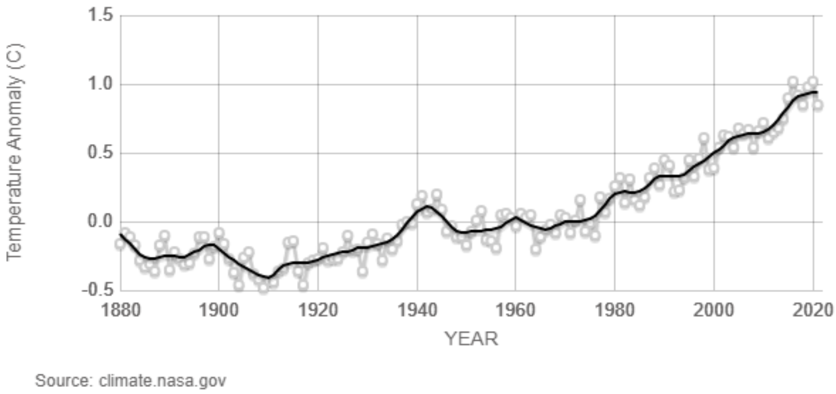


Figure 3 Global land-ocean temperature index (What is Climate Change?, 2022)

1.3 Flexible Pavements and temperature

The temperature of an asphalt mix is a determining factor of the performance of a flexible pavement. The temperature changes the properties of the asphalt mix, and thereby its response to traffic loads. This dependence is described by the elastic modulus of an asphalt layer, which can be determined as a function of a pavement temperature according to several equations:

$$AASHTO: E_1(t) = 15000 - 7900 * log(t)$$
$$UTM: E_{AC-ATB}(t) = 14225 - 3636 * \ln{(t)}$$

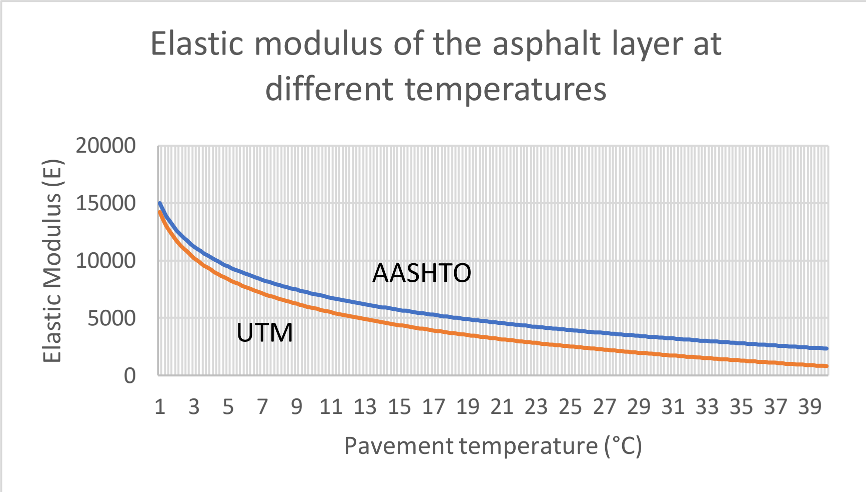


Figure 4 Elastic modulus of an asphalt layer at different temperatures (Tatam, Hainin, Yusoff, Wu, & Nayan, 2013)

As can be seen in Figure 4, the elastic modulus (E) of an asphalt mix decreases with an increasing pavement temperature, which modifies the stress statues inside the pavement. This alteration in stress can influence the stiffness of the underlying unbound layers, since they generally show stress reliance. This together leads to the structural bearing capacity being reduced. This will significantly affect pavement performance, and will thus lead to different pavement structures, to ensure performance. (Tatam, Hainin, Yusoff, Wu, & Nayan, 2013) and (El-Maaty, 2017)

INTERMEZZO (Higashiyama, Sano, Nakanishi, Takahashi, & Tsukuma, 2016)

Higashiyama et al. have researched several asphalt pavements, making field measurements of the road surface temperature under certain temperatures. The asphalt pavements were water retaining with several cement-based grouting materials poured into the voids of open graded asphalt pavements (porous asphalt pavements) to reduce the surface temperature in the hot summer climate. It was established that the water retaining pavements (UCZ and UCF in Figure 5) reduced the surface temperature of the pavement with at least 10 °C, compared to the porous asphalt pavement without the cement-based grouting materials. (PoAs in Figure 5)

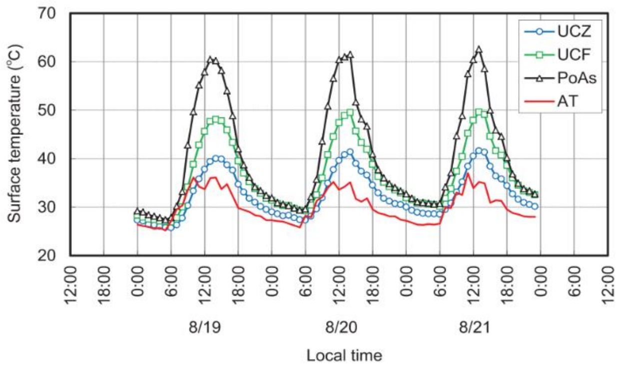


Figure 5 Surface temperature distributions of several asphalt pavement structures for three days in August (Higashiyama, Sano, Nakanishi, Takahashi, & Tsukuma, 2016)

1.4 Rigid Pavements and temperature

Rigid pavements are also influenced by temperature (fluctuations). A typical section of a rigid pavement consists of a subgrade layer (existing soil), base layer such like a sand bed (optional) and a concrete layer. Concrete slabs, such as shown in Figure 6, tend to crack transversely due to volumetric changes caused by cement hydration, external drying and thermal effects. These stresses can be restrained in several ways using cracks, joints and reinforcement.[1] These ways of restraining the shape the common types of concrete pavement: jointed plain (JPCP), jointed reinforced (JRCP), continuously reinforced (CRCP).

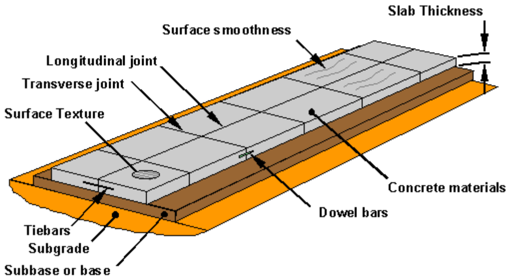


Figure 6 Typical section of a rigid pavement (Concrete Pavement Fundamentals, 2022)

1.5 Rigid pavements or flexible pavements

The decision to either use a rigid or flexible pavement is made by (all sorts of) clients and they have their own arguments to support those decisions.
For example, in the Netherlands the preferred construction is a flexible pavement. Noise disturbance is a big criterion in this decision, since flexible pavements produce less noise when traffic rides over it. Additionally, the government also highly values the drainage system of pavements, and in ZOAB (very open asphalt concrete, a flexible pavement) the drainage capacity is much higher than in a concrete pavement. This prevents spray of water behind vehicles driving over the pavement, in contrast to concrete pavements, where there often is a lot of spray. Finally, the subsoil also is a factor in the Netherlands, where settlements occur often, which complicates things with concrete pavements, since those often respond to (uneven) settlements by cracking, significantly decreasing their lifespan.
In Belgium however, they often choose rigid pavements over flexible pavements, since they have a different philosophy regarding those topics. Noise is often seen as a result of a decision made by people living nearby highways. In other words, if you choose to live near a highway, you choose for more noise disturbance. Additionally, in Belgium the drainage is less important, since rain is seen more as a rare occurrence, and they don't want to invest significantly more funds for something that doesn't occur as much. (On average it rains 7.6% of the time in Flanders (Boosere, 2010)) Finally, there also is a different maintenance strategy in Belgium. In the Netherlands regular (small) maintenance is done to preserve the quality of the road, where in Belgium, maintenance is avoided as much as possible. Whenever the pavement is at the end of its lifetime and maintenance is unavoidable, the pavement is replaced in its entirety, which also favors rigid pavements, since those often don't need any maintenance. Hence, there are relatively more rigid pavements in Belgium than in the Netherlands.

In this thesis, the role of temperatures will be further investigated for rigid pavements. Rigid pavements have been chosen over flexible pavements for several reasons. First of all, rigid pavements are more durable, due to the relatively low amount of maintenance needed after construction. Secondly, rigid pavements are more environmentally friendly than flexible pavements, because of the materials of flexible pavements being finite, which isn't the case for rigid pavements. Finally, this thesis will primarily focus on the design of highways, which are characterized by a high traffic load. Rigid pavements are better suited to deal with this higher traffic load.

[1] The stresses are referred to as restraint stresses and increase more rapidly than the strength of the concrete at early ages of the concrete pavement.

1.6 Problem statement

The increasing temperatures (fluctuations) will start influencing the design and performance of pavement structures. Different pavement types respond differently to the higher temperatures (fluctuations), meaning some pavement structures might become more viable in the future, while others become less. The role of temperatures (fluctuations) will be investigated in this thesis by means of discussing several design methods for rigid pavements. These design methods will then be used to develop a new design method, in which the role of temperatures is expanded to better reflect the current and future climate. Additionally, the consequences of the changing climate will also be highlighted through a comparison of pavement thickness in the current climate and expected future climate.

1.7 Research question

In this thesis, an analysis will be made of the effects of increasing temperature (fluctuations) on continuously reinforced concrete pavement by answering the following research question:

What is the effect of increasing environmental temperatures and temperature variations in the next 60 years on the design of continuously reinforced concrete pavements used for constructing highways?

To answer this research question, several sub questions have been formulated. These sub questions will all be answered in the report. The sub questions have been formed to answer the research question in two phases. The first three research questions are aimed at understanding the current situation regarding climate change and continuously reinforced concrete pavements and have been addressed in the theoretical framework. The following three questions focus on combining climate information with the current design of continuously reinforced concrete pavements and finally make a model that takes into account climate change while designing continuously reinforced concrete pavements.

- What is the expected temperature rise and what are the expected temperature fluctuations, due to global climate change in Western Europe?
- What is a continuously reinforced concrete pavement and how does it differentiate from other pavement structures?
- Which design methods/principles are being used for continuously reinforced concrete pavement?

- What is known of continuously reinforced concrete pavement in other (warmer) climates?
- Which variable are temperature and temperature fluctuations in the design of continuously reinforced concrete pavement?
- What variables are needed to calculate a continuously reinforced concrete pavement, while taking into account climate change?

- How can a model for calculating continuously reinforced concrete pavements be created which takes into account global climate change?

1.8 Phoenix Engineering

Phoenix Engineering is an international engineering company that calculates constructions and does follow up for infrastructure projects (airports, highways, business parks, port sites, marina’s), buildings for public use (hotels, company offices), and private buildings (reconstruction of houses). Currently, most of the work consists of project site management, however, calculating constructions and pavement structures are markets that phoenix engineering wants to enter in the future. However, to do so, an in-company structure is needed to make the work profitable/efficient, for which, for example, pavement calculation models could be an important factor. It is the goal of the company to have a working model for rigid pavements, that is future proof (and thus takes into account climate change) to give adequate advice to clients, when it comes to consulting on projects. One of these projects will be highlighted in the case study.

2. Theoretical framework

2.1 Rigid pavements

Concrete block pavement

Concrete block pavements are used in urban streets, where cables and pipes are underneath the roads and water run-off is minimalized by the ability of water being able to infiltrate in between the concrete elements. Especially on sandy sub soils, where the permeability coefficient is relatively high (1*10-3 m/s), concrete pavements are resulting in a high drainage capacity and less water run-off and risk of overloading the capacity of sewer systems. The thickness of these blocks is in general between 8-10 cm.

Besides uses in urban streets, concrete block pavements are used in port areas, where heavy loads occur. In a normal pavement, the strong materials (with high elastic modulus) are on top of the pavement, and underneath weaker (=less expensive) materials are used. However, on ports the surface layer can incidentally be damaged (falling weight), to avoid high maintenance costs, block pavements are used, because they are easily and quickly be replaced, so the terminal remains reliably accessible. The form of the blocks is often deviating from straight surfaces to increase the interlocking effect between the blocks. The thickness of this blocks is between 12-14 cm.

Figure 7 Concrete block pavements Urban area (Egyed, 2022)

Figure 8 Concrete block pavements on ports (Sieglen Jr. & Langsdorff, 2004)

Concrete slabs with side support from steel U-profiles

These concrete slabs are often used for temporary pavements where heavy loads can temporarily occur. For example, when a windmill is constructed during the construction stage, but also for stacking of temporary loads for logistic operations. The slabs are relatively easy to place and re-use in other areas.

Concrete reinforced slabs

Most of the concrete structures are concrete slabs that are reinforced on the sides by dowel bars (in the longitudinal direction) and tire bars (in the transverse direction). The sides of the slabs are the weakest part of this type of the pavement, in case a high load is on the side of the slab, the reinforcement is used to transfer the load on an edge to the connecting slab.

Concrete reinforced slabs are used on airport aprons, due to the high loads, warm temperatures, impermeability of the concrete material and chemical resistance against oil and deicing fluids that occur on aprons. Additionally, concrete reinforced slabs are used in case the subsoil is a sandy subsoil and relatively high loads are expected (agricultural roads). Furthermore, concrete slabs are used in case torsion forces are expected due to heavy duty traffic (roundabouts, traffic lights, stop signs).

Figure 9 Highways in US, near NY (O'hare, 2022)

Figure 10 Apron Schiphol AMS airport, Heavy loads, potential oil spills (Veldmeijer, 2015)

Continuously reinforced concrete pavements

In the US and Belgium, continuously reinforced concrete pavements are used on highways with high traffic intensity in combination with a high percentage of high loads (trucks). The lifetime of these roads is long and maintenance is minimized due to the fact that no joints are necessary. In the Netherlands, continuously reinforced concrete is also used on highways due to the fact that concrete reinforced slabs cannot be used in case an open asphalt layer is applied above the joints between the concrete slabs. Due to small movements, reflective cracks would occur in the asphalt, which will result in a damaged asphalt surface layer. (Egyed, 2022)

Figure 11 Ring Road Antwerp Continuously concrete (High intensity (+/- 200.000 pae/day and high load traffic 20% trucks) (Volkskrant, 2015)

2.2 Rigid pavements – design

Concrete block pavement

The structural design of concrete block pavement is based on the occurrence of deformations in the wheel tracks, (more than 20 mm) in case there is an unbound base (f.e sand or gravel). In case there is a bound base (sand cement stabilization) the structural lifetime is based on the occurrence of structural cracks.

To avoid deformation on top of the unbound base, the bearing capacity must be high enough to bear the traffic loads to the subsoils. Based on the acceptable level of deformation, the N_{eq} value (the amount of equivalent standard axle loads during the lifetime of the road) and the subsoil the thickness of the unbound base, will be determined. The deformation is calculated in a visco-elastic multi-layer program and used in an empiric fatigue formula to determine the N_{eff} value (the maximum amount of equivalent standard axle load repetitions). N_{eff} should always be higher than N_{eq}, otherwise the deformation will be more than 20 mm during the desired structural lifetime.

To avoid cracks the strains will be determined via the same variables as mentioned for an unbound base. Based on a certain thickness for the bound base the strains will be calculated in a visco-elastic calculation program. The strain will be used as a variable in the specific fatigue relation for the bound material, which results in a N_{eff} value. In case the N_{eff} is higher than the N_{eq} value, the thickness of the layer is high enough to assure that the strains will not exceed a value, which would cause a lower N_{eff} value.

Concrete slabs

The difference with the concrete block pavement is that for this type of pavements, the cracks would occur in the concrete slab material. In this case, the maximum wheel load, resulting in a maximum tensile stress in the middle of the slab, should not exceed the maximum bending stress.

Concrete reinforced slabs

The calculation of concrete reinforced slabs is different than is the case for concrete blocks or concrete slabs with U profiles. The biggest difference is that not only the loads are resulting in stress values but also temperature stress and shrinkage stress. Additionally, depending on the position where stress occurs and how the traffic load is deviated to the surroundings, slabs have a significant influence on the design of concrete pavements. This calculation is further described in *3.2 Variants: Design of Continuously* reinforced concrete.

Continuously reinforced concrete pavements

The design of this construction is determining the stress in the same way as with concrete reinforced slabs. The occurring stress is dependent on the place of the slab, the bearing capacity of the subsoil, the traffic load and temperature differences, which are causing stress in the concrete material. The lifetime of continuously reinforced concrete pavements is determined by the crack width at the surface, this should be lower than 4 mm. Depending on the circumstances, the amount of reinforcement is determined. (Egyed, 2022)

2.3 Rigid pavements – execution

Concrete block pavement

To realize the desired lifetime, it is important that the blocks are put closely against each other to create the interlocking effect. Additionally, the drainage in the construction also needs to be ensured by using a relatively low permeability on the surface to a high permeability in the foundation towards the subsoil or horizontal drainage system.

A road construction consists of a sand base and a pavement construction. Prior to applying the sand base, the subsoil will be leveled with a grader and compacted with a compactor (sandy soils) or a sheepsfoot compactor (clayey soils). In case of clayey soils, a horizontal drainage must always be foreseen as well as a geo membrane to avoid suffusion.

To respect the necessary minimum bearing capacity and minimum frost/thaw thickness: applying a sand bed is required in the Netherlands. This sand is required to fulfill the correct sand sieve curve boundaries as mentioned in the RAW standard, additionally the sand also needs to be well compacted. The compaction must be carried out under optimum moisture conditions (OMC) of the sand (7-9%) and is dynamically and statically compacted by a compactor to achieve the rule of 95% of the maximum proctor density (MPD). On top, a foundation is applied. In case an unbound base is used, the required thickness must be respected and compaction must be done by a compactor (dynamically and statically) during OMC conditions, in order for the MPD (at least 95%) to be respected. In case of a bound base, the mixture needs to be properly managed so the properties are reliable over the total area and depth. In case a bound layer is applied (f.e. a sand cement stabilization) every 1m20 a hole must be drilled with a diameter of 15 cm and filled with gravel. These holes are necessary to drain the water through the sand bed and/or subsoil/horizontal drainage pipes. If percolating water from the joints of the blocks cannot be drained the lifetime of the block pavement construction is lowered.

On top of the foundation, a bearing layer is applied of 3-4 cm, this layer is applied to enable the blocks to be put equally and drain out the water to the foundation. Usual sharp angular formed sand 1/4 mm is used or in heavy duty pavements split 2/4 mm is used. Split has a higher shear strength than sand. This layer is compacted in combination with the applied blocks.

The blocks are applied by hand in certain locations (at the sides, the beginning, and the end of constructions) and by machines in the larger areas. After the blocks are applied, a layer of fine sand will be put on top of the blocks and will slightly go into the joints, during the dynamic compaction of the blocks with the bearing layer. In general, this is how a block pavement is applied.

Figure 12 Check blocks with a machine (Van den Berg Bestratingen, 2022)

Figure 13 Dynamic compaction (bft-international, 2022)

Concrete U profile / concrete elements

The subsoil/ foundation and bearing layer is constructed the same as for concrete block pavements, in case the concrete elements are applied for a longer period (+/- 1 year). The elements vary in surface area (0,8*0,8m - 2,0*2,0m), shape (square or rectangular) and thickness 10-15 cm. Depending on the expected loads, subsoil and time, the correct elements will be placed. Due to the weight of the elements, the elements will be placed by machines. If the elements are placed temporarily, the subsoil will be levelled by a grader, and dynamic and static compaction will take place before applying the elements.

Figure 14 Placement of concrete elements (De Keij Betonplaten, 2022)

Concrete reinforced slabs

The preparation is done the same as for concrete block pavements until the point of the bearing layer. In case of an unbound base, an impermeable membrane is used (to avoid moisture differences at the bottom of the concrete slab and above the reinforcement near the joints) and will be applied on the right height. In general, dowel bars are placed in the transverse joints every 30-40 cm, with a length of two times the height of the slab. Dowel bars are foreseen of a coating to prevent that they are chemically bound with the concrete. Tire bars are foreseen in the longitudinal joints of the concrete pavements. The function of the reinforcement is to achieve a load transfer at the edges of the slabs. Dowel bars are used to make small movements possible due to temperature differences and the tire bars are used to prevent movements in transverse direction, to avoid that large longitude joints will occur. By placing the reinforcement however, a concrete pouring plan must be respected in order for the slipform paver and the concrete mixing trucks to arrive on site. With a slipform paver, the concrete is placed at the correct thickness and compaction level. Sometimes the concrete is pumped from the side of the road to the lane that will be poured and later on, the new constructed concrete lane will be the transportation lane for the construction of the next lane. After the concrete is applied, the joints are sawed and curing needs to take place to avoid large differences in shrinkage behavior on the top and bottom of the slab. After 14 days the joints are filled with an elastofill, so the percolating of water or other fluids is minimized.

Continuously concrete pavements

With continuously reinforced concrete pavements the same construction method needs to be followed as for the reinforced concrete slabs. However, the reinforcement is placed differently, which will have an effect on the sides of this pavement in case more than two lanes are applied or if anchor and end constructions are foreseen. There is more reinforcement foreseen due to high shear and tensile stresses. The concrete is also poured and compacted with a slipform paver. After applying the concrete thoroughly, curing is very important to avoid shrinkage cracks. (Egyed, 2022)

Figure 15 Slipform paver (Betoninfra, 2022)

2.4 Rigid pavements and climate

Rigid pavements are influenced by (changing) temperatures. These influences, their effects, and how pavement structures are designed to deal with these effects will be highlighted in this section.

Changing temperatures cause the concrete volume to either shrink or expand. Shrinkage can occur in different ways, plastic shrinkage, drying shrinkage, autogenous shrinkage, and shrinkage due to changing temperatures.
Plastic shrinkage occurs when water starts evaporating on the surface of the pavement layer right after construction. Plastic shrinkage is only a factor during the hardening of the concrete pavement during the early age after construction.
Drying shrinkage occurs after the concrete layer has expanded in the very early stages because of absorption of moisture occurring in the pores, which during the hydration starts evaporating and thus results in the concrete layer shrinking. This effect primarily occurs during the early age of the concrete lifetime, right after the construction of the pavement. It also occurs during the rest of the pavement's lifetime, but this effect is insignificant relevant to the early age shrinkage and the other occurring shrinkage's during the pavement's lifetime.
Autogenous shrinkage is a special type of hardening shrinkage. When the concrete has a low water-cement factor, the water gets used with the increasing hydration during the early stages after construction. This results in a decrease in volume of the cement stone. This effect only occurs during the early age of the concrete lifetime, right after the construction of the pavement.
Thermal shrinkage(/expansion) occurs because of daily temperature fluctuations. The concrete layer slightly shrinks when it cools down during the night and it slightly expands when it warms up during the day. (Beton Lexicon, 2022)

The final aspect of the interaction between rigid pavements and the climate is relaxation. Relaxation is the decrease of stress in the concrete or steel (reinforcement) due to a constant mechanical load. In this case, the mechanical load is the normal force created by the shrinkage and expansion of the concrete pavement. Relaxation is an important and beneficial phenomenon in concrete pavements, because it gradually lowers the occurring stresses due to the volume changes in concrete pavements. This in turn decreases the cracking.

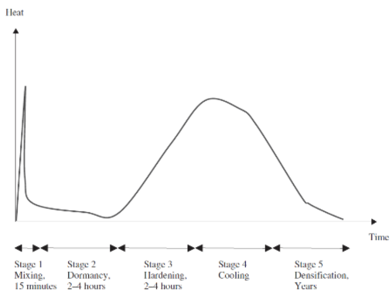


Figure 16 Illustration of the Different Stages of Hydration (Papagiannakis & Masad, 2008)

All these volume changes arise primarily due to hydration (the evaporation of water) a process in which cement and water react with each other and because of which the concrete eventually shrinks during the cooling in stage 4 and drying in stage 5 as shown in Figure 16.

INTERMEZZO TU DELFT

In 2007 a model was made by the TU Delft, based on field test regarding the tensile strength of continuously reinforced concrete pavements and the tensile stresses occurring in these pavements. The theory was that the month in which construction of the pavement was done was of major influence on the tensile strength and occurring tensile stresses. These[2] together heavily influence the cracking pattern, since cracks occur every time the tensile stresses exceed the tensile strength of the pavement. It was expected that the tensile stresses in the pavement would increase if the temperature would be lower at night, which would cause cracks to occur.

To check this theory, field tests were done for four different pavements, which were each built in different months, February, May, August, and November. These field tests resulted in graphs representing the tensile strength/stresses of pavements over time after the construction was built. Stark differences were reported between (for example) the months May, August, and November. In

[2] tensile strength of the pavement and tensile stresses occurring in the pavement

May the tensile stresses didn't exceed the tensile strength, mostly due to the temperature difference between day and night being low, and the temperatures not being as cold at night as in the other months. This resulted in Figure 17.

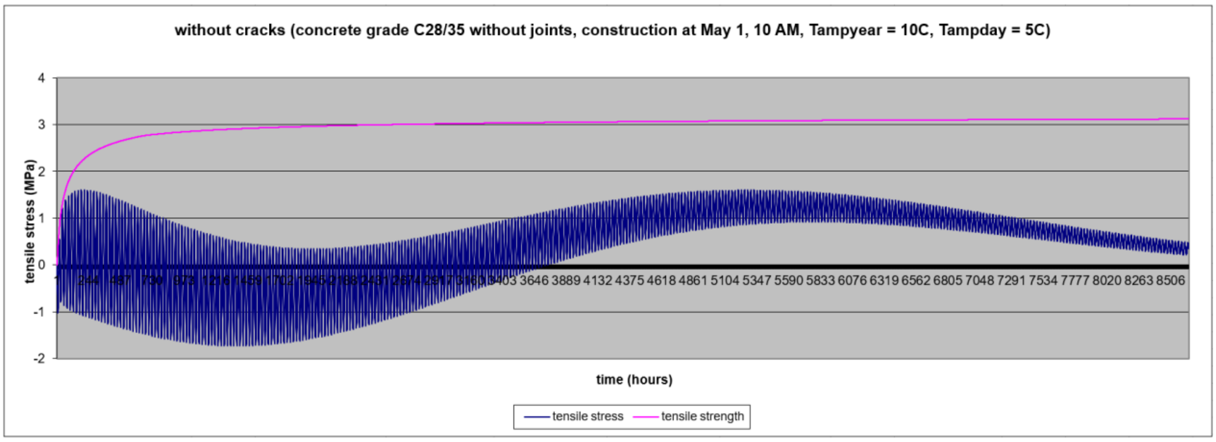


Figure 17 Tensile stresses and strength in May (TU Delft, 2007)

However, in August, the average temperature during the day of construction was significantly higher than in May (25 °C instead of 15 °C in May). This resulted in the tensile strength increasing more rapidly. Additionally, overnight, the temperature regularly dropped further than it did after construction in May, which caused higher tensile stresses to occur in the pavement. Eventually the higher tensile stresses resulted in primary cracks appearing, approximately 730 hours after construction. The tensile stresses then dropped, before slowly increasing. However, this time, crucially, the tensile stresses didn't exceed the tensile strength of the pavement construction, yielding no further cracks.

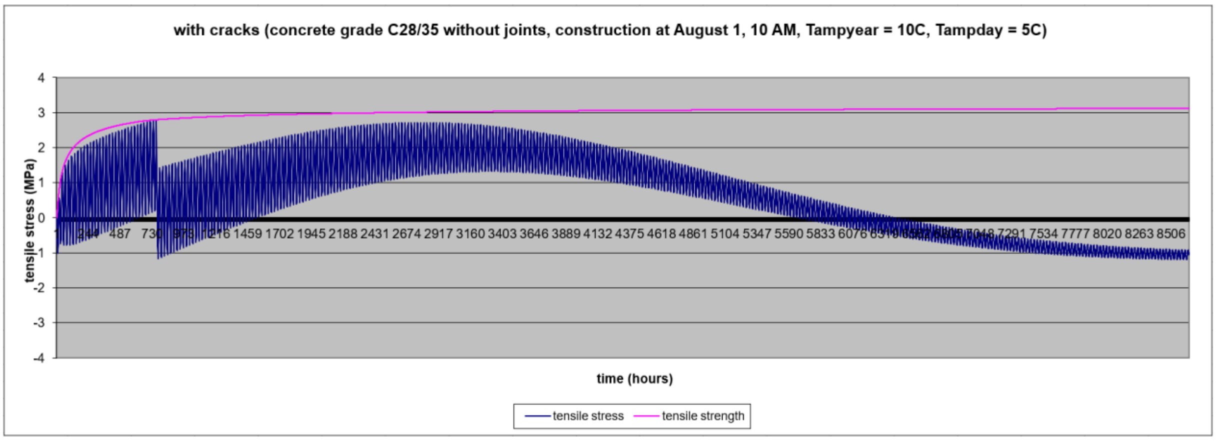


Figure 18 Tensile stresses and strength in August (TU Delft, 2007)

Apart from the tensile stresses and tensile strength of the pavement, the crack width was also monitored during this study. In August, the importance of the temperatures at construction day was shown. The crack width was around 8 millimeters, which is double the allowed crack width for pavements, since otherwise the construction becomes vulnerable to (for example) chemicals penetrating into the pavement. The higher crack width was partially caused by the rapid development of the tensile strength due to the high temperatures during the day, which accelerated the hydration process, and thus the strength gain of the pavement. This in turn delayed the moment of the primary cracks appearing, which led to the pavement's elasticity modulus increasing too far, resulting in a very stiff pavement that, when it eventually cracked, produced a crack that didn't satisfy the requirements[3].

[3] The maximum crack width normally is 4mm.

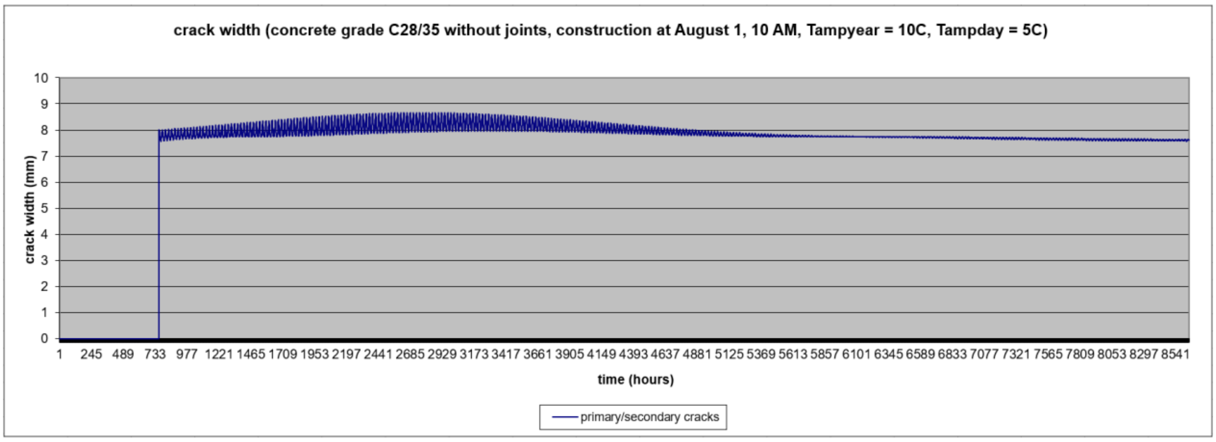


Figure 19 Crack width August (TU Delft, 2007)

In November, the daily average temperatures during construction were again lower (than in, at 15 °C. However, due to the temperatures dropping further overnight than in August, the tensile stresses increased quicker than in August, resulting in primary cracks occurring in approximately 20 hours after construction. After cracking, the tensile stresses dropped somewhat, but quickly started increasing again. This eventually resulted in the tensile stresses exceeding the tensile strength a second time at approximately 487 hours after construction, which lead to the secondary cracks appearing in the pavement. Which can be seen in Figure 20 and Figure 21.

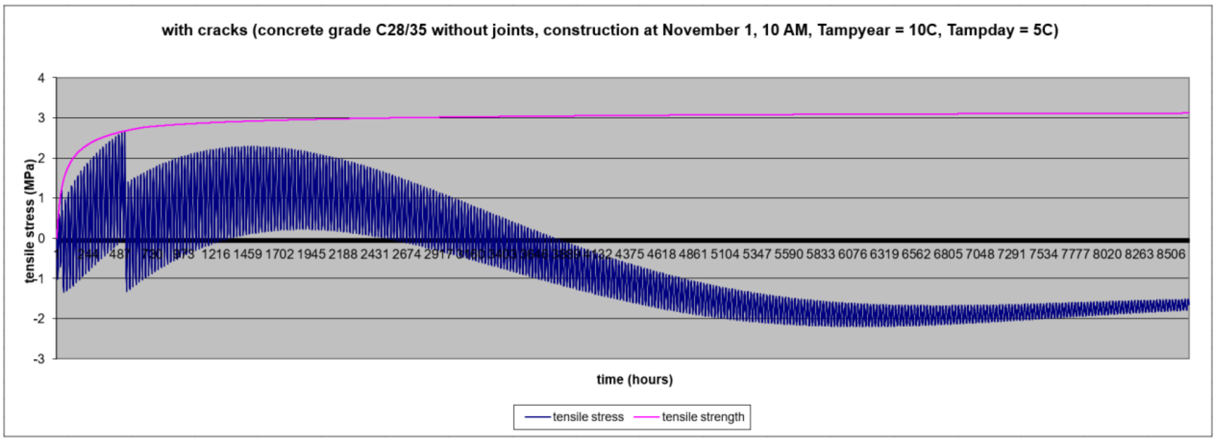


Figure 20 Tensile stresses and strength in November (TU Delft, 2007)

When comparing the crack widths in August (Figure 19) and November (Figure 21), there is a very noticeable difference in crack widths. In November the crack width does not exceed 2,75 millimeters, which is clearly lower than in August. An explanation for this is the moment at which the primary cracks occurred in the pavement, which was within a day of construction, therefore, the tensile stresses got significantly lowered. Additionally, due to the higher temperatures during the day in August, the elasticity modulus is much higher at the time of cracking (resulting in a stiffer pavement) than in November and a stiffer pavement cracking will result in a higher crack width.

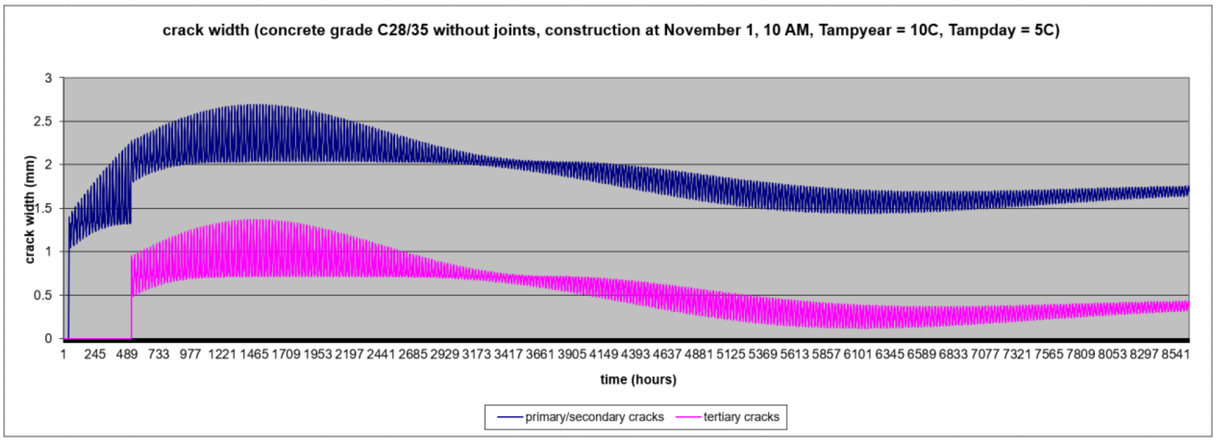


Figure 21 Crack width November (TU Delft, 2007)

Earlier in this intermezzo, the tensile strength of pavements and its development is discussed. It was concluded that high temperatures during the day (the month after construction) result in a faster increase of the tensile strength, when compared to colder temperatures. When the tensile strength develops slower due to colder temperatures, the tensile stress will more often exceed this tensile strength and cause more cracks to form. However, due to the low tensile forces that are needed to cause cracking, the crack widths will be lower than in warmer climates. In warmer climates, where the hydration process is sped up causing the tensile strength to increase rapidly, the tensile stress required for cracking to occur in the pavement is higher, resulting in a stiffer pavement with higher crack widths. It can thus be concluded that the cracking pattern will differ significantly depending on the climate that the construction is made in. The cracking pattern thus is influenced by the development of the tensile strength and tensile stresses in the pavement, in which temperatures are an important factor.

The amplitude of the tensile stresses also is seen to be decreasing over time in all pavements. This is due to relaxation occurring over time, decreasing the tensile stresses in the pavement.

Volume changes can have several effects, first of all, the increase of tensile forces regulate the dimensions of the cracks (width and depth). The higher the shrinkage is in concrete, the higher the tensile forces will be, and thus the bigger the cracks will become. To counteract this effect in CRCP, reinforcement is used, to help the concrete resist the tensile forces. Secondly, curvatures can occur in concrete, when the degree of shrinkage differentiates between the bottom and top of the pavement. When one side shrinks significantly more than the other side, a curvature arises. A rotation (curvature) over the width of the concrete pavement can occur due to a combination of several factors;

1) a settlement of the soil underneath the pavement,
2) a traffic load on top of the pavement,
3) a negative temperature gradient from the surface to the bottom of the pavement,
4) a shrinkage gradient from the surface to the bottom of the pavement.

In this thesis, only a negative curvature will be considered, since the traffic load on top of the pavement should counteract a positive curvature. A negative temperature gradient can occur when there is a sudden decrease in temperature. In a concrete pavement, the surface of the concrete pavement then shrinks due to the pavement itself also cooling down (for example overnight), while the bottom layer doesn't shrink due to it taking longer to be affected by the drop in temperature. A shrinkage gradient arises when the occurring total shrinkage (autogenous shrinkage and shrinkage due to moisture loss) on the surface of the concrete pavement is bigger than the total shrinkage on the bottom of the concrete pavement.

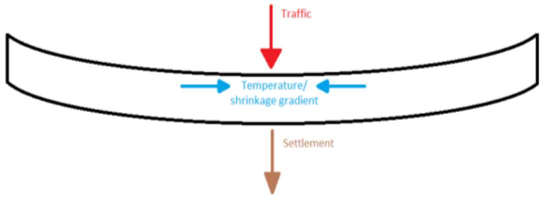


Figure 22 Schematic drawing of the forces across the width of a concrete pavement

These normal forces due to volume changes are counteracted in rigid pavements. These ways can be divided into several categories, these are: using the design, altering material properties, and measures during construction/execution.

Using the design, continuously reinforced concrete pavements use reinforcement to counteract the tensile forces (due to shrinkage) Additionally, CRCP also uses end constructions that anchor the pavement down, preventing it from moving. Examples of these constructions are given in Figure 23 and Figure 24. The combination of these measures ensures the deformations being obstructed in the pavement.

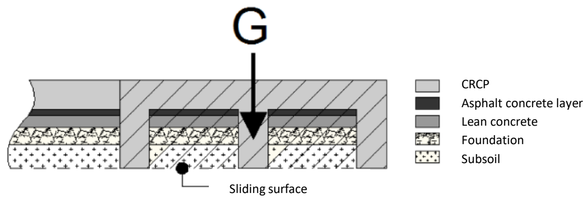


Figure 23 End reinforcement in CRCP (Sarens, 2012)

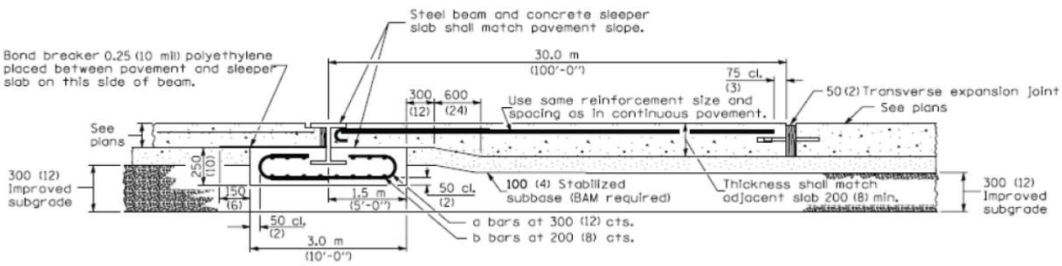


Figure 24 Wide-flange beam joint (Sarens, 2012)

Secondly, in jointed pavements, the joints allow for the pavement to slightly expand and shrink without major repercussions.
Altering material properties, one way of changing material properties is to tailor the concrete mixture towards a specific project. For example, the ratio of the materials in the concrete and size of the grains can be changed to increase the strength of the concrete, and thus its ability to deal with tensile and compressive forces exerted on it due to temperature. Additionally, additives can also be used to alter the bonding and hardening properties of the cement. Increasing the quantity of tensile forces that the cement can take. This can result in a pavement that shrinks/expands less than other pavements.
Measures during construction/execution could for example be using a polyethylene sheeting to cover the pavement, in order to reduce the amount of water evaporating from the surface, and thus the decreasing the amount of plastic shrinkage occurring after construction. Another measure could be curing, the covering of the construction with water (and oil) to retain moisture levels in the

pavement, and thus delaying drying shrinkage until the concrete is strong enough to resist shrinkage cracking.

2.5 Climate in the Netherlands

This thesis will primarily be about the climate in the Netherlands. In the Netherlands the KNMI (Koninklijk Nederlands Meteorologisch Instituut) is the national weather forecasting service, which also keeps daily climate records from different weather stations in the Netherlands, of which some even date back to 1901. These records contain data regarding wind (direction/speed), temperature (minimum/maximum), radiation, air pressure, visibility, and humidity. The headquarters of the KNMI is located in the municipality “de Bilt”, which has been used as the norm in the Netherlands, and thus also for this thesis. “De Bilt” has been used as the norm due to its central positioning and because it wasn’t influenced as much by city conditions, where it tends to be warmer.
As has been discussed in the previous section "Rigid pavements", the weather mostly influences CRCP through the occurrence of a temperature gradient over the height of the pavement. This gradient exists due to the temperature at surface of the pavement reacting quicker to daily rising temperatures than the temperature at the bottom of the pavement structure. This delay results in a gradient from the higher temperatures at the surface to the lower temperatures at the bottom of the pavement structure. In this section several trends will be analyzed: 1) average temperatures, 2) minimum and maximum temperatures, 3) daily temperature fluctuations. Finally, based on the data, a prediction for the expected minimum/maximum temperatures and the temperature fluctuations will be made for the next 30 and 60 years[4].

3. Methodology

3.1 Answering the sub questions

In the following section, the answering of all sub questions will be briefly discussed. First of all, the activities will be described that have or will be done to answer these questions. Secondly, the final product, that will answer the sub question, will be specified. Finally, the types of sources used to answer the sub question will be listed.

- *What is the expected temperature rise and what are the expected temperature fluctuations, due to global climate change in Western Europe?*

To answer this question, the Dutch weather will be seen as representative for Western Europe. In the Netherlands, the KNMI produces data recorded at the weather stations, of whom the weather station in “de Bilt” is seen as the standard for the country, partially due to its central location. The data from this weather station will be used to analyze the occurring trends regarding temperatures and temperature fluctuations. Once this trend is established, it will be used to predict the coming 60 years.
The end product will be three scenarios that predict the future. The first scenario will be that the temperatures and temperature fluctuations stay the same as they are currently. The second scenario will be that the trend continues. The third situation will be that the trend accelerates at twice the speed it did before.
Data retrieved from the KNMI will be used to answer this question.

- *What is a continuously reinforced concrete pavement and how does it differentiate from other pavement structures?*

[4] 60 years is roughly equal to 2 concrete pavement lifetimes

To answer this question, a literature study has been done together with an interview with a field expert. Several articles, discussing the design, execution and properties of the different types of rigid pavement structures have been read and used to write the theoretical framework. In addition, there also has been an interview, which helped to answer questions that weren't answered by reading the literature. This interview was done with a senior colleague in the company.
The end product that answers this sub question is the theoretical framework, in which all the types of rigid pavements have been generally described/compared.
Literary sources have been used together with an interview with a field expert.

- *Which design methods/principles are being used for continuously reinforced concrete pavement?*

This sub question will be answered when the variants are discussed. In this thesis, several design methods for CRCP will be analyzed, described, and put through a Multi Criteria Analysis. The three methods chosen for this thesis are "AASHTO 1993", "VENCON 2.0", and "FLOOR 3.0". These methods will be analyzed by going through background reports of the methods, provided either online or with the product by the designers of the method.
The end product will be a detailed description of the three design methods, which will all be a variant in the Multi Criteria Analysis, used to decide which design method will be used as a basis for the model developed later in the thesis.
Background reports detailing all the calculations/formulas used in the design methods to calculate pavement thickness have been acquired and will be analyzed.

- *What is known of continuously reinforced concrete pavement in other (warmer) climates?*

To answer this question, the results of a study/model of the TU Delft has been discussed in the theoretical framework. In this study/model 4 separate reinforced pavements were monitored on the tensile stresses/tensile strength after construction. Each of these reinforced pavements was built in a different month, resulting in different circumstances regarding day-night temperatures. The findings of this study can be used to predict how reinforced pavements will perform in colder/warmer climates. This prediction will be made in *6. Conclusions*, and will be supported by an interview with an expert (Christophe Egyed).
The end product will be a prediction for how reinforced pavements respond to colder and warmer climates based two sources: a study/model of the TU Delft and an expert opinion.

- *What variable are temperature and temperature fluctuations in the design of continuously reinforced concrete pavement?*

This question will be answered through the same way as the third sub question, regarding the design methods of CRCP. In those design methods there are several ways in which temperature and temperature fluctuations are taken into account, these will be discussed in the analysis of the variants (design methods).
The end product will be a description of the influences of temperatures (fluctuations) as a part of the variant study.
Background reports detailing all the calculations/formulas used in the design methods to calculate pavement thickness have been acquired and will be analyzed.

- *What variables are needed to calculate a continuously reinforced concrete pavement, while taking into account climate change?*

This question will be answered through the same way as the third and fifth sub question, regarding the design methods of CRCP. In those design methods there are quite a few variables which influence

the design of CRCP. These variables will first be clearly visible in flow charts, presented in the beginning of the variant study, and will be described in further detail later in the section, in addition with the representative formulas.
The end product will be a variant study, which will include all the different variables used to calculate CRCP, while taking into account climate change.
Background reports detailing all the calculations/formulas used in the design methods to calculate pavement thickness have been acquired and will be analyzed.

- *How can a model for calculating continuously reinforced concrete pavements be created which takes into account global climate change?*

This final question will be answered in the result section of this thesis, through *4.4 Model based on* . This section will entail the findings from the model made based upon the best scoring variant in the MCA. This model allows the user to also see what is happening in between the input and output, which normally is a black box, but now all the formulas are accessible. This model will be made in excel, based upon the literature provided to analyze all the different design methods in the variant study.
The end product will be an excel model based upon the best scoring variant in the MCA. This model in turn will help analyze the variable of temperatures and temperature fluctuations in the design of CRCP.
Background reports detailing all the calculations/formulas used in the design methods to calculate pavement thickness have been acquired and will be analyzed.

3.2 Variants: Design of Continuously reinforced concrete

3.2.1 AASHTO 1993

AASHTO (American Association of State Highway and Transportation Officials) was founded in 1914, then still under the name AASHO, and it strongly advocated for federal-aid funding for highways to promote the development of highway systems nationwide and to allow major routes to span state lines. In the early years, the highway system was highly impractical, due to the transportation system being regulated by the individual states themselves. Even though, through the years, the states made plenty of progress, building high quality urban and rural roads, their interests weren't to connect with other states, but mainly to improve the connection in their own state. As Americans went off to fight in World War II, they observed the efficiency of the German autobahn, eventually sparking the Eisenhower project through the federal aid highway project., to create the Interstate highway system, providing federal-aid funding was crucial in this plan, as it provided 90% of the funding of the project. The way these roads were build, was highly standardized, to ensure a constant quality of the transportation network, which was not dependent on the state in which it was built. It eventually took roughly 40 years to complete this interstate highway system, and Eisenhower's vision was realized, travelling by car from the east coast to the west coast now took 42 hours, instead of the 62 days it took in 1919. This highlights the influence that the interstate network has had for trade/transportation of goods.

AASHTO itself publishes, amongst other things, specifications, test protocols, and guidelines that are (highly standardized) used in highway design and construction throughout the United States. One of the guidelines they have published, is the AASHTO Guide for Design of Pavement Structures 1993. This document originations from the AASHO Road Test, which had as its objective to provide information that could be used to develop pavement design criteria and pavement design procedures. This has thus been used as a "Guide for the Design of Rigid and Flexible Pavements", and has been updated through the years, leading to the document considered in this thesis, the 1993 version.

The considered document contains details regarding the design, maintenance and rehabilitation of flexible and rigid pavements, and it consists of 4 parts:

- Part I: Pavement design and management principles
- Part II: Pavement design procedures for new construction or reconstruction
- Part III: Pavement design procedures for rehabilitation of existing pavements
- Part IV: Mechanistic-empirical design procedures

During this thesis, the first and second part are used, seen as they describe the implications on the design of rigid pavements. In these parts several factors are considered when designing a rigid pavement, being:

- Pavement performance
- Traffic
- Roadbed soil
- Materials of construction
- Environment
- Drainage
- Reliability
- Life-cycle costs

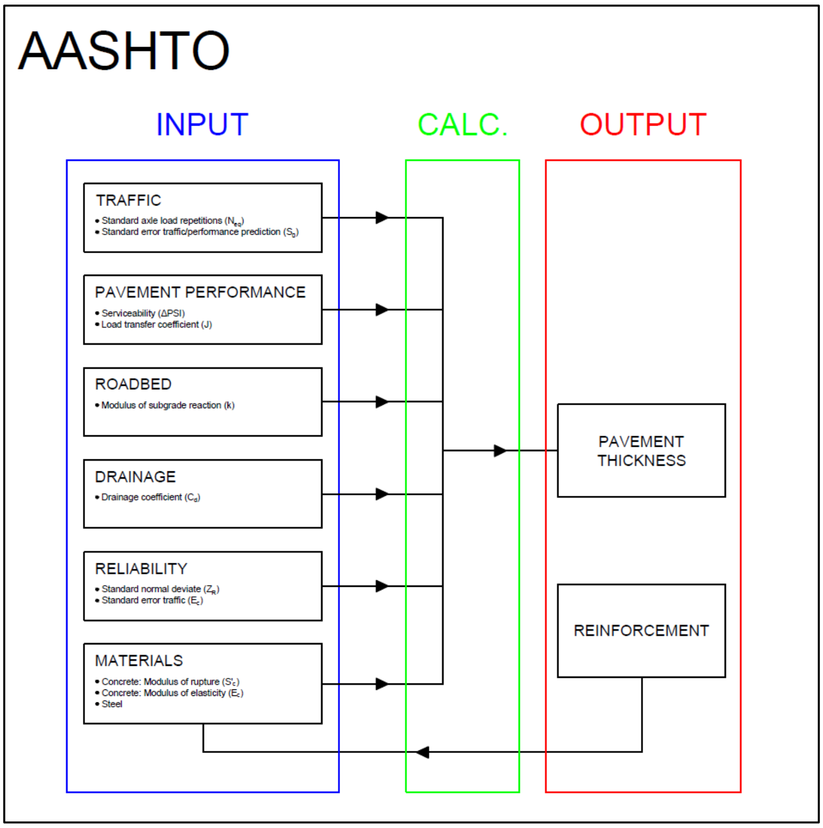


Figure 25 Flow chart of the AASHTO 1993 model

An overview of most of these factors is given in a flow chart (Figure 25) and are represented by 10 different variables. These variables together form the design of rigid pavements. In the following section, each variable will be discussed, and assumptions regarding their attributed values in this thesis will be argued. A summary of these assumptions will be listed in section 2.3 Assumptions. (AASHTO, 2022)

Predicted number of 18-kip equivalent single axle load applications – $\hat{w}_{18}$
The design procedures for both highways and low-volume roads are all based on cumulative expected 18-kip equivalent single axle loads (ESAL) during the analysis period. This variable is representing the number of 18.000 pound (roughly 80 kN) equivalent single-axle loads associated with vehicular traffic on a certain road in a certain period of time. This value is then multiplied with 1) a directional distribution factor, expressed as a ratio, that accounts for the distribution of ESAL units by direction, e.g., east-west, north-south, and 2) a lane distribution factor, expressed as a ratio, that accounts for the distribution of traffic when two or more lanes are available in one direction. The relation between the predicter number of 18-kip ESAL applications and the thickness of the pavement slab is quite clear, the higher the amount of traffic that is going to be on the road, the higher the pavement slab will be, to be able to deal with this traffic.

In this thesis a directional distribution factor of 0.50 will be used, since this factor is generally used for most roadways, assuming the same weight is distributed in both directions. Secondly, a lane distribution factor of 0.90 is used, due to the assumption of having 2 lanes going into each direction. The cumulative two-directional 18-kip ESAL units predicted for a specific section of highway during the analysis period will be varied, based upon the type of road being modelled, a highway will have more and heavier traffic, where a provincial road will have less.

$$w_{18} = D_D \times D_L \times \hat{w}_{18}$$

w_{18} = traffic in the design lane
D_D = a directional distribution factor expressed as a ratio, that accounts for the distribution of ESAL units by direction, e g, east-west, north-south, etc.
D_L = a lane distribution factor, expressed as a ratio, that accounts for the distribution of traffic when two or more lanes are available in one direction
$\hat{W}_{18}$ = the cumulative two-dimensional 18-kip ESAL units predicted for a specific section of highway during the analysis period (from the planning group)

Standard normal deviate – Z_R
According to the general definitions that have been selected from the highway research literature there are several aspects when it comes to reliability. These aspects are whether or not the 1) serviceability according to the users is maintained, 2) the load applications the pavement can withstand are exceeded or not, and 3) the probability that the pavement system will perform its intended function over its design life.
To account for uncertainty in the reliability, the standard normal deviate is used as a way of building in a safety factor. In this thesis a reliability of 95% is deemed to be satisfactory, and thus -1.645 is used as a value for the standard normal deviate.

Combined standard error of traffic/performance prediction - S_o
As with most predictions/results, there also is standard deviation when it comes to predicting traffic and performance of the pavement construction. To take this into account, there is a variable S_0 which represents the overall standard deviation, suitable to the specific pavement. In AASHTO 1993, this variable takes on a value of 0.30 – 0.40 for rigid pavements and 0.40 and 0.50 for flexible pavements. In this thesis this value will assumed to be 0.35.

Serviceability index - ΔPSI
The serviceability of a pavement is defined as its ability to serve the type of traffic which uses the facility. The primary measure of the serviceability is the Present Serviceability Index (PSI) which ranges from 0 (impossible road) to 6 (perfect road). Selection of the lowest allowable PSI or *terminal serviceability index* (p_t) is based on the lowest index that will be tolerated before rehabilitation, resurfacing or reconstruction becomes necessary. In AASHTO, there is a table provided (Table 1)

in which 3 different terminal serviceability levels are listed paired with the percent of people stating is unacceptable. In this thesis a value of 2.75 will be used as the terminal serviceability level.

Terminal Serviceability Level	Percent of People Stating Unacceptable
3.0	12
2.5	55
2.0	85

Table 1 Terminal Serviceability Level (AASHTO, 2022)

Since the time at which a given pavement structure reaches its terminal serviceability depends on traffic volume and the original or initial serviceability (p_0), values observed at the AASHO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavements.

$$\Delta PSI = p_O - p_t$$

ΔPSI = total change in serviceability
p_O = the initial design serviceability index
p_t = the design terminal serviceability index

Modulus of rupture - S'_c
The modulus of rupture (flexural strength) of Portland cement concrete is required only for the design of a rigid pavement. The modulus of rupture required by the design procedure is the mean value determined after 28 days. In this thesis a PCC modulus of rupture of 578 psi will be used, since this value is also used in the example calculation of the AASHTO Guide.

Load transfer coefficient - J
The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer (distribute) load across discontinuities, such as joints or cracks. Load transfer devices, aggregate interlock, and the presence of tied concrete shoulders all have an effect on this value. Generally, the J-value for a given set of conditions increases as traffic loads increase since aggregate load transfer decreases with load repetitions. In the AASHTO Guide the value of J recommended for CRCP without tied concrete shoulders is between 2.9 and 3.2 depending on the capability of aggregate interlock (at future transverse cracks) to transfer load.
In this thesis, a value of 3.05 will be used for the load transfer coefficient.

Drainage coefficient – C_d
Proper drainage is important to ensure a high-quality long-lived pavement; moisture accumulation in any pavement structural layer can cause problems. Moisture in the subgrade and aggregate base layer can weaken these materials by increasing pore pressure and reducing the materials' resistance to shear. Additionally, some soils expand when moist, causing differential heaving (the roadway heaves up as the underlying soil expands). (Pavement interactive, 2022)
In the AASHTO Guide the drainage coefficient is determined through selecting the "Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation" and the "Quality of Drainage". In this thesis, the "Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation" is assumed to be 5-25% and the quality of the drainage is assumed to be "Good". This together yields a Drainage coefficient of 1.10 – 1.00, and thus 1.05 will be used.

Modulus of elasticity – E_c
The modulus of elasticity of concrete is the measurement of the stiffness of the concrete which is a good indicator of strength. At a higher value of modulus of elasticity, the concrete can withstand higher stress and become brittle. In this thesis, the equation provided by the AASHTO Guide will be used, together with a PCC compressive strength of 4000 psi, since most pavement PCC has a compressive strength between 3000 and 5000 psi. (Pavement Interactive, 2022)

$$E_C = 57{,}000(f'_c)^{0.5}$$

Where:
E_c = PCC elastic modulus (in psi)

F’c = PCC compressive strength (in psi) as determined using AASHTO T 22, T 140, or ASTM C 39

Modulus of subgrade reaction - k

The modulus of subgrade reaction is defined as the pressure per unit deformation of the subgrade at specific pressure or deformation. The field plate load test can be used to determine the modulus of subgrade reaction. In this test, compressive stress is applied to the soil layer through rigid plates, and the deflections are measured for different values of stress. (Neenu, 2022)

When estimating the modulus of subgrade reaction, several steps are done, which will be discussed:

First of all, several combinations will be defined beforehand: Subbase type, subbase thickness, Loss of support, and depth to the rigid foundation.

Through laboratory tests, the seasonal roadbed soil resilient modulus values are entered for (for example) each month. The purpose of identifying seasonal moduli is to quantify the relative damage a pavement is subjected to during each season of the year and treat it as a part of the overall design.

The next step is to assign subbase elastic modulus for each season/interval. This is also done through laboratory tests.

Then the composite modulus of subgrade reaction is either estimated by using Figure 26, or in case of the slab being placed directly on the subgrade, the following formula:

$$k = \frac{M_R(roadbed\ modulus)}{19{,}4}$$

This will result in a k-value for each of the months.

In step 5, the k-value is further developed based on the effect of a rigid foundation near the surface. This step should be disregarded if the depth to a rigid foundation is greater than 10 feet.

The sixth step is to estimate the thickness of the slab that will be required, and to use Figure 27 to determine the relative damage. Of this relative damage, a yearly average will be taken, which in turn is used in combination with Figure 27 to calculate the average effective modulus of subgrade reaction.

The final step in the process is to adjust the effective modulus that follows from the previous slab, and adjust it for the potential loss of support arising from subbase erosion, for which Figure 28 is used.

In this thesis, the example calculation provided in the AASHTO Guide will be used, which leaves an effective modulus of 320 pci.

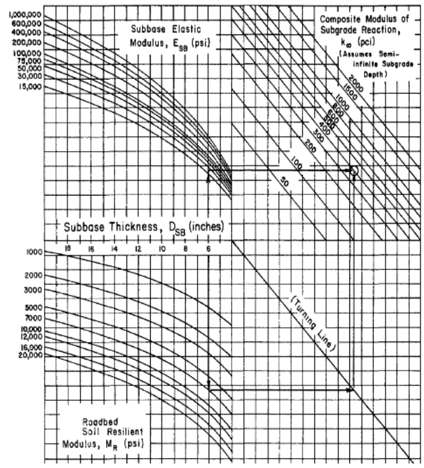


Figure 26 Chart for estimating composite modulus of subgrade reaction (AASHTO, 2022)

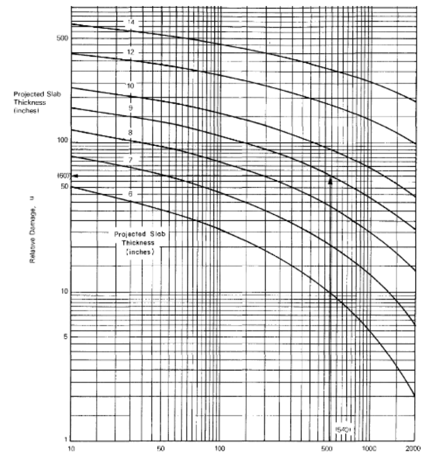


Figure 27 Chart for estimating relative damage (AASHTO, 2022)

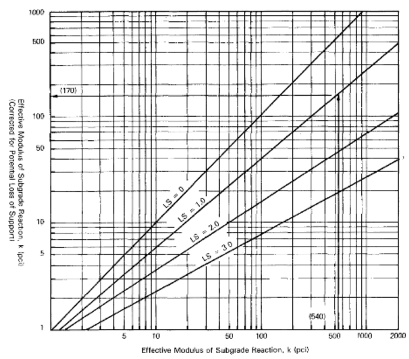


Figure 28 Correction of k for potential loss of subbase support (AASHTO, 2022)

Thickness of the pavement slab

Lastly, the thickness of the pavement slab is determined, this is an iterative process, the overall formula for rigid pavements, as provided by the AASHTO Guide will be entirely filled in (all variables except the thickness), after which an iterative process follows to determine what the advised thickness is of the pavement under those conditions.

In this thesis, primarily the relation between the traffic load and the thickness of the pavement slab will be analyzed, in the result section of the report. (AASHTO, 2022)

$$0 = Z_R * S_0 + 7{,}35 * \log_{10}(D+1) - 0{,}06 + \frac{\log_{10}\left(\frac{\Delta PSI}{4{,}5-1{,}5}\right)}{1 + \frac{1{,}624 * 10^7}{(D+1)^{8{,}46}}} + (4{,}22 - 0{,}32 * p_t) * \log_{10}\left[\frac{S'_c * C_d * (D^{0{,}75} - 1{,}132)}{215{,}63 * J * \left[D^{0{,}75} - \frac{18{,}42}{\left(\frac{E_c}{k}\right)^{0{,}25}}\right]}\right] - \log_{10}(w_{18})$$

Reinforcement of the pavement slab

The design of reinforcement in AASHTO is quite straightforward. During the first step, the longitudinal reinforcement is calculated for 3 different parameters: crack spacing, crack width, and steel stress. For each of these parameters, several variables are used to solve the equations, which are:

Crack spacing

- Desired crack spacing ($\bar{x}$)
- Thermal coefficient ($\frac{\alpha_s}{\alpha_c}$)
- Diameter of the bars ($\emptyset$)
- Concrete shrinkage at 28 days (z)
- Tensile stress due to a wheel load (σ_w)
- Concrete tensile strength at 28 days (f_t)

$$\bar{x} = \frac{1{,}32 * \left(1 + \frac{f_t}{1000}\right)^{6.70} * \left(1 + \frac{\alpha_s}{\alpha_c}\right)^{1.15} * (1+\emptyset)^{2.19}}{\left(1 + \frac{\sigma_w}{1000}\right)^{5.20} * (1+P)^{4.60} * (1 + 1000 * z)^{1.79}}$$

Crack width

- Desired crack width (c_w)
- Diameter of the bars ($\emptyset$)
- Tensile stress due to a wheel load (σ_w)
- Concrete tensile strength at 28 days (f_t)

$$c_w = \frac{0.00932 * \left(1 + \frac{f_t}{1000}\right)^{6.53} * (1+\emptyset)^{2.20}}{\left(1 + \frac{\sigma_w}{1000}\right)^{4.91} * (1+P)^{4.55}}$$

Steel stress

- Steel stress (σ_s)
- Design temperature drop (DT_d)
- Concrete shrinkage at 28 days (z)
- Tensile stress due to a wheel load (σ_w)
- Concrete tensile strength at 28 days (f_t)

$$\sigma_s = \frac{47300 * \left(1 + \frac{DT_d}{100}\right)^{0.425} \left(1 + \frac{f_t}{1000}\right)^{4.09}}{\left(1 + \frac{\sigma_w}{1000}\right)^{\left(\frac{3}{4}\right)} * (1+P)^{2.74} * (1 + 1000 * z)^{0.494}}$$

As with most calculations in AASHTO, nomographs are used to solve bigger equations. After these formulas have been solved, they will set boundary conditions for the reinforcement rate, and selecting them is an iterative process. The next step is to determine the range in the number of reinforcing bars required, based on the minimum/maximum reinforcement rate, the total width of the pavement section, the thickness of the concrete layer and the reinforcing bar diameter.

3.2.2 VENCON 2.0

In the eighties, a calculation method has been developed as a table book, made by the VNC, which was used for the dimensioning concrete pavements. However, with the introduction of pc's, a new automized calculation program was developed in the nineties, VENCON (V EN C ONtwerpmethode). This program converts input data into a layer thickness for unreinforced concrete pavements. However, these programs weren't user friendly, and weren't compatible to windows, which led to the development of a "new" VENCON by the CROW. This new program was able to calculated both unreinforced concrete pavements and reinforced pavements, and to make it more user friendly, standard constructions/roads/climate settings have been added. (Faasen, et al., 2005)

The structural design of the concrete pavements (without reinforcement) is based on a fatigue analysis for potentially critical areas in the pavement. These areas include: at the edge of a plate (that isn't fixed), at longitudinal joints, and transverse joints located at the tracks (common place of surface contact of the wheels of traffic).

The analysis consists, firstly, of traffic related stresses, which are calculated using the Westergaard-formula. There are several Westergaard formula, of which, most commonly, used is one that considers either a circular contact surface or a half circular contact surface for the wheels on the pavement construction.
Secondly, the analysis also consists of stresses due to a temperature gradient. These stresses are calculated using an altered version of the theory of Eisenmann.

To determine the thickness of a continuously reinforced concrete pavement, an adjustment needs to be made, seen as this method is based upon the structural design of concrete pavements without reinforcement. Therefore, the reinforced pavements will be considered as a jointed pavement, with different horizontal dimensions (because of the transverse cracks instead of joints) and a higher load transfer at the transverse cracks. (Houben, 2007)

This all yields the following structure:

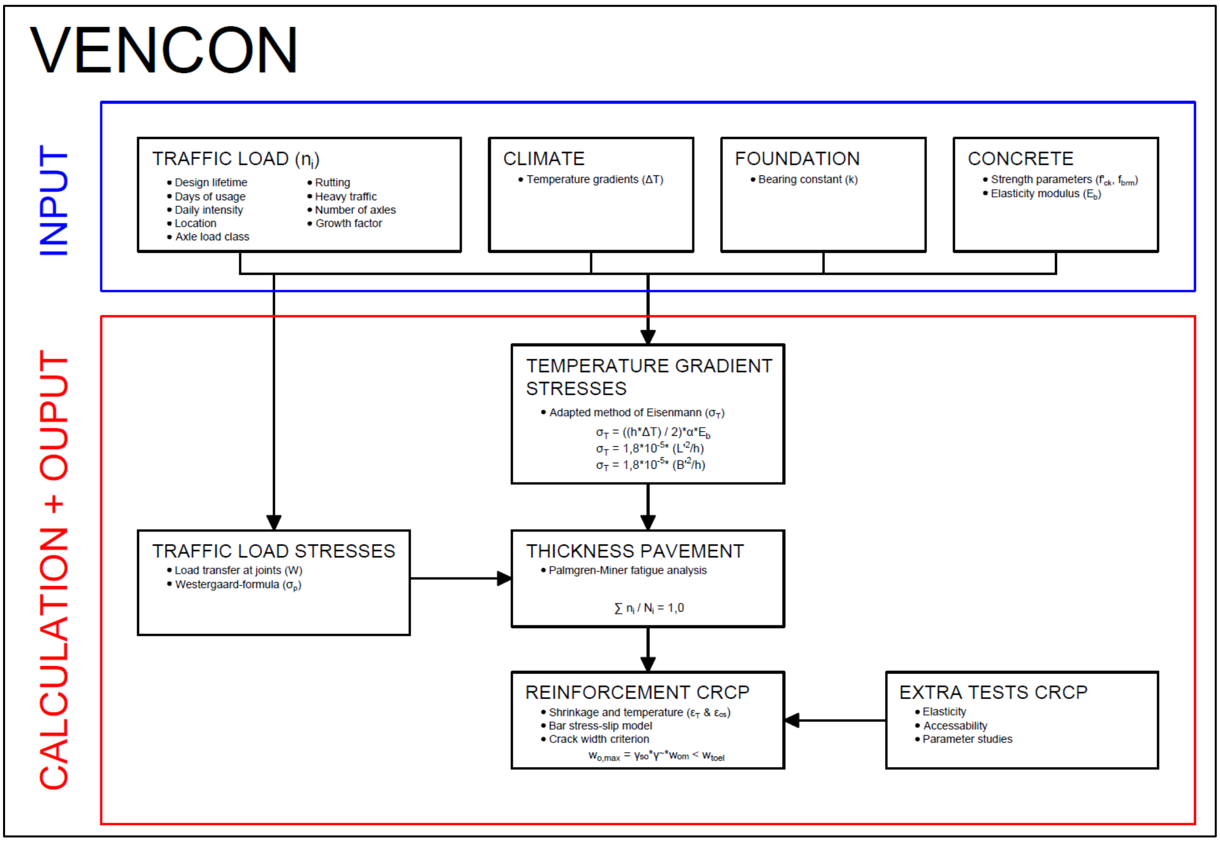


Figure 29 Flow chart of the VENCON 2 model

In the following part of the VENCON section, the individual parts and their calculations will be highlighted.

Traffic load

First of all, the *axle load repetitions* are calculated, through the following equation:

$$\sum_i t_{design} * t_{usage} * axle_i * i_{daily} * f_{position} * f_{rutting} * f_{heavy\ traffic} * n_{axles} * G$$

t_{design}	= design lifespan (days)
t_{usage}	= days of usage per year (days)
$axle_i$	= the frequency in which an axle load class occurs on the road (-)
i_{daily}	= average daily intensity (motor vehicles)
$f_{position}$	= factor that accounts for the position of the vehicle on the road (-)
$f_{rutting}$	= factor that accounts for rutting of (heavy) vehicles (-)
$f_{heavy\ traffic}$	= factor that accounts for the presence of heavy vehicles in the specific lane (-)
n_{axles}	= average number of axles on the vehicles (-)
G	= growth factor traffic, $G = \frac{(1+\frac{g}{100})^{t_{design}} - 1}{\frac{g}{100} * t_{design}}$ in which g = growth percentage per year.

This equation is built up with a number of variables, which are:
The **design lifespan**, for how long is the pavement construction going to be used. The **number of days per year the road is used**, which accounts for irregularities in usage, such as holidays and weekends. The **daily intensity**, which is the number of vehicles that drive over the road per day on average. The **position of the vehicles on the road**, do they drive over the middle of the concrete

plate, where two adjacent concrete plates meet, or on the edge of the road? Whether or not **rutting** occurs, this only occurs when vehicles are driving over the same "tracks", and thus load the same spot in the pavement. The **number of lanes**, which influences the percentage of heavy vehicles present in each lane, if there are less lanes, the percentage of heavy vehicles in the right-most lane increases. The **average number of axles**, which influences the number of load repetitions directly. The **growth factor**, representing a future growth in traffic using the road. The **frequency of an individual axle load class**, on highways, heavier axle load classes occur (percentage-wise) more often than on for example a country road. This distribution is represented in Table 2, where highways often tend to attract more heavy vehicles, and smaller roads tend to attract fewer heavy vehicles.

Axle Load Class (kN)	Average Wheel Load P (kN)	Axle Load Frequency Distribution (%) for Different Types of Roads						
		Heavily Loaded Highway	Normally Loaded Highway	Heavily Loaded Provincial Road	Normally Loaded Provincial Road	Municipal Main Road	Rural Road	Bus Lane
20-40	15	20.16	14.84	26.62	24.84	8.67	49.38	-
40-60	25	30.56	29.54	32.22	32.45	40.71	25.97	-
60-80	35	26.06	30.22	18.92	21.36	25.97	13.66	-
80-100	45	12.54	13.49	9.46	11.12	13.66	8.05	-
100-120	55	6.51	7.91	6.50	6.48	8.05	2.18	100
120-140	65	2.71	3.31	4.29	2.70	2.18	0.38	-
140-160	75	1.00	0.59	1.64	0.83	0.38	0.38	-
160-180	85	0.31	0.09	0.26	0.19	0.38	0.00	-
180-200	95	0.12	0.01	0.06	0.03	0.00	0.00	-
200-220	105	0.03	0.01	0.03	0.00	0.00	0.00	-
Average number of axles per heavy vehicle		3.5	3.5	3.5	3.5	3.5	3.1	2.5

Table 2 Default Axle Load Frequency Distribution for Different Types of Roads

After calculating the number of axle load repetitions, the equivalent number of standard axle load[5] repetitions are calculated by using the following formula:

$$N_{eq_i} = \left(\frac{P_i * 2}{100}\right)^a * axle\ load\ repetitions$$

Wherein:

N_{eq} = equivalent number of standard axle load repetitions
P_i = average wheel load of an individual axle load class
a = power factor dependent on the construction (in VENCON 2.0 a power factor of 4 is used)

Climate

The climate is defined by a vertical temperature gradient in the pavement. This gradient is represented in several classes by ΔT in °C/mm, for which a frequency distribution can be entered. A

[5] A standard axle load repetition is 100 kN in the Netherlands, and 80 kN in America

default frequency distribution has been given, based upon testing, but this can be changed according to the climate of a case-by-case basis. The gradient classes and default frequency distribution have been given in Table 3.

Temperature Gradient Class (^oC/mm)	Average Temperature Gradient ΔT (^oC/mm)	Frequency Distribution (%)
0.000-0.005	0.0025	59
0.005-0.015	0.01	22
0.015-0.025	0.02	7.5
0.025-0.035	0.03	5.5
0.035-0.045	0.04	4.5
0.045-0.055	0.05	1.0
0.055-0.065	0.06	0.5

Table 3 Default Temperature Frequency Distribution

Foundation

The foundation for the pavement can be inputted via a bearing constant k, which is calculated for each individual layer, where the k_0 value is the k value of the layer below or the k_0 of the subsoil in case of calculated the k value for the first layer (on top of the subsoil). It is important to keep in mind that the dynamic elastic modulus should increase with each higher layer, and that your eventual bearing constant value shouldn't be above 0,16 and it should fulfil the following condition:

$$\log(k) \leq 0.73688 * \log(E_b) - 2.82055$$

$$k = 2{,}7145 \cdot 10^{-4} \cdot (C_1 + C_2 \cdot e^{C3} + C_4 \cdot e^{C5})$$

Wherein:

C_1 = $30 + 3360 \cdot k_o$

C_2 = $0{,}3778 \cdot (h_b - 43{,}2)$

C_3 = $0{,}5654 \cdot \ln(k_o) + 0{,}4139 \cdot \ln(E_b)$

C_4 = -283

C_5 = $0{,}5654 \cdot \ln(k_o)$

k_o = Bearing constant at the topside of the lower layer (N/mm^3)

h_o = thickness of the relevant layer (mm)

E_b = dynamic elastic modulus of the relevant layer (N/mm^2)

k = Bearing constant at the top of the relevant layer (N/mm^3)

Concrete

Concrete is the last of the "input blocks", and the concrete class mostly determines the value of most parameters. These parameters include the characteristic compressive cube strength after 28 days (f'_{ck}), the average compressive cube strength after 28 days (f'_{cm}), the average bending tensile stress after 28 days(f_{brm}), the elasticity modulus (E_b), the Poisson-number, and the linear thermal expansion coefficient (α). The formulas/tables to calculate these parameters are given below.

Concrete Class	the characteristic compressive cube strength after 28 days, f'_{ck} (N/mm^2)
C28/35	35
C35/45	45

Table 4 Characteristic Compressive Cube Strength after 28 Days (f'_{ck})

The Average Compressive Cube Strength after 28 Days (f'_{cm})

$$f'_{cm} = f'_{ck} + 8$$

The Average Bending Tensile Stress in Concrete Pavement after 28 Days (f_{brm})

$$f_{brm} = 1{,}3 \cdot \frac{1600 - h}{1000} \cdot \frac{1{,}05 + 0{,}05 \cdot (f'_{ck} + 8)}{1{,}2}$$

Elastic Modulus of Concrete

$$E_b = 22250 + 250 \cdot f'_{ck} \left(\frac{N}{mm^2}\right) \; where \; 15 \le f'_{ck} \le 65$$

Poisson-number (v): *0,15 – 0,20* (in this thesis assumed to be 0,15)

The Linear Thermal Expansion Coefficient (α): *$1*10^{-5}$ – $1.2*10^{-5}$* (°C^{-1}) (in this thesis assumed to be $1*10^{-5}$)

Traffic load stresses

The occurring bending tensile stresses due to a wheel load P_i are calculated at 3 normative positions:

- Free edges
- Transverse cracks
- Longitude cracks

These bending tensile stresses due to traffic loads are calculated for each possible wheel loads using the “new” Westergaard-formula:

$$\sigma_P = \frac{3(1+v)P_{design}}{\pi(3+v)h^2} \left\{ \ln\left(\frac{E_b h^3}{100ka^4}\right) + 1{,}84 - \frac{4}{3}v + \frac{1-v}{2} + 1{,}18(1+2v)\frac{a}{l} \right\}$$

Wherein:

σ_P = bending tensile stress (N/mm²)

P_{design} = wheel load (N)

a = radius of the circular contact area (mm)

E_b = elastic modulus of concrete (N/mm²)

v = Poisson value of concrete (in this thesis 0,15)

h = thickness concrete pavement (mm)

k = bearing value of the foundation (N/mm³)

l = $\sqrt[4]{\frac{E_b h^3}{12(1-v^2)k}}$ = relative stiffness radius

The design wheel load (P_{design}) and the radius of the circular contact area of the wheel (a), which are both calculated for all average wheel loads, and are calculated using the following equations:

The design wheel load (P_{design})

$$P_{design} = \left(1 - \frac{0{,}5W}{100}\right) \cdot P = \left(1 - \frac{W}{200}\right) \cdot P$$

Wherein:

W = Load transfer at the end of the design lifespan (%)

P = average wheel load (N) of the axle load class (Table 2)

Equivalent Radius of the Contact Area of the Tires

$$a = b \cdot \sqrt{(0{,}0028 \cdot P + 51}$$

Wherein:

b = parameter depending on tire type [6](Table 5)

P = average wheel load (N) of the axle load class (Table 5)

[6] The tire type parameter b is calculated by multiplying all b parameters with the percentage they occur in divided by 100 (35 → 0.35) based on the expected traffic for the road. The b parameters for each tire then get added up, to get a total value for the parameter b.

Tire Types	Width of the rectangular contact area (mm)	Value of Parameter b in formula 1 (lol)	Frequency Distribution (%)	
			Roads	Bus Lanes
Single Tire	200	9.2	39	50
Double Tire	200-100-200	12.4	38	50
Wide Tire	300	8.7	23	0
Super Wide Tire	400	9.1	0	0

Table 5 Value Parameter b for Different Tire Types and Their Frequency Distribution

After filling in the Westergaard-formula, the occurring bending tensile stresses for every *individual wheel load* are known.

Temperature gradient stresses

Together with the individual traffic load stresses for all wheel loads, the individual temperature gradient stresses are calculated for each temperature gradient occurring in the selected climate. In VENCON, only the stresses due to a positive temperature gradient at the edge of the pavement slabs are calculated, because these are always normative. These positive temperature gradients are then divided into a small positive temperature gradient, and a large temperature gradient.

Bending Tensile Stress due to a small temperature gradient

$$\sigma_T = \frac{h \cdot \Delta T}{2} \cdot \alpha \cdot E_b$$

Wherein:

σ_T = bending tensile stress due to small temperature gradient (^{o}C/mm)
h = thickness of the concrete plate
α = linear thermal expansion coefficient of concrete
E_b = elastic modulus of concrete (N/mm^2)
ΔT = average temperature gradient i (Table 3)

However, when big temperature gradients occur, a different formula for the bending tensile stresses due to temperature gradients becomes
Bending Tensile Stress due to a big temperature gradient

$$\sigma_T = 1{,}8 * 10^{-5} * \frac{L'^2}{h}$$
$$\sigma_T = 1{,}8 * 10^{-5} * \frac{B'^2}{h}$$

Wherein:

σ_T = bending tensile stress due to big temperature gradient (^{o}C/mm)
L' = span of the concrete slab in longitudinal direction (mm)
B' = span of the concrete slab in transverse direction (mm)
h = thickness of the concrete plate

The Span of the Concrete Slab

$$L' = L - \frac{2}{3} \cdot C$$
$$B' = B - \frac{2}{3} \cdot C$$

Wherein:

L = length of the plate (mm)
B = width of the plate (mm)
C = support length (mm) [EISENMANN]

$$C = 4{,}5 \cdot \sqrt{\frac{h}{k \cdot \Delta T}} \quad for\ C << L$$

ΔT = average temperature gradient i (Table 3)
k = bearing value of the foundation (N/mm^3)
h = thickness of the concrete plate (mm)

After using Eisenmann's theory, several possibly occurring temperature gradient stresses are known for each average temperature gradient. The actual occurring temperature gradient stress is always the lowest one of either 1) the formula for small gradients or 2) the formula for big gradients (depending on in which direction the stresses are calculated, either use the longitudinal or transverse gradient stress). The end result is a temperature stress for each occurring temperature gradient.

Thickness of the pavement

After calculating both the traffic load stresses and temperature gradient stresses, N_i (allowable axle load repetitions) is calculated for each possible combination of wheel load P_i and temperature gradient.[7] These individual allowable axle load repetitions are calculated using the following formula:

Fatigue Relation for Reinforced Concrete Pavements

$$\log N_i = \frac{12{,}903(0{,}995 - \sigma_{max} / f_{brm})}{1{,}000 - 0{,}7525\sigma_{min} / f_{brm}} \quad with\ 0{,}5 \le \sigma_{max} / f_{brm} \le 0{,}833$$

Wherein:
N_i = allowable number of wheel load repetitions
σ_{min} = minimum bending tensile stress (= σ_{Ti})
σ_{max} = maximum bending tensile stress (= $\sigma_{Ti} + \sigma_{Pi}$)
f_{brm} = average bending tensile strength after 28 days with short term loading (N/mm^2)

Where the minimum bending tensile stress is the lowest possible temperature gradient stress and the maximum bending tensile stress is the lowest possible temperature gradient stress combined with the traffic load stress.

To assess whether or not the designed construction is a good match to the occurring standard axle load repetitions the cumulative fatigue damage formula of Palmgren-Miner is used.

Cumulative fatigue damage (Palmgren-Miner):

$$\sum_i \frac{n_i}{N_i} = 1{,}0$$

Wherein:
n_i = occurring number of standard axle load repetitions during the design lifespan of the concrete pavement in combination with the temperature gradient
N_i = allowable number of repetitions during the design lifespan of the concrete pavement in combination with the temperature gradient

The occurring number of standard axle load repetitions (n_i) is calculated for each possible combination of wheel load P_i and temperature gradient, meaning that the total N_{eq} is multiplied with both the "Axle Load Frequency Distribution"/100 and the "Temperature Gradient Frequency Distribution"/100. This will result in an equivalent number of axle load repetitions for each combination. Seeing as the same has been done for the allowable number of wheel load repetitions,

[7] In case of a highway and the default temperature gradient, this will result in 10 (wheel/axle load classes) x 7 (temperature gradient classes) = 70 individual allowable axle load repetitions for the designed construction.

the cumulative fatigue damage relationship can now be utilized for each possible combination, which all added together should be 1,0. This is an iterative process, seeing as the height of the pavement is changed until the Palmgren-Miner equation equals zero.

Reinforcement CRCP
To calculate the reinforcement needed for the pavement, first of all the occurring strains are determined. These strains can be caused by either concrete shrinkage or temperature changes. Concrete shrinkage in VENCON is determined using the Eurocode (2 – design and calculation of concrete structures), and is seen as the combination of both autogenous shrinkage and drying shrinkage.
The autogenous shrinkage develops during the hardening of the concrete and the most important autogenous shrinkage thus happens in the first days after the pouring of the concrete. Autogenous shrinkage is a linear function of the concrete class/strength.

Autogenous Shrinkage

$$\varepsilon_{ca}(t) = \beta_{as}(t)\varepsilon_{ca}(\infty)$$

Wherein:
$\varepsilon_{ca}(t)$ = the autogenous shrinkage of concrete after t days
$\varepsilon_{ca}(\infty)$ = the autogenous shrinkage (dependent on the characteristic 28-day compressive strength)
t = time in days
$\beta_{as}(t)$ = time coefficient

The drying shrinkage develops (in contrast to autogenous shrinkage) slowly, because it's a function of the migration of water in the hardening concrete pavement. Drying shrinkage is dependent on 1) the thickness of the pavement, 2) the age of the pavement, 3) the relative humidity, and 4) the type of concrete and cement.

Drying Shrinkage

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s)k_h\varepsilon_{cd,0}$$

$\varepsilon_{cd,0}$ = base shrinkage
$\beta_{ds}(t, t_s)$ = time coefficient
k_h = coefficient for the nominal thickness

Total shrinkage

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$$

After calculating the total shrinkage, the difference in shrinkage between the top and bottom of the concrete pavement can be estimated, for which the CUR is used (recommendation 36). Assumed is that on top of the pavement, the total occurring shrinkage is 90% of the total shrinkage (ε_{cs}) and that on the bottom of the pavement, the total occurring shrinkage is 60% of the total shrinkage (ε_{cs}).

The shrinkage due to changing temperatures is also calculated using the following equations:

Temperature Differences in the Concrete Slab

$$\delta T_b = T_b - T_{rb}$$
$$\delta T_o = T_o - T_{ro}$$

Wherein:
T_b = temperature on top of the pavement (°C)
T_{rb} = reference temperature on top of the pavement (at the beginning of the pavement life, when there aren't any stresses due to the temperature changing) (°C)
T_o = temperature on the bottom of the pavement (°C)

T_{ro} = reference temperature on the bottom of the pavement (at the beginning of the pavement life, when there aren't any stresses due to the temperature changing) (°C)

Occurring Strains due to Temperature Differences

$$\varepsilon_{Tb} = \alpha * \delta T_b$$
$$\varepsilon_{To} = \alpha * \delta T_o$$

α = linear thermal expansion coefficient of concrete (°C^{-1})

Cracks will happen once the strain at the topside of the pavement reaches the critical value (ε_{cr}).

Critical Strain Reinforced Concrete Pavement

$$\varepsilon_{rb} + \varepsilon_{Tb} = \varepsilon_{cr} = \sigma_{cr}/E_b$$

Then the tensile strength of the (steel in the) concrete pavement is calculated at 3 specific times, slightly before cracks occur (σ_{cr}), immediately after cracks occur at the crack position ($\sigma_{s,cr}$), and after cracks occur (σ_s), since the tensile strength increases slightly. This is done using the following equations:

Tensile strength slightly before cracks occur

$$\sigma_{cr} = 0{,}54 \cdot \{1{,}05 + 0{,}05 \cdot (f'_{ck} + 8)\}$$

Tensile stress immediately after cracks occurring (reinforcement is centric in the pavement)

$$\sigma_{s,cr} = \sigma_{cr} \cdot \frac{1 + n \cdot \omega}{\omega}$$

Wherein:

σ_{cr} = stress just before crack formation

ω = reinforcement percentage:

$\omega = A_s / (B * h)$

Where:

A_s = steel cross section

B = width of the concrete plate

h = thickness of the concrete plate

n = ratio between elastic modulus of steel and concrete

Tensile stress immediately after cracks occurring (reinforcement is eccentric in the pavement)

$$\sigma_{s,cr} = \sigma_{cr} \cdot \frac{h \cdot (1 + n \cdot \omega)}{2 \cdot d \cdot \omega}$$

Wherein:

h = thickness concrete plate (mm)

d = distance to the center of the reinforcement from the underside of the concrete plate (mm)

Tensile strength after cracks occur

$$\sigma_s = \sigma_{s,cr} + \Delta\sigma_s$$

Increase in Steel Stress after cracks occur

$$\Delta\sigma_s = E_s \cdot \frac{(\varepsilon_{max} - \varepsilon_{cr})^2}{2 \cdot (\varepsilon_{sy} - \varepsilon_{cr} - \Delta\varepsilon_{ts})}$$

Wherein:

ε_{max} = max strain due to creep and temperature drop

ε_{cr} = strain just before crack formation

ε_{sy} = theoretical strain due to flow of the reinforcement

$\Delta\varepsilon_{ts}$ = decrease in the steel stress due to tension stiffening

After cracks forming, the average crack width and maximum crack width can be calculated using the following formulae:

Average Crack Width Incomplete Crack Pattern

$$w_{om} = 2 \cdot \left\{ \frac{0{,}4 \cdot \emptyset}{f'_{cm} \cdot E_s} \cdot \sigma_{s,cr} \cdot \left(\sigma_{s,cr} - n \cdot \sigma_{cr}\right) \right\}^{0{,}85}$$

Wherein:

$\emptyset$ = diameter reinforcement

$\sigma_{s,cr}$ = stress in the reinforcement at the crack immediately after occurrence

σ_{cr} = tensile stress in the plate just before crack formation

Max Crack Width

$$w_{o,max} = \gamma_{so} \cdot \gamma_{\sim} \cdot w_{om} < w_{allow}$$

Wherein:

γ_{so} = factor for the spreading of the crack width, in the incomplete patter =1,3

$\gamma_{\sim}$ = factor for long-term repeated loading:

For $\sigma_s \leq 295 \frac{N}{mm^2} : \gamma_{\sim} = 1{,}3$

For $\sigma_s > 295 \frac{N}{mm^2} : \gamma_{\sim} = \frac{1}{(1-9 \cdot \sigma_s^3 \cdot 10^{-9})}$

W_{allow} = max allowed crack width

$$w_{allow} = 0{,}2 \cdot k_c$$

Wherein:

k_c = c/c_{min} ($1 \leq k_c \leq 2$)

c = actual cover layer over reinforcement (mm)

c_{min} = minimum cover for reinforcement (= 35 mm

However, in reality, the cover layer is always bigger than 70mm, yielding a k_c that is equal to 2, which is used in this thesis.

Furthermore, in VENCON there isn't a formula that represents the influence of different types of reinforcements. However, in the background report it is specified that the pavement thickness can be decreased by 5 millimeters when reinforcement is used.

3.2.3 FLOOR 3.0

In the nineties, the association of the Dutch cement industry (VNC) took the initiative to develop a computer program FLOOR. Elastic supported concrete floors and pavements can be calculated using the program. The calculations are based upon Dutch standards, being:

- CUR – Recommendation 36-2011
- NEN-EN 1992-1-1 (Eurocode 2: design and calculation of concrete structures)
- NEN-EN 206-1 (concrete: specifications, properties, manufacturing, conformity)
- CROW – publication 220 'Manual concrete pavements – basic structures'
- CUR – Recommendation 65

The program FLOOR, in general, has been used for the dimensioning of industrial floors and pavements of unreinforced concrete and pavements of steel fiber concrete. With the new 3.0 version, which is considered in this thesis, it is now also possible to calculate reinforced concrete pavements. FLOOR's general structure is centered around designing a construction and then doing several checks. These checks are regarding the pavement's reinforcement, concentrated loads, static loads, and deformations (in case of CRCP) to see whether or not the construction can deal with the occurring forces/stresses/strains, as is represented in the flow chart in Figure 30.

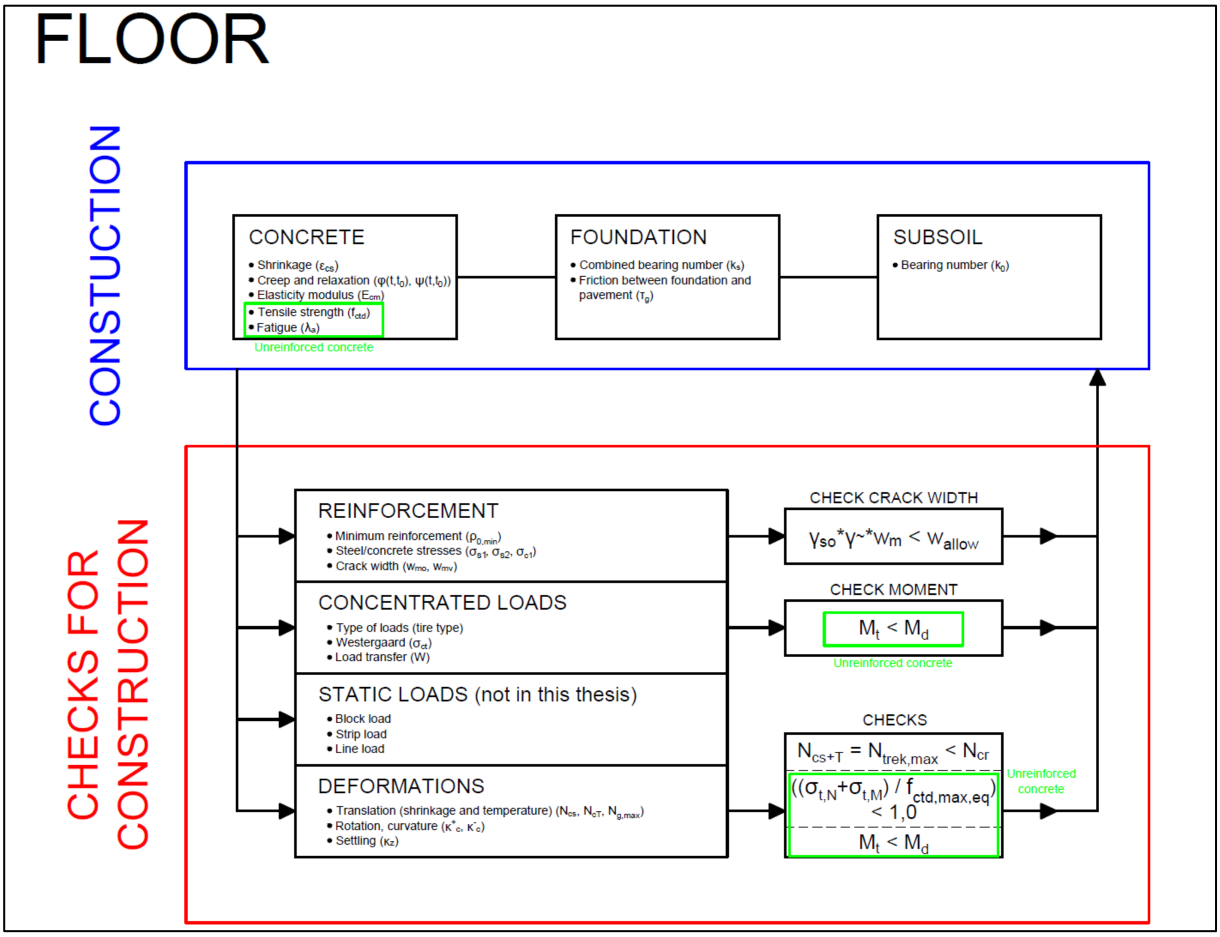


Figure 30 Flow chart of the FLOOR 3 model

FLOOR is very similar to VENCON, since both of the calculation methods originate from the VNC table book, used for concrete pavement calculation in the eighties. This will result in many of formula's looking very similar (or being the exact same) in FLOOR as they are in VENCON. In the following section, some parts might be explained briefer than they would normally due to this reason, the main emphasis will be on explaining the differences in structure between the two programs, and the parts that differentiate. The design model uses the following structure:

Concrete

Several effects, occurring in concrete pavements, are considered in FLOOR, these are 1) Shrinkage, 2) Creep and relaxation, 3) Elasticity modulus, 4) tensile strength, and 5) fatigue (only for unreinforced concrete pavements and steel fiber concrete). These effects will all be discussed in the following section.

1. Shrinkage

Shrinkage consists of both autogenous shrinkage and drying shrinkage. Autogenous shrinkage is caused by the hydration of cement and drying shrinkage is caused by the evaporation of water from the concrete, both influencing the pores, and resulting in the shrinkage of concrete. Both of the factors are calculated using the Eurocode (2 – design and calculation of concrete structures) in FLOOR, which is exactly the same as it is in VENCON.

Autogenous Shrinkage

$$\varepsilon_{ca}(t) = \beta_{as}(t)\varepsilon_{ca}(\infty)$$

Wherein the shrinkage is dependent on the time and the concrete class.

Drying Shrinkage

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s)k_h\varepsilon_{cd,0}$$

Wherein the shrinkage is dependent on the time passed, the concrete class, and the dimensions of the concrete pavement.

2. Creep and relaxation

Creep is a deformation of concrete that increases with time due to a constant load. When this load increases, the deformation will also increase. Often the material will not return to its original length after the load has decreased, this however depends on the sort of material being used, the magnitude of the load applied, and the length of the period that the load has been applied. In FLOOR, creep is calculated using the following formula, which also originates from the Eurocode (2):

$$\varphi(t, t_0) = \varphi_0 * \beta_c(t, t_0)$$

Wherein:

φ_0 = nominal creep coefficient

$\beta_c(t, t_0)$ = coefficient describing creep over time.

In this formula, the nominal creep coefficient is dependent on the relative humidity, the concrete strength (class) and a factor that represents the ageing of concrete.

Relaxation is the decrease of stress in a concrete pavement over time, in case of a constant deformation. Relaxation is an important and beneficial phenomenon in concrete pavements, because it counteracts the occurring stresses due to the volume changes in concrete pavements. This in turn decreases the cracking. In FLOOR two situations are considered, gradually occurring stresses and abruptly occurring stresses, which are calculated using the formula's below:

Relaxation coefficient due to gradually occurring stresses

$$\Psi(\mathrm{t}, t_0) = \frac{1}{1 + \rho\varphi(t, t_0)}$$

Relaxation coefficient due to stresses occurring suddenly

$$\Psi(\mathrm{t}, t_0) = 1 - \frac{\varphi(t, t_0)}{1 + \rho\varphi(t, t_0)}$$

Wherein:

$\varphi(\mathrm{t}, t_0)$ = Creep coefficient of concrete at time t due to the load applied at t_0

ρ = aging coefficient (average value = 0,8)

Elasticity Modulus

Again, this modulus is formulated in FLOOR based on Eurocode 2, and dependent on the concrete class and an aggregate factor:

28 Day Secant Modulus of Elasticity

$$E_{cm} = \alpha_E 22000(\frac{f_{cm}}{10})^{0,3}$$

Wherein:

E_{cm} = 28-day secant modulus of elasticity of concrete [N/mm^2]

f_{cm} = average 28-day compressive strength [N/mm^2]

α_E = aggregate factor

This leads to the following table of compression strengths and elasticity moduli:

Parameter	Concrete strength classes				
	C20/25	C28/35	C35/45	C45/55	C53/65
f_{ck}	20	28	35	45	53
$f_{ck, cube}$	25	35	45	55	65
f_{cm}	28	36	43	53	61
E_{cm}: α_E = 1,15 (basalt)	34456	37154	39189	41726	43523
E_{cm}: α_E = 1,0 (quartz)	29962	32308	34077	36283	37846
E_{cm}: α_E = 0,9 (limestone)	26966	29077	30669	32655	34062
E_{cm}: α_E = 0,7 (sandstone)	20973	22616	23854	25398	26492

Table 6 Compression strength and secant elasticity modulus of concrete for several types of aggregates (α_E)

Tensile strength (unreinforced concrete)

The tensile strength in FLOOR is calculated based on the compressive strength using the following formula:

Tensile Strength

$$f_{ctd,\infty} = \frac{f_{ctm,0}}{\gamma_m} = \frac{0{,}9[1{,}05 + 0{,}05(f_{ck,cube} + 8)]}{\gamma_m}$$

Wherein:

$f_{ctd,\infty}$ = design value of the long-term tensile strength of unreinforced concrete [N/mm^2]

$f_{ctm,0}$ = average 28-day short-term axial tensile strength of concrete [N/mm^2]

$f_{ck,cube}$ = characteristic 28-day compressive strength (cube) [N/mm^2]

γ_m = material factor for concrete under tension (=1,2)

This leads to the following table of average tensile strengths for the concrete classes:

Parameter	Concrete strength classes				
	C20/25	C28/35	C35/45	C45/55	C53/65
$f_{ck, cube}$	25	35	45	55	65
$f_{ctm,0}$	2.43	2,88	3,33	3,78	4,23
$f_{ctd,\infty}$	2,03	2,40	2,78	3,15	3,53

Table 7 Values for short- and long-term tensile strengths (28-days) of concrete

Fatigue (unreinforced concrete)

FLOOR also calculates the fatigue, based upon the axle load repetitions. This is the only time in the calculation where FLOOR considers axle load repetitions, meaning it doesn't consider them at all for reinforced concrete pavements.

Axle Load Fatigue

$$N = t_{old} \cdot n_{etm} \cdot n_{as} \cdot n_{days} \cdot p_{sv}/100$$

N = total number of axle load repetitions during the design lifespan

t_{old} = design lifespan [years]

n_{etm} = number of vehicles per day

n_{as} = average number of axles per vehicle

n_{days} = number of of use days per year

p_{sv} = percentage traffic in line

The axle load repetitions are then used to calculate the fatigue factor, which is a variable in a unity check of unreinforced pavements regarding a "testing" moment and the actual moment the pavement is designed/calculated for. This test will not be included in this report since this report primarily focusses on reinforced pavements.

Fatigue Factor

$$\lambda_a = 1{,}0 + a\left(\frac{\log N}{log2 \cdot 10^6}\right)\ (1 \leq N \leq 2 \cdot 10^6)$$
$$\lambda_a = 1{,}0 + a\ (N > 2 \cdot 10^6)$$

Wherein:
N = number of load repetitions
A = concrete type factor
= unreinforced concrete = 0,4
= steel fiber concrete = 0,2

Foundation/Subsoil
The subsoil and the foundation underneath the concrete pavement are taken into account in FLOOR via the VNC Sizing Method, which is also used in VENCON. It follows the following equation:

VNC Sizing Method 1992
$$k_s = 2{,}7145 \times 10^{-4}\ (C_1 + C_2 e^{C_3} + C_4 e^{C_5})$$

There are several conditions to which the composed bearing constant needs to adhere, which haven't been mentioned in this thesis before. These conditions can either be limitations for layer height or for the end result of the composed bearing constant:

Conditions (layer height):
h_2 ≥ 150mm (bound material foundation)
≥ 200mm (unbound material foundation)
≤ 400mm (sand bed)
≤ 600mm (foundation)

Conditions (end result):
$$\log k_s \leq 0{,}73688 \log E_2 - 2{,}82055$$
$$k_s \leq 0{,}16\, N/mm^3$$

Additionally, the foundation also influences the pavement through the friction with the concrete pavement. This friction is taken into account when calculating the deformations in the pavement structure, since maximum value for tensile forces in the concrete pavement will either be limited by the occurring normal force at which concrete cracks or by the maximum allowable mobilized normal force due to friction. The friction is dependent on several factors being the cohesion (τ0) and the friction (μ_0) which is multiplied by the evenly distributed load. Through the following equations:

Friction Between Concrete and Subsoil
$$\tau_g = \tau_0 + \mu_0 p$$

Reinforcement
The amount of reinforcement needed is determined based upon whether or not the crack width doesn't exceed the maximum allowable crack width. To calculate the crack width, two formulas can be used, depending on whether or not the cracking pattern is completed or not:

Average crack width *incomplete* crack pattern
$$w_{mo} = 2\left[\left(\frac{0{,}4\emptyset}{f_{ccm,0}E_s}\right)\sigma_{s,cr}\left(\sigma_{s,cr} - n\sigma_{cr}\right)\right]^{0{,}85}$$

Wherein:
Ø = index diameter of the given reinforcement [mm]
E_s = the modulus of elasticity of the reinforcement [N/mm^2]; (E_s = 200000 N/mm^2)
$f_{ccm,0}$ = the average 28-day cube compressive strength [N/mm^2]
n = E_s / E_{cm}
$\sigma_{s,cr}$ = the tension in the reinforcement at cracks in the concrete [N/mm^2]
σ_{cr} = the tension in the concrete at crack [N/mm^2]

Average crack width *complete* crack pattern

$$w_{mv} = 1{,}8 w_{mo} \left[\left(\frac{\sigma_s}{\sigma_{s,cr}} \right) - 0{,}5 \right]$$

Wherein:
w_{mv} = average crack width with a complete crack pattern [mm]
σ_s = the tension in the reinforcement at a crack at SLS: ($N_{BGT} > N_{cr}$)
$\sigma_{s,cr}$ = the tension in the reinforcement at cracks in the concrete [N/mm^2]

The required tensions in either the steel reinforcement or the concrete can be calculated at two different time's during the lifetime of the pavement; immediately after the first crack occurring or after the pavement's cracking pattern is completed. Then the crack width is checked using the following formula:

Crack Width

$$w_m \gamma_s \gamma_\infty \leq w_{req}$$

Wherein:
w_{req} = the shared crack width criteria by specific environmental class [mm]
w_m = average short-term crack width [mm]
γ_s = factor for imposed widening
Incomplete crack pattern:
: $\gamma_s = 1{,}3$
Complete crack pattern:
- Tension: $\gamma_s = 1{,}5$
- Bending: $\gamma_s = 1{,}7$

γ_∞ = factor for long-term / changing loading:
$\sigma_s \leq 295$ N/mm^2 $\gamma_\infty = 1{,}3$
$\sigma_s \geq 295$ N/mm^2 $\gamma_\infty = \frac{1}{1 - 9\sigma_s^3 10^{-9}}$

wherein:
σ_s = steel tension in the crack [N/mm^2]

Based on whether or not the construction meets this requirement is dependent on the reinforcement chosen. If the occurring crack width is higher than the criteria, the reinforcement needs to be made more robust, decreasing the steel tension, and thus decreasing the crack width. When the crack width is satisfactory, the crack distance can also be calculated using the following formula:

Average crack distance complete crack pattern

$$\Delta l_m = 1{,}8 w_{mo} \left(\frac{E_s}{\sigma_{s,cr}} \right)$$

Concentrated loads
Concentrated (wheel) loads in FLOOR are calculated roughly the same way as they are in VENCON, using Westergaard's formulas. Individual formulas are used to calculate the stress at: 1) the middle of

the plate, 2) the plate edge, and 3) the plate corner. These stresses are dependent on a number of variables such as the characteristic value for the concentrated load (wheel load), surface area of the tire, stiffness radius of the concrete plate and the thickness of the plate.

Concrete Stress at Middle of Plate

$$\sigma_{ct,inner} = \left(\frac{3F_k(1+v_c)}{2\pi h^2}\right)\left(\ln\left(\frac{2l_0}{b}\right) + 0{,}5 - \gamma_E\right) + \left(\frac{3F_k(1+v_c)}{64h^2}\right)\left(\frac{b}{l_0}\right)^2$$

Wherein:

F_k = characteristic value for the concentrated load [N]
v_c = coefficient for transverse contraction of concrete, = 0,20
h = thickness concrete plate [mm]
l_0 = stiffness radius concrete plate [mm]
γ_E = Euler constant (= 0,577216)
b = effective radius circular loading area [mm]

Concrete Stress at Plate Edge

$$\sigma_{ct,edge} = \left(\frac{3F_{k,red}(1+v_c)}{\pi(3+v_c)h^2}\right)\left(\ln\left(\frac{E_{cm}h^3}{100k_s a^4}\right) + 1{,}84 - \frac{4v_c}{3} + \frac{(1-v_c)}{2} + 1{,}18(1+2v_c)\left(\frac{a}{l_0}\right)\right)$$

Wherein:

$F_{k,red}$ = characteristic (reduced) value for the concentrated load [N]
v_c = coefficient for transverse contraction of concrete, = 0,20
h = thickness concrete plate [mm]
l_0 = stiffness radius concrete plate [mm]
a = effective radius circular loading area [mm]

Concrete Stress at Plate Corner

$$\sigma_{ct,corner} = \left(\frac{3F_{k,red}}{h^2}\right)\left(1{,}0 - \left(\frac{a\sqrt{\pi}}{l_0}\right)^{0{,}72}\right)$$

Load transfer is the ability of a pavement to distribute a wheel load over multiple plates, this leads to a decrease in load that an individual plate is receiving. In FLOOR the load transfer (W) is used to reduce the characteristic wheel load (F_k becomes $F_{k,red}$), and it is either chosen (based upon the type of joint being used) or it is calculated using the following formula:

Load Transfer at Plate Joints/Edges

$$W = 100\left(\frac{2w_0}{w_0 + w_b}\right)$$

Wherein:

W = load transfer [%]
w_0 = deflection unloaded plate edge [mm]
w_b = deflection loaded plate edge [mm]

Unity check

In the background report of FLOOR, only information has been provided regarding unity checks for unreinforced pavements. It is unclear whether or not these checks can also be applied to reinforced pavements, this is supported by the fatigue factor being used in the check, which is only specified for unreinforced pavements. This will be taken into consideration in the MCA, when the different variants will be compared.

With the unity check, 1) a calculated moment (M_d)[8] based on what the plate should be able to handle, and 2) a test/checking moment (M_t)[9]. When the calculated moment is higher than the test/checking moment, the construction satisfies the requirements. The calculated moment is calculated as follows:

$$M_d = \sigma_{ctd} * W$$

Wherein:
M_d = Calculated moment [kNm/m]
σ_{ctd} = Concrete stress at the edge of the plate, middle of the plate, or plate corner [N/mm^2]
W = the moment of resistance [mm^3/m]

The test/checking moment is dependent on the duration of the load:

Long lasting load (λ_a = 1): $$M_t = M_d$$

Short lasting load ($\lambda_a \geq 1$): $$M_t = \frac{M_d * \lambda_a}{1,4}$$

Wherein:
λ_a = fatigue factor [-]

Static loads
Static loads will not be discussed in this thesis, seeing as these loads aren't a significant variable in the design of pavements, in contrary to, for example, factory floors.

Deformations
In FLOOR there are several deformations considered in the calculations, these are:
- Shrinkage (autogenous + drying)
- Temperature changes
- Settling of the subsoil

These deformations are an important variable in the design of roads, since they need to be counteracted by the friction between the pavement and the foundation/subsoil together with the measures at the end of the end of the pavement, as has been discussed in *2.4 Rigid pavements and climate*. However, the background report doesn't make any statements regarding reinforcement end constructions and their influence on the obstructed deformations. FLOOR takes into account the frictional forces with the foundation/subsoil, with the following formula:

The Maximum Tensile Strength Due to Soil Tension

$$N_{g,max} = \frac{\tau_g b L}{2}$$

Wherein:
τ_g = soil tension induced shear stress at the underside of the plate [N/mm^2]
b = unit width of the concrete plate [mm], b = 1000 mm
L = length of the concrete plate [mm]

The normal forces occurring in the pavement are the sum of the normal force due to shrinkage and temperature changes:

Normal Force in Concrete Plate

$$N_{cs+T} = N_{cs} + N_{cT}$$

[8] This calculated moment is based on either the stress calculated at the middle of the plate, the plate edge, or the plate corner and on the moment of resistance.
[9] This test/checking moment is based on the fatigue factor

Wherein:
N_{cs} = tensile force in the concrete plate due to shrinkage [N]
N_{cT} = normal force in the concrete plate due to temperature change [N]

The normal force caused by shrinkage is mostly dependent on the strain due to shrinkage (ε_{cs}), which is calculated the same way as in VENCON, using the Eurocode (2).

Normal force Caused by Shrinkage

$$N_{cs} = -\varepsilon_{cs}\Psi E_{cm}A_c$$

Wherein:
ε_{cs} = imposed even shrinkage [mm/mm]; $\varepsilon_{cs} \leq 0$
A_c = concrete cross section per meter plate [mm^2]
Ψ = relaxation coefficient [-]
E_{cm} = 28 Day Secant Modulus of Elasticity [N/mm^2]

The normal force caused by temperature change is mostly dependent on the strain due to temperature change (ε_{cT}).

Normal Force as a Result of Temperature Change

$$N_{cT} = -\varepsilon_{cT}\Psi E_{cm}A_c$$

Wherein:
α_{cT} = linear expansion coefficient of concrete [mm/mm*K]; α_{cT} = 10.10^{-6} [mm/mm. ^{o}C]
ε_{cT} = strain due to even temperature change [mm/mm]
A_c = concrete cross section per meter plate [mm^2]
Ψ = relaxation coefficient [-]
E_{cm} = 28 Day Secant Modulus of Elasticity [N/mm^2]

The strain is calculated using an average change of the concrete temperature, which is an average of the change at the top and bottom of the pavement, to a reference temperature, which is taken when the concrete stress was 0, at the construction of the pavement.

Strain due to even temperature change

$$\varepsilon_{cT} = \alpha_{ct}\delta T$$

The construction then needs to satisfy the following formula, which states that in case of imposed deformation, the normal forces due to temperature and shrinkage will be normative, and will thus have to be lower than the normal force of the concrete when cracking.

$$N_{cs+T} = N_{trek,max} \leq N_{cr}$$

It is important to note that the background report does not contain any information regarding the normal force when cracking of reinforced concrete. This will be taken into account in the MCA.

The positive bending moment in the concrete plate, which takes into account a transverse curvature occurring in the pavement, can also be calculated using the following formulas.

Positive Bending Moment in Concrete Plate

$$M_1 = E_{cm}I_{c0}\kappa_c^+\Psi$$

$$M_2 = \frac{pl_{ef}^2}{8}$$

Wherein:
κ_c^+ = positive imposed bending (due to a temperature gradient and settling of the subsoil) [mm/mm^2]; $\kappa_c^+ \geq 0$

κ_{g0} = limit value positive bending [mm/mm^2]
p = evenly distributed load per unit of plate width [N/mm^2]
l_{ef} = effective length [mm]
E_{cm} = secant elastic modulus of concrete [N/mm^2]
I_{c0} = moment of Inertia concrete cross section [mm^4]
Ψ = creep coefficient

The positive bending moment in the concrete plate, which takes into account a transverse curvature occurring in the pavement, can also be calculated using the following formulas.

Bending moment due to negative bending:

$$M = -\frac{p(L-a)^2\Psi}{8}$$

Maximum moment with no bending in long plates

$$|M| \leq |E_{cn} I_{c0} \kappa_c^- \Psi|$$

Wherein:
E_{cm} = 28-day elastic modulus of concrete [N/mm^2]
I_{c0} = moment of Inertia concrete cross section [mm^4]
κ_c^- = negative imposed bending [mm/mm^2]; $\kappa_c^+ < 0$
Ψ = creep coefficient

In which the negative Bending Moment (κ_c^-) is composed of a negative bending due to shrinkage, temperature changes, and settling.

There are additional unity checks given in the background report, which are much alike the unity checks detailed for the concentrated loads. These take several things into account:
- the moment occurring because of the stresses caused by normal forces (shrinkage/temperature)
- the positive bending moment
- the negative bending moment

These moments are then being compared to the allowable moment to check whether or not the pavement construction satisfies the requirements. However, since this is all done in case of unreinforced pavements, and it is not made clear what needs to be done in case of reinforced concrete, this is beyond the scope of this thesis, due to time restraints. This will also be taken into consideration in the MCA.

3.3 Criteria for the MCA

To analyze the different variants for calculating continuously reinforced concrete pavements, a Multi Criteria Analysis (MCA) will be done. The goal of this MCA is to determine which design method is best suited to serve as a basis/starting point for the model that will be developed to analyze the relationship between the changing climate, and thus higher temperature (fluctuations) and the thickness of the concrete pavement. All the variants (design methods for CRCP) will be evaluated based on several criteria. These criteria are: traffic applied loads (N_{eq}), Traffic effective stresses (σ_P), Temperature effective stresses (σ_T), Reinforcement, variation in material properties, and bearing capacity of the subsoil. All the criteria will be briefly described in this section.

Traffic applied loads

The first criterion will be the traffic applied loads. In the design of continuously reinforced concrete pavements (CRCP's) an important part is making a prediction regarding the amount of traffic that is expected to use the road. This traffic is described through the amount of standard axle load repetitions that are expected. These standard axle load repetitions are the number of times a certain axle load is expected on the road. Depending on the variant, a standard axle load will either be 80 kN or 100 kN, this difference will be negated when needed during calculations. The standard axle load repetitions are as effective as they are, since they don't simply count the amount of vehicles that use the road, they take into account what the damage is a certain type of vehicle does, since a heavily loaded truck will cause more damage to the pavement than a small car. In the multi criteria analysis, the variants will be compared based on how representative their calculation for the traffic applied loads is of reality, in other words, into how much detail do the variants go when describing the expected traffic.

Traffic effective stresses
The second criterion are the traffic effective stresses that are exerted on the pavement. These bending tensile stresses are caused by the several wheel loads of the traffic that uses the road. These stresses occur at the bottom of the pavement at the free edge of the pavement, in the transverse cracks, and the longitudinal joints (in case of a pavement that is not reinforced). These stresses, together with the temperature effective stresses and the properties of the concrete, are a major factor in determining the allowable load that a pavement can withstand. How these stresses are formulated/calculated is thus important, since this directly influences the allowable load that the pavement is designed for.

Temperature effective stresses
The following criterion are the temperature effective stresses that are exerted on the pavement. In this thesis the relationship between temperatures (variations) and the thickness of the pavement is the research question. It is thus very much vital to the thesis that the variable "temperature", when designing a concrete pavement, is analyzed and shown through graphs. The way in which the temperature is a factor in the design, is mostly through the stresses that are caused by a temperature gradient in the pavement construction. These stresses are caused by normal forces, exerted on the pavement structure in the form of the concrete volume changing. Volume changes occur both during the early ages after concrete placement and during the entirety of the pavement lifetime. During the early age of a concrete pavement volume changes are caused by temperature and moisture changes. The normal force due to shrinkage continues over the entirety of the pavement's lifetime. Additionally, the concrete pavement also slightly expands with increasing temperatures and slightly shrinks with decreasing temperatures, which also yields a normal force. The variants will be scored based upon how they take these stresses into account in the design of the concrete pavement.

Reinforcement
The next criterion is the calculation of the reinforcement needed for the CRCP. When a pavement is continuous, tensile forces are higher than when using, for example, concrete slabs. To counteract these tensile forces, reinforcement is needed, and vital in preventing the concrete from expanding/curving more than is allowed/preferred. The concrete pavement expanding/curving can be caused by factors such as temperature variations and settlements of the subsoil.
However, the most crucial function the reinforcement has in CRCP is to control the crack width. It is vital for the strength/durability of a CRCP that the cracks are as small as possible and that the pattern is predictable. Wide cracks will initially allow chemicals to penetrate the concrete layer and damage the concrete/steel reinforcement. Wide cracks can eventually also cause punchout, which is far from desirable since punchout could signify the end of the lifespan of the concrete pavement, since a concrete pavement is often entirely replaced once a part is irreversibly damaged. The variants will be scored based upon the detail in which the reinforcement is calculated.

Variation material properties
The material properties of the construction can be an important factor in the design. Properties of the concrete, foundation, and reinforcement steel can change for specific projects, to account for certain circumstances. The properties of concrete, for example, are heavily dependent on the used aggregates. Additionally, the consistence, compaction and post-treatment (for example curing) are leading when it comes to the quality of the concrete pavement, and especially the surface of the concrete pavement. (Romijn, 2022) Being able to change these parameters is vital in the design, and not often not possible in design programs, due to there being made a lot of assumptions, when the program takes the (set) input values and converts them to an end-result. The program in which these material properties can be varied the best will score the highest for this criterion.

Bearing capacity
The stability and support of the foundation, which get translated to a bearing capacity, is the next criterion for the analysis. The foundation of the concrete pavement entails all the layers underneath the concrete pavement, these layers could be a (natural) subsoil, a sand bed, and a small asphalt concrete layer, right underneath the concrete pavement. The degree of support is expressed through the bearing constant at the top of the foundation. The bearing constant directly influences both the traffic effective stresses that are applied on the pavement and the amount of curvature occurring at the pavement. The variants will be scored based upon the detail in which the foundation is taken into account when calculating the bearing constant (or in the case of AASHTO, modulus of subgrade reaction).

3.4 Weight of the criteria

After being described, all these criteria have been assigned a weight, the most important criterion for this thesis will receive the highest weight, and the criterion with the lowest importance will receive the lowest weight.

Criteria	Factor
Traffic applied loads	0.20
Traffic effective stresses	0.15
Temperature effective stresses	0.35
Reinforcement	0.10
Variation material properties	0.15
Bearing capacity subsoil	0.05

Table 8 Weights for the criteria in the Multi Criteria Analysis

Traffic applied loads – 0.20
Describing the traffic load expected to be exerted on the pavement construction is, together with the determination of the allowable axle load repetitions, central in the calculation of the pavement thickness. The traffic (applied loads and effective stresses) and temperature (effective stresses) are seen as equally important in this thesis, due to these two factors being central in the design, or research question. This results in the traffic and temperature having the same factor (0.35). Regarding traffic, this factor is divided between the effective stresses and applied loads, the distribution is a bit skewed towards the applied loads due to this factor being primarily responsible for the calculation of the expected standard axle load repetitions.

Traffic effective stresses – 0.15
As stated before, describing the influence of the traffic on the design is important, both via the expected axle load repetitions and the stresses caused by the traffic. Seen as a combined factor of 0.35 is used for both these criteria, and traffic applied loads is deemed slightly more important, this yields a factor of 0.15.

Temperature effective stresses – 0.35
This criterion is the most vital when it comes to answering the research question, and this thus yields the biggest factor. The most detailed/accurate model regarding the temperature stresses will result in the most precise answer for the research question.

Reinforcement – 0.10
Reinforcement is important when it comes to controlling the crack widths and crack pattern, however, this is not the primary topic of research in this thesis, which thus yields a lower score, relative to most of the other criteria.

Variation material properties – 0.15
The material properties can be significantly different between multiple projects, and being able to adjust the program to these material properties of for example concrete, steel or subsoil is very important, resulting in a factor of 0.15 in this thesis.

Bearing capacity – 0.05
The bearing capacity, despite being important in the construction of concrete pavements is deemed to be the least divisive when calculating concrete pavements with various methods. This is especially visible due to the bearing capacity being very similar, if not the same, between different design methods.

In this MCA all variants will receive scores for each criterion that will range from 1 to 5, being:

- 1: Poor
- 2: Mediocre
- 3: Average
- 4: Good
- 5: Great

These scores will be multiplied with the assigned weight of the respective criterion. The summation of all these scores will yield a score for the entire variant. The variant with the highest score is most applicable for this thesis.

4. Results

4.1 Climate

From the climate data available from the KNMI (KNMI, 2022), the variables minimum and maximum temperature have been used to analyze the temperature trends. These minimum and maximum temperatures have been used to create a third variable, the temperature fluctuation, which represents the difference between a daily maximum and minimum temperature. As said before, the data available from the weather station at “de Bilt” will be used in this thesis, using the data from the previous 60 years. To make the data easier to use, it will be grouped to sets, each consisting of the average values of 5 years of data. Each of these datapoints will then be used to form a line, representing the average values and thus the trend over the past 60 years at weather station “de Bilt”. These trends can be seen in Figure 31. An upward going trend is very much notable, since all variables seem to be increasing, which was ought to be expected, due to the documentation of climate change over that period.

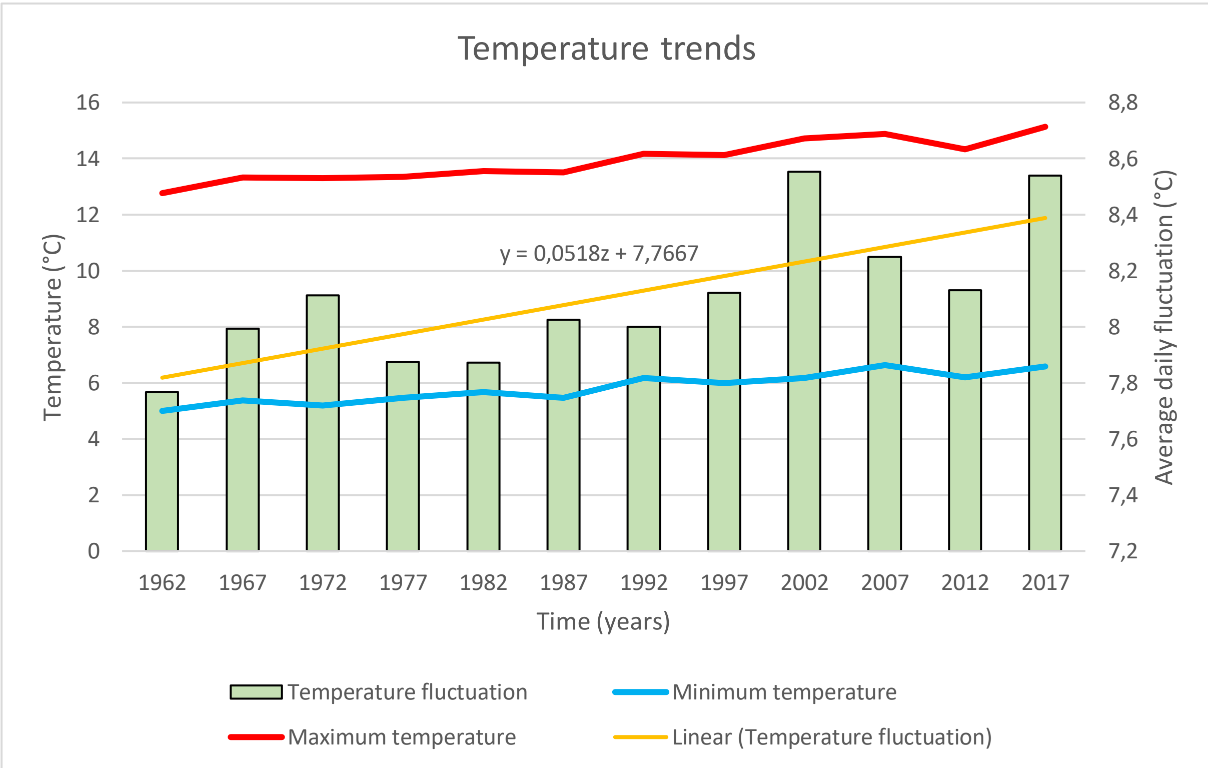


Figure 31 Temperature trends in "de Bilt", Netherlands

A second manner of showing the development of the temperature fluctuations can be given through the number of occurrences of temperature fluctuations in between a certain specified range[10]. These ranges were used to tally occurrences over a period. For this period, 10 years was used, due to a period of 5 years not yielding a clear trend. The results can be seen in Figure 32,

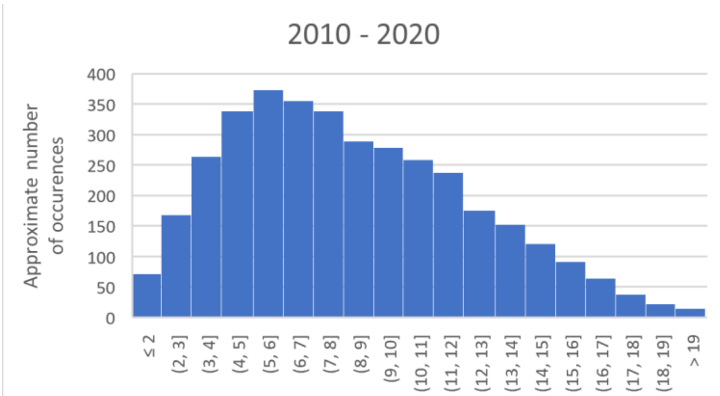


Figure 32 Daily fluctuations occurring between 2010 and 2020

[10] A range could for example be daily fluctuations between 5 and 6 degrees Celsius

where a histogram was made from the occurrences of temperature fluctuations in between 2010 and 2020.
Next, several trendlines were made using the histograms of all of these periods from 1960 until 2020. These trendlines then were combined into Figure 33. In this figure it can be seen that over the years, the graph becomes more skewed. In other words, the most common occurrences happen less, and the most extreme occurrences (in this case the highest fluctuations) happen more often. It also seems to be the case that over the last 20 years the trend of increased temperature fluctuations has been accelerated, because of the representing lines being significantly more skewed than the others.

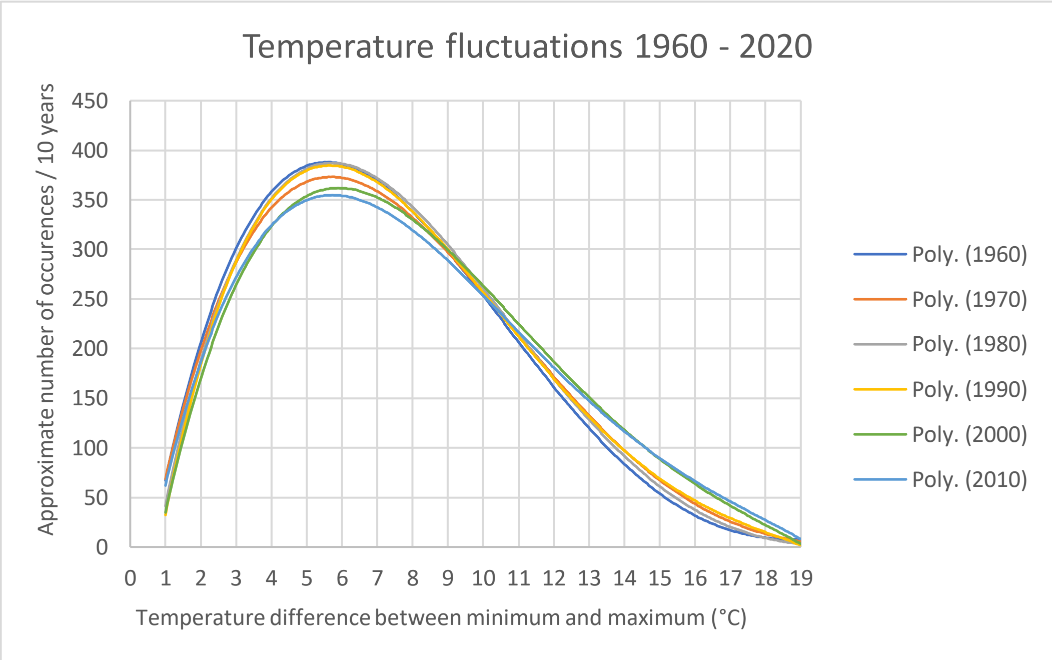


Figure 33 Daily temperature fluctuations in "de Bilt", Netherlands over a 60-year period

Apart from the minimum and maximum daily temperatures, the KNMI also keeps track of the minimum temperatures 10 centimeters above the surface (ground). When comparing these data to the minimum temperatures, it quickly becomes clear that the temperature decreases towards the surface. (Figure 34) However, since there is no data known about maximum temperatures 10 centimeters above the surface, no conclusions can be made about temperature fluctuations 10 centimeters above the surface differentiating from the other measurements, taken at 1,5 meters above the surface.

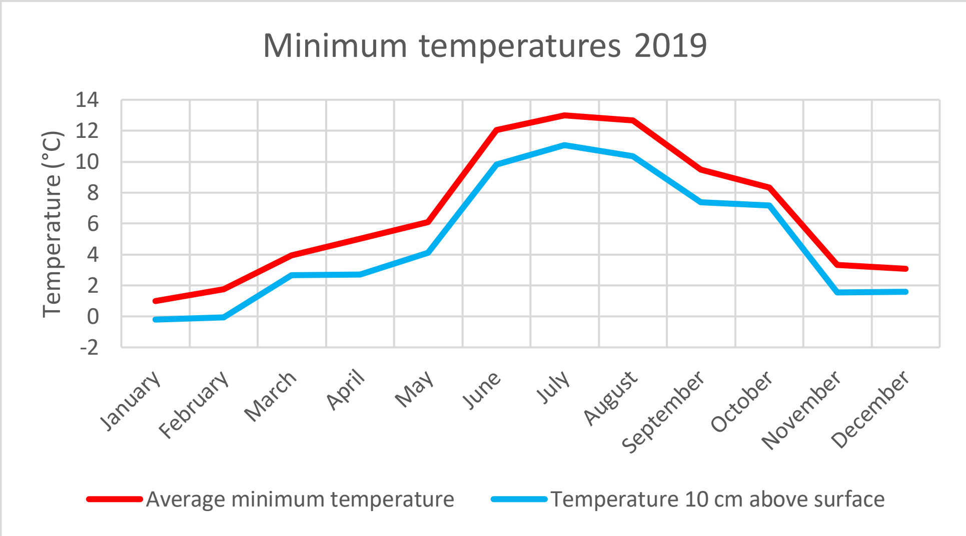


Figure 34 Minimum temperatures in 2019 in "de Bilt", Netherlands

After observing the temperature trends, the next step is to use the trends to make a prediction for the future. For this thesis, 3 possible situations will be used:

1. The temperature will stay as it was in the latest datapoint, averages in 2019;
2. The temperature will continue according to the established trends;
3. The temperature trend will accelerate, by a factor of 2.

The trends can be quantified by computing a trendline that fits best to the datapoints. These trendlines are given in Figure 35 (maximum and minimum temperature) and Figure 31 (temperature fluctuations).

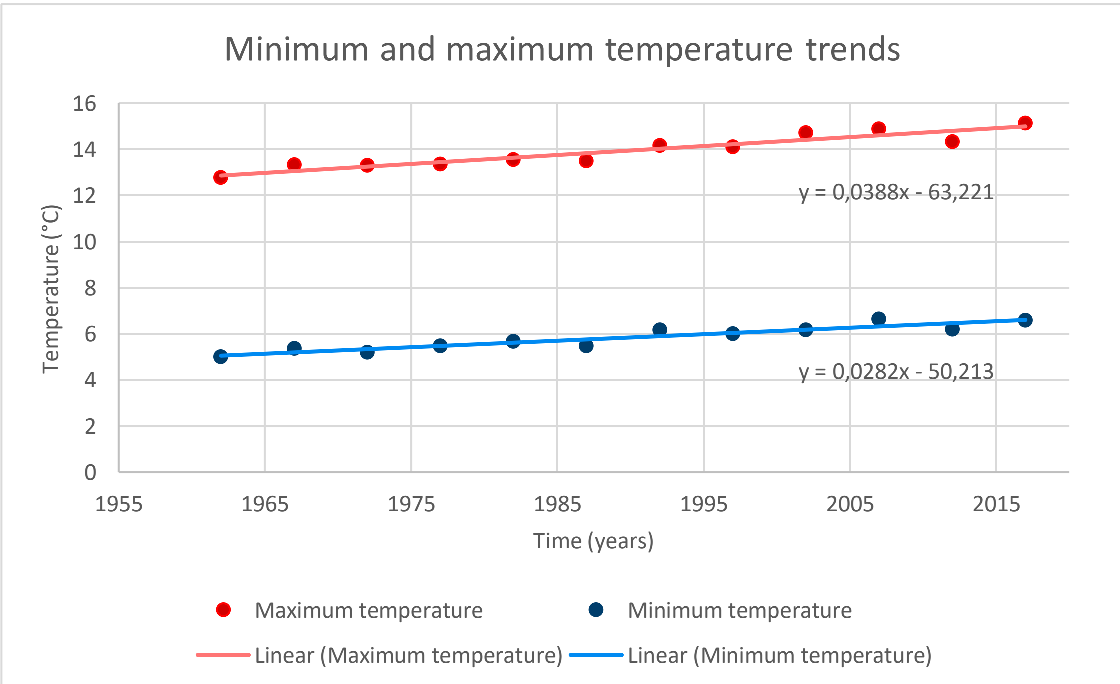


Figure 35 Minimum and maximum temperature trends in "de Bilt", Netherlands

In **situation 1** the following values will be used for the next 60 years:
Minimum temperature = 6.20 °C

Maximum temperature = 14.33 °C
Temperature fluctuation = 8.13 °C

In **situation 2** the values of Table 9 will be used as average for the next 60 years, using the following formula's:

$$Minimum\ temperature: y = 0{,}0282 * x - 50{,}213$$
$$Maximum\ temperature: y = 0{,}0388 * x - 63{,}221$$
$$Temperature\ fluctuation: y = 0{,}0518 * z + 7{,}7667$$

Where z = 1 in 1962 and increases by 0,2 every year.

	2020 - 2030	2030 - 2040	2040 - 2050	2050 - 2060	2060 - 2070	2070 - 2080
Minimum	6,89	7,17	7,46	7,74	8,02	8,30
Maximum	15,35	15,74	16,13	16,51	16,90	17,29
Fluctuation	8,47	8,52	8,57	8,63	8,68	8,73

Table 9 Situation 2 predictions for minimum/maximum temperatures and temperature fluctuations

In **situation 3** the values of Table 10 will be used for the next 60 years:

	2020 - 2030	2030 - 2040	2040 - 2050	2050 - 2060	2060 - 2070	2070 - 2080
Minimum	7,12	7,68	8,25	8,81	9,37	9,94
Maximum	15,66	16,44	17,21	17,99	18,76	19,54
Fluctuation	8,52	8,63	8,73	8,83	8,94	9,04

Table 10 Situation 3 predictions for minimum/maximum temperatures and temperature fluctuations

A challenge during this thesis will be is creating a link between the temperature variations detected at the weather station "de Bilt" and the temperature variations that as a result of those will occur in a concrete pavement. Field test are required to make an accurate estimate, however, since that is beyond the scope of this thesis, the percentage increase of the temperature variations will be used to estimate the expected increase in temperature gradient in the pavement. This percentage increase will be represented by a factor to account for an increase in temperature gradient for all situations:

Temperature fluctuations	2020 - 2030	2030 - 2040	2040 - 2050	2050 - 2060	2060 - 2070	2070 - 2080
Situation 1	1,000	1,000	1,000	1,000	1,000	1,000
Situation 2	1,042	1,048	1,054	1,062	1,068	1,074
Situation 3	1,048	1,062	1,074	1,086	1,100	1,112

Table 11 Factor for temperature fluctuations

4.2 Multi Criteria Analysis

4.2.1 Reporting the results

[weight factor] Criteria	AASHTO	VENCON	FLOOR
[20] Traffic applied loads	3	5	1
[15] Traffic effective stresses	1	5	4
[35] Temperature effective stresses	1	5	4
[10] Reinforcement	4	5	5
[15] Variation material properties	1	3	5
[05] Bearing capacity subsoil	4	4	5
Score	2.55	4.65	3.55

Table 12 Multi Criteria Analysis results

In Table 12 an overview of the results is given for each of the design methods. In the following section the results for each of the criteria will be discussed.

Traffic applied loads
Both VENCON and AASHTO take into account equivalent standard axle load repetitions as a factor in calculating the thickness of their pavement, where FLOOR only calculates it for determining the fatigue occurring in unreinforced concrete pavements. However, since only continuously reinforced concrete pavements are considered FLOOR, has been awarded the lowest score. The difference between AASHTO and VENCON is in the number of factors being considered when calculating the equivalent standard axle load repetitions. AASHTO specifies the vehicle types, growth factors and a design factor for the axle load repetitions, but doesn't include specifically the lifespan, days of usage/year, rutting which VENCON does include. This is why VENCON gets a maximum score and AASHTO gets 3/5.

Traffic effective stresses
VENCON and FLOOR have a very similar approach when it comes to traffic, calculating the stresses in locations such as the longitudinal/transverse cracks/joints, the edge of the pavement and the middle of a slab. To do these calculations the Westergaard's formulas are used, which takes into consideration factors such as a characteristic wheel load, contact surface of the tire, strain (Poisson factor), load transfer over the cracks/joints, elasticity modulus of concrete, and in case of VENCON also the foundation. However, where VENCON takes reinforcement into account, this is not the case for FLOOR. FLOOR mentions reinforcement, but crucial parts in the background report are missing. This difference between VENCON and FLOOR is the reason for the different scores. AASHTO does not consider checking occurring stresses and whether or not the pavement construction can deal with those stresses. That's why this model gets the lowest possible score for this criterion.

Temperature effective stresses
Both FLOOR and VENCON consider stresses (or forces) that occur due to temperature changes. These stresses are calculated for the same locations as the traffic effective stresses; however, they do rely on different methods. FLOOR's temperature effective stresses are based on the CUR, which considers both shrinkage and temperature stresses. VENCON's approach is a variation of Eisenmann's formula's, which are used in VENCON for small temperature gradients. For larger temperature gradients a different method is used in which the curvature of the road is considered. Both FLOOR and VENCON consider quite a few variables when calculating the temperature stresses, however, FLOOR solely provides a way of calculating unreinforced/steel fiber concrete and not reinforced concrete, yielding a slightly lower score than VENCON. AASHTO does not consider checking occurring stresses and whether or not the pavement construction can deal with those stresses. That's why this model gets the lowest score for this criterion.

Reinforcement
In VENCON both temperature and shrinkage tensile strains are considered using the axial force – strain relation diagram in reinforced concrete, steel and concrete stresses are calculated, and used to determine a relationship between the crack dimensions and the amount of reinforcement. FLOOR roughly follows the same method of calculation. This method is widely used and supported by research, resulting in a maximum score. AASHTO design its reinforcement calculations is based on allowable crack spacing, crack width and the stresses occurring in the steel. This is an iterative process in which the dimensions of the reinforcement get changed in order to meet with regulations. This process however is in less detail than VENCON and FLOOR, hence the score being slightly lower.

Variation material properties
In AASHTO there is not any room in the calculations to vary with different material properties, a lot is for example determined based on the concrete class being used for the pavement. This is why

AASHTO scores the lowest in this criterion. In VENCON, there is room to differentiate with material properties, but this still is very limited. Based on the output from a calculation in the program (*Appendix E Input designed model for Beverentunnel),* in which the input is noted down, it can be seen that most of the input is very limited. Variation is possible by changing the concrete class, foundation and reinforcement, but these, especially for the concrete properties, don't go into detail Most of the inputs is regarding the traffic load expected on the pavement, which does not qualify as being able to vary with material properties. This is why VENCON scores "average" for this criterion. FLOOR does allow its user to widely vary with the properties of the materials used. Examples of properties being able to manually changed regarding concrete are: the concrete strength class, the relative humidity, 28-day bending and split tensile strength, the aggregates used, the shrinkage/post treatment measures, the cement class, and creep/relaxation. This is the reason that FLOOR scores the highest in this category.

Bearing capacity

Both FLOOR and VENCON use the VNC dimensioning method to calculate the bearing capacity of each of the layers, based on the bearing capacity of the layer below, the elasticity modulus of the layer, and thickness of the layer. This yields a bearing capacity that represents the foundation. FLOOR then additionally also calculates the friction between the pavement and foundation, which is something VENCON doesn't include in their calculations, that is why VENCON's score is slightly lower than FLOOR's.

In AASHTO the bearing capacity is represented by the modulus of subgrade reaction. It mainly is influenced by laboratory tests of the soil used on a specific project, after which a yearly average of the composite modulus is calculated and adjusted based on relative damage and possible loss of support. Due to these laboratory tests, the modulus of subgrade reaction is highly representative of specific projects, resulting in the same score as VENCON.

4.3 Assumptions

In this section all assumptions that were made primarily for the model that was analyzed and in lesser extent also for the representation of the AASHTO model. The assumptions for the own model were mostly based on the case study of the "Beverentunnel" and are mostly arbitrary.

4.3.1 Subsoil

Assumptions	Value	Unit
Ground Water Level	-1	m
Settlement	Consolidated	-
Frost/Thaw sensibility	None	-
Cables and pipes	None	-
Water drainage (AASHTO)	Good	-
Modulus of subgrade reaction (k - AASHTO)	320	pci
Drainage coefficient (C_d - AASHTO)	1.05	-
Bearing number *subsoil* (k_0 – MODEL)	0.147 (sand-gravel)	N/mm^3
Thickness *foundation 1* (h – MODEL)	250	mm
Elasticity modulus *foundation 1* (E_{cm} - MODEL)	800 (concrete granulate)	N/mm^2
Thickness *foundation 2* (h – MODEL)	50	mm

Elasticity modulus *foundation 2* (E_{cm} - MODEL)	7500 (asphalt)	N/mm^2

Table 13 List of subsoil assumptions made for this thesis

4.3.2 Material properties

Concrete

Assumptions	Value	Unit
Concrete class (MODEL)	C35/45 or C28/35	N/mm^2
Modulus of elasticity (E_c - AASHTO)	3604997	psi
Linear expansion coefficient (α - MODEL)	0,00001	mm/mm*K
Weight density concrete (ρ_c - MODEL)	24	kN/m^3
Modulus of rupture (S'_c - AASHTO)	578	psi
Poisson number (v - MODEL)	0.15	-

Table 14 List of concrete assumptions made for this thesis

Steel

Assumptions	Value	Unit
Steel grade	BE500	-
Yield strength (f_{yk})	235	N/mm^2
Minimum diameter reinforcement steel	8	mm
Minimum cover for reinforcement (c_{min})	35	mm
Actual cover in practice for reinforcement (MODEL)	(At least) 70	mm

Table 15 List of steel assumptions made for this thesis

4.3.3 Climate effects

For the temperature effects the situation in the Netherlands is analyzed, where the KNMI collects data in weather stations all over the country. In this thesis the data from the weather station in “de Bilt” is used. This is because of the KNMI headquarters being there and its central position in the country, however most importantly, the weather in “de Bilt” is used as representative for the rest of the Netherlands. Three situations were analyzed:

1. The temperature will stay as it was in the latest datapoint, averages in 2019;
2. The temperature will continue according to the established trends;
3. The temperature trend will accelerate, by a factor of 2.

These situations led to the following assumptions:
In **situation 1** the following values and

Temperature fluctuations	2020 - 2030	2030 - 2040	2040 - 2050	2050 - 2060	2060 - 2070	2070 - 2080
Situation 1	1,000	1,000	1,000	1,000	1,000	1,000
Situation 2	1,042	1,048	1,054	1,062	1,068	1,074
Situation 3	1,048	1,062	1,074	1,086	1,100	1,112

Table 11 will be used for the next 60 years:
Minimum temperature = 6.20 °C

Maximum temperature = 14.33 °C
Temperature fluctuation = 8.13 °C

In **situation 2** the values of Table 9 and

Temperature fluctuations	2020 - 2030	2030 - 2040	2040 - 2050	2050 - 2060	2060 - 2070	2070 - 2080
Situation 1	1,000	1,000	1,000	1,000	1,000	1,000
Situation 2	1,042	1,048	1,054	1,062	1,068	1,074
Situation 3	1,048	1,062	1,074	1,086	1,100	1,112

Table 11 will be used as average for the next 60 years

In **situation 3** the values of Table 10 and

Temperature fluctuations	2020 - 2030	2030 - 2040	2040 - 2050	2050 - 2060	2060 - 2070	2070 - 2080
Situation 1	1,000	1,000	1,000	1,000	1,000	1,000
Situation 2	1,042	1,048	1,054	1,062	1,068	1,074
Situation 3	1,048	1,062	1,074	1,086	1,100	1,112

Table 11 will be used for the next 60 years:

Additionally, for AASHTO the "Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation" is assumed to be 5-25%

4.3.4 Traffic loads

Assumptions	Value	Unit
Load transfer coefficient (J – AASHTO)	3.05	-
Directional distribution factor (D_D – AASHTO)	0.50	-
Lane distribution factor (D_L – AASHTO)	0.90	-
Design lifespan road (MODEL)	30	years
Days of usage road (MODEL)	300	days per year
Number of lanes (MODEL)	2	Lanes
Factor heavy traffic distribution, heaviest lane (MODEL)	0.95	-
Factor rutting (at free edge and longitudinal joints - MODEL)	1.00	-
Factor rutting (at transverse cracks - MODEL)	0.92	-
Average number of axles (MODEL)	3	Axles
Traffic growth (MODEL)	3	% per year
Load transfer at longitudinal joints (MODEL)	70	%
Load transfer at free edges (MODEL)	35	%
Load transfer at transverse cracks (MODEL)	80	%

Traffic right next to longitudinal joints (MODEL)	10	% of total
Traffic at the free edge (MODEL)	3	% of total
Traffic at transverse cracks (right lane - MODEL)	95	% of total
Traffic at transverse cracks (left lane - MODEL)	4	% of total
Traffic at transverse cracks (emergency lane - MODEL)	1	% of total
Tire spectrum - Wide tires - Double air - Single air	 23 38 39	 % % %

Table 16 List of Traffic loads assumptions made for this thesis

4.3.5 Extra assumptions

Assumptions	Value	Unit
Serviceability index (ΔPSI – AASHTO)	(4.5 – 2.75) = 1.75	-
Combined standard error of traffic/performance prediction (S_0 – AASHTO)	0.35	-
Standard normal deviate (Z_R – AASHTO)	-1.645	-
Width of the pavement (MODEL)	3.75	m
Length of the pavement (MODEL)	4.5	m
Strains due to shrinkage topside of pavement (ε_{rb}) (MODEL)	0.9*ε_{cs}	-
Strains due to shrinkage topside of pavement (ε_{rb}) (MODEL)	0.6*ε_{cs}	-

Table 17 List of extra assumptions made for this thesis

4.4 Model based on VENCON

To analyze the effects of increasing temperatures and temperature variations in the next 60 years on the design of continuously reinforced concrete pavements used for constructing highways, a model was made in excel, based on the winning variant of the multi criteria analysis in *4.2 Multi Criteria Analysis*. The input values used for this model have been stated in *4.3 Assumptions*, are arbitrary and mostly based on a project regarding the Beverentunnel. These input values can be categorized:

- Equivalent standard axle load repetitions
- Materials (concrete)
- Foundation
- Temperature gradient

For the equivalent standard axle load repetitions, several assumptions were made regarding; design lifespan, days of usage, number of lanes, heavy traffic, rutting, number of axles, traffic growth, load transfer, traffic distribution on the road, and tire spectrum. However, there is one factor that will vary based on the analysis, which is the "axle load frequency distribution". This factor represents which type of road is being analyzed and the overview is given in Table 2. Normally in this thesis a heavy loaded highway is analyzed, because of the research question. However, to visually represent the differences between certain types of roads, a rural road will also be considered in the results.

For the materials (concrete), there also have been some assumptions, namely the Poisson number, linear expansion coefficient, and weight density of concrete. The variable that is able to change for the analysis is the concrete class, which will either be C28/35 or C35/45. For most analysis' the concrete class will be C35/45, however, in *Appendix C Extra figures model* a comparison has been given between the two classes.

For the foundation the example of the Beverentunnel has been followed, which is a subsoil of sand-grind, a foundation layer of concrete granulates, and an asphalt layer is directly below the concrete pavement. This foundation results in a bearing constant of 0,16 being used for these analyses.

The category of the input values is the temperature gradient, the standard temperature gradient (represented by several gradients and the frequency in which they occur) is given in Table 3. This standard distribution will initially be used; however, adjustments will be made to analyze the effects of the changing climate.

4.4.1 Traffic loads

First of all, a comparison has been made between AASHTO, which has been modelled in excel as well, and the model developed in this thesis. The standard input values were used to calculate the pavement thickness for several equivalent standard axle load repetitions. The results, shown in Figure 36, tell that the pavement thickness does increase quicker with the increasing axle load repetitions for AASHTO than for the model. Additionally, the model has a higher boundary limit, in other words, for lower axle load repetitions, it calculates a higher pavement thickness. The reasoning behind this effect will be explained in detail later in this section.

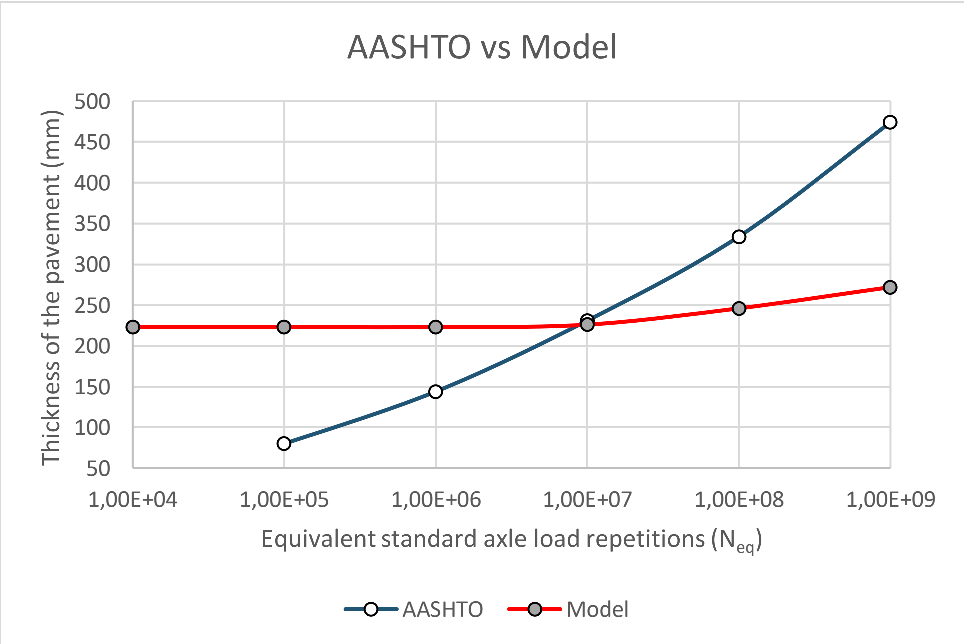


Figure 36 AASHTO vs Model for different N_{eq}

In the designed model, the same axle load frequency distributions have been used as in VENCON, which are listed in Table 2. To visually represent the difference in these percentages, two figures (Figure 37 and Figure 38) have been made comparing a rural road and a heavily loaded highway. In Figure 37 it can be seen that a rural road is used relatively more by vehicles with a lower wheel(/axle) load, where a heavily loaded highway sees more mid-high wheel(/axle) loads.

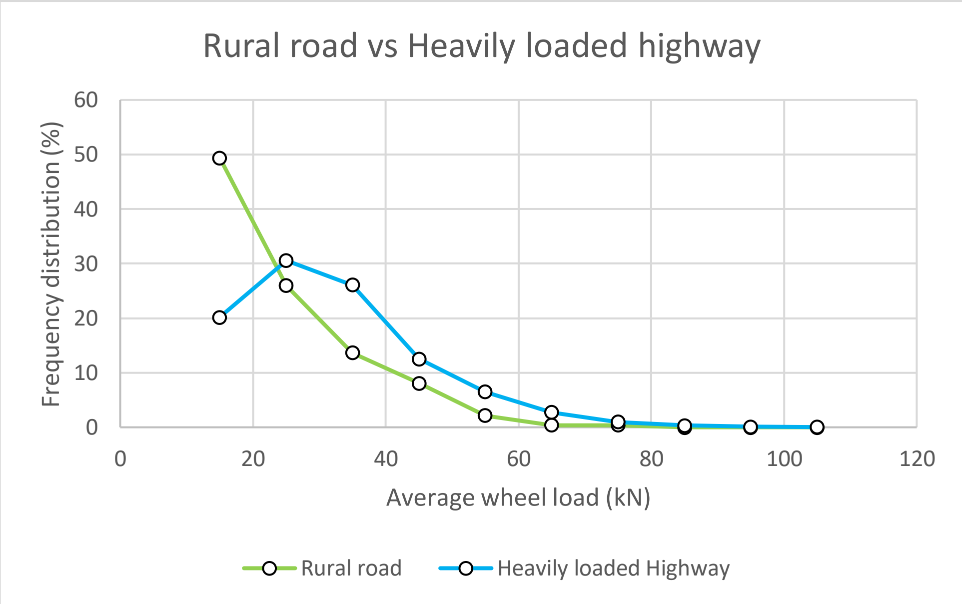


Figure 37 Frequency distribution: Rural road vs Heavily loaded highway

A second figure, Figure 38, was made showing what the differences on pavement thickness where when different axle load frequency distributions were used. The same, standard, inputs were used as with the comparison of AASHTO and the model, and the pavement thickness was calculated for several equivalent standard axle load repetitions. These thicknesses have been shown with the two continuous lines in Figure 38. Additionally, there is a condition given for the calculation of the allowable axle load repetitions in *3.2.2 VENCON 2.0* (page 32):

$$0{,}5 \leq \sigma_{max} / f_{brm} \leq 0{,}833$$

This equation gives a two boundary conditions for several combinations of the maximum tensile stress and the average bending tensile strength for a short-term load. This boundary conditions exists to eliminate combinations of the traffic and climate stresses that are smaller than half of the average bending tensile strength of the pavement, since these stresses would be insignificant for the pavement life. Additionally, if there would be combinations that would be bigger than 83,3% of the bending tensile strength, then the pavement will be significantly damaged, resulting in a lower lifespan. When there are combinations (of traffic and climate stresses) that are over 83,3% of the bending tensile strength, then the pavement thickness is supposed to be increased, until these combinations don't occur anymore. The end result of these boundary's is a minimum pavement thickness[11], which is represented by the dotted line in Figure 38. This minimum thickness has the most influence in situations with lower standard axle load repetitions, since these situations often result in a lower pavement thickness, as can be seen in Figure 38.

[11] The climate and traffic stresses aren't influenced by the equivalent standard axle load repetitions since they represent the stresses occurring in the pavement due to one specific wheel/axle load.

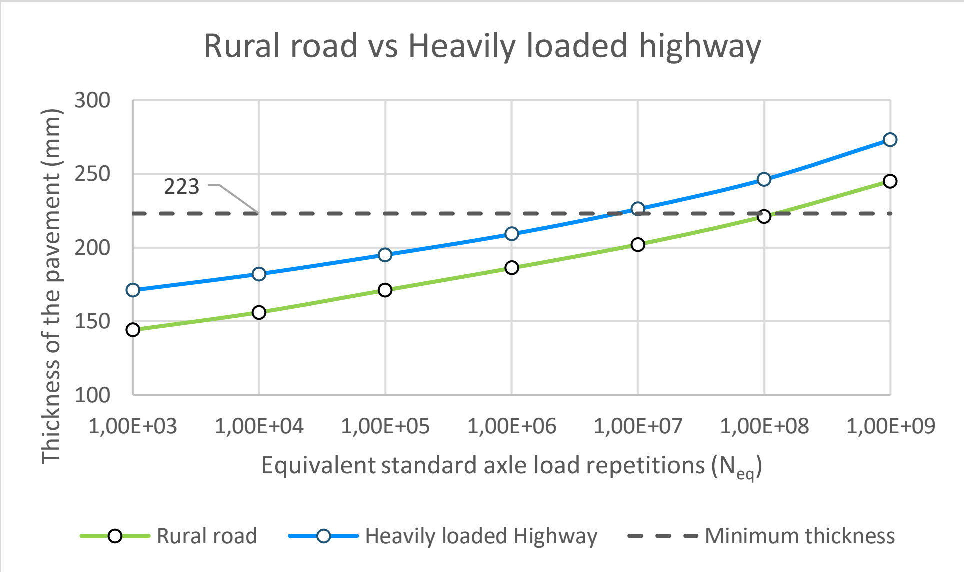


Figure 38 Pavement thickness: Rural road vs Heavily loaded highway

The effect of this boundary condition can also be represented as a result of a wheel load (traffic) and a temperature gradient (climate). In Figure 40 the value of the equation is given for several temperature gradients (°C/mm) and for several different wheel loads (kN). It can be seen that the equation is getting closer to the boundary the higher the temperature gradient is, and the higher the wheel load is, which is to be expected since σ_{max} is the sum of the traffic stress and the normative temperature stress. However, for $1*10^8$ equivalent standard axle load repetitions, the values don't exceed the boundary, and the pavement thickness thus doesn't need to be increased in order to accommodate the condition.

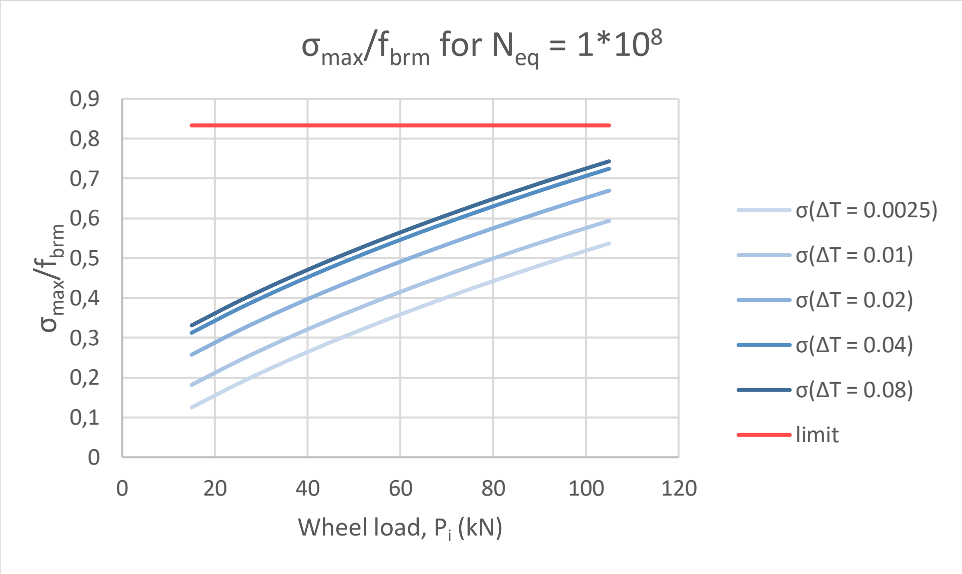


Figure 40 Boundary condition for $N_{eq} = 1*10^8$

However, the situation changes when a lower number of equivalent standard axle load repetitions is considered. For $1*10^6$ equivalent standard axle load repetitions, the boundary is exceeded when a combination occurs of a temperature gradient of 0.04 °C/mm or higher and a wheel load of (approximately) 92 kN and above. (Figure 39) This will result in the pavement thickness being increased, because of which the f_{brm} will increase as well, and thus the value of σ_{max}/f_{brm} will decrease.

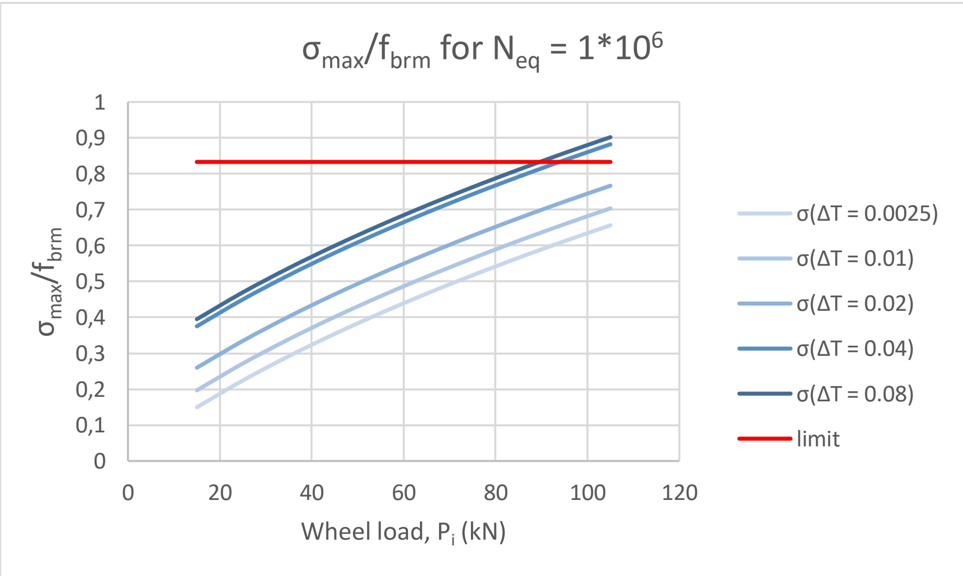


Figure 39 Boundary condition for $N_{eq} = 1*10^6$

Finally, an important factor in the calculation of pavement thickness with the model, is the location at the edge of the plate[12] that is considered. This position can be at the free edge, longitudinal joint, or transverse crack. This location is of in influence on both the traffic stresses and climate stresses, and the location at which the required pavement thickness is the highest, will be the norm for the entire pavement construction. To analyze the pavement thicknesses that result from the different locations, a comparison is made by the required pavement thickness at each of these locations for

[12] From a constructive point of view the weakest place of the pavement is always at the edge of the concrete plate and never within a concrete plate. (Houben, 2007)

several equivalent standard axle load repetitions in Figure 41. In this figure it can be seen that for the assumptions made in this thesis, the free edge will always be the norm for the pavement thickness.

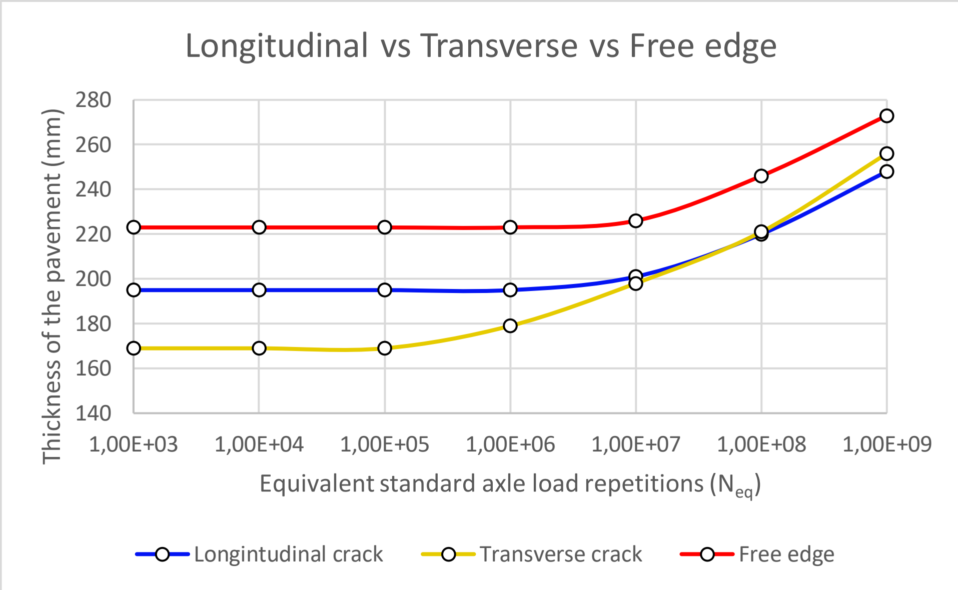


Figure 41 Longitudinal vs Transverse vs Free edge

4.4.2 Temperature gradients

Temperature gradients have also been analyzed in this thesis. The temperature gradient input consists of two parts; the gradients itself and their frequency distribution. The standard frequency distribution of the gradients, given in Table 3, has altered to see what the trends would be if the temperature gradient increased, or if higher temperature gradients would occur more often.

First of all, the frequency distribution was altered three times, resulting in the following distributions:

Temperature gradient (°C/mm)	**Standard distribution (%)**	**2nd frequency distribution (%)**	**3rd frequency distribution (%)**	**4th frequency distribution (%)**
0.0025	59	53	47	41
0.01	22	20	18	16
0.02	7.5	10.2	12.9	15.6
0.03	5.5	7.5	9.5	11.5
0.04	4.5	6.0	7.5	9.0
0.05	1.0	2.0	3.0	4.0
0.06	0.5	1.3	2.1	2.9

Table 18 Frequency distributions used for analysis

Pavement thicknesses where then calculated for every frequency distribution and for several equivalent standard axle load repetitions. The results are shown in Figure 42. A translation of the line can be seen for each frequency distribution for N_{eq} higher than $1*10^6$. Until an N_{eq} of $1*10^6$ all frequency distributions yield the same pavement thickness, because the boundary condition for the maximum tensile stress and bending tensile strength of the pavement is normative. Since the frequency distribution does not influence the bending tensile strength of the concrete (only pavement thickness and the concrete class do) or the maximum tensile stress, the minimum pavement thickness stays the same, regardless of the frequency distribution of the temperature gradients.

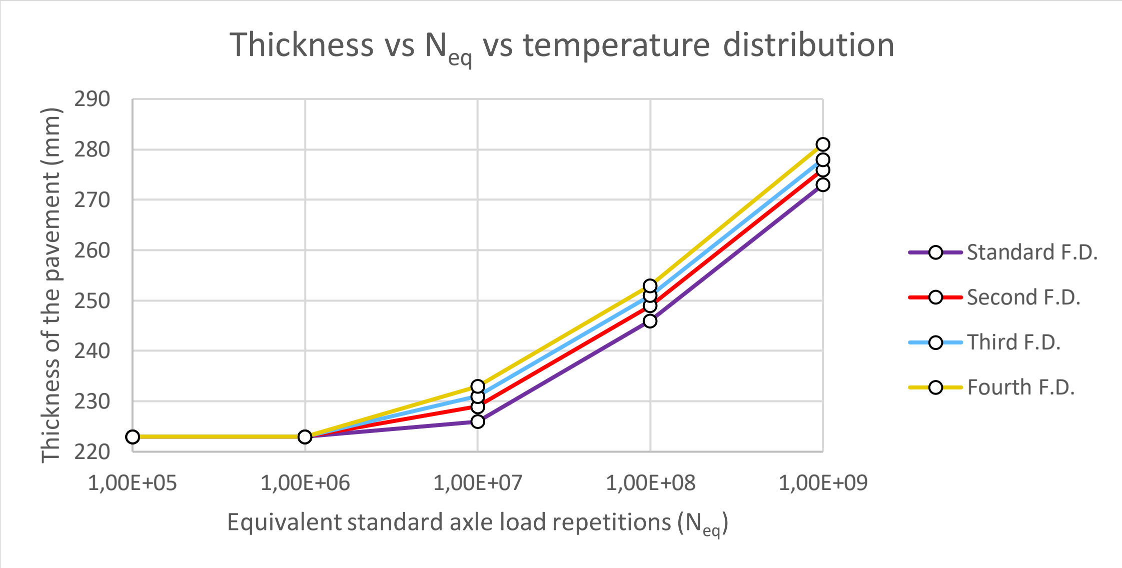


Figure 42 Thickness of the pavement for varying temperature gradient frequency distributions

After altering the frequency distribution, the temperature gradient itself was analyzed. Several (more extreme) temperature gradients were compared to the standard temperature gradient, much in the same way as it was done for the frequency distribution. For every single situation, the temperature gradient was made more extreme, and the standard frequency distribution was used. The following temperature gradients were used:

Frequency distribution (%)	**Standard temp. gradient (°C/mm)**	**2nd temperature gradient (°C/mm)**	**3rd temperature gradient (°C/mm)**	**4th temperature gradient (°C/mm)**
59	0.0025	0.0031	0.0039	0.0049
22	0.01	0.0125	0.0156	0.0195
7.5	0.02	0.0250	0.0313	0.0391
5.5	0.03	0.0375	0.0469	0.0586
4.5	0.04	0.0500	0.0625	0.0781
1.0	0.05	0.0625	0.0781	0.0977
0.5	0.06	0.0750	0.0938	0.1172

Table 19 Temperature gradients used for analysis

Pavement thicknesses where then calculated for every temperature gradient and for several equivalent standard axle load repetitions. The results are shown in **Error! Reference source not found.**. A translation of the entire line can be seen, including for $N_{eq} < 1*10^6$, in contrast to the previous figure. The cause for this is that the temperature gradient is altered, which means that the tensile stress caused by the temperature gradient has been altered as well, causing the equation (σ_{max}/f_{brm}) to change. In this case the temperature gradient has increased, causing for a higher tensile stress due to a temperature gradient, an increase of the maximum tensile stress, and thus an increase in the equation. This means that for $N_{eq} < 1*10^6$ the minimum pavement thickness increases with an increasing temperature gradient. For $N_{eq} > 1*10^6$ the pavement thickness increases as well due to the tensile stresses increasing.

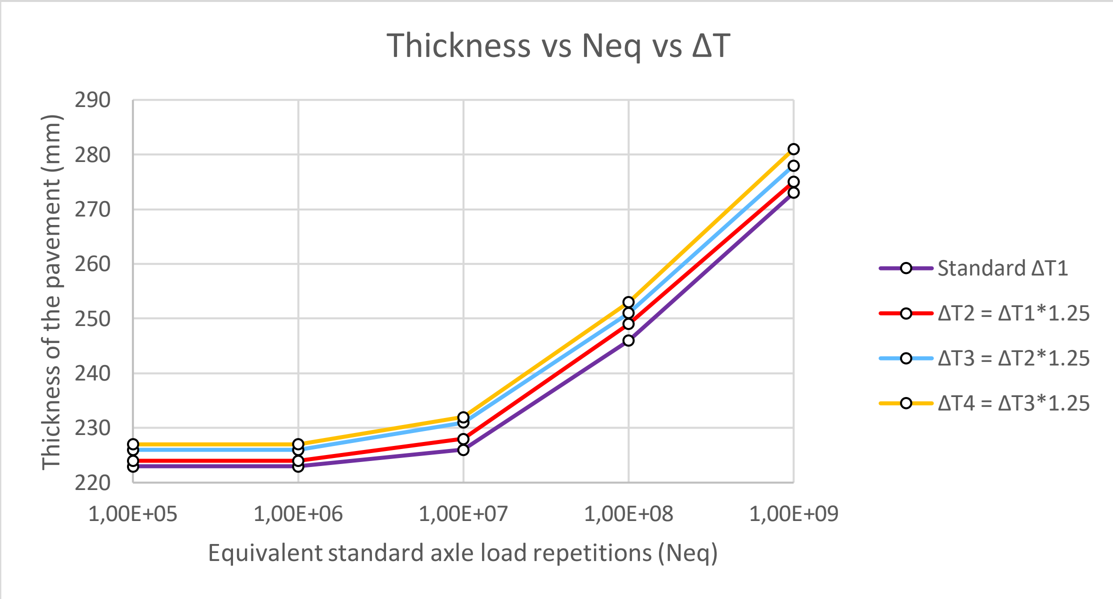


Figure 43 Thickness of the pavement for varying temperature gradients

To further explain the temperature gradients provided in Table 19, two figures (Figure 44 and Figure 45) are made for two temperature gradients (ΔT1 and ΔT4). In these figures, the temperature differences between the top of the pavement and the bottom of the pavement are represented for three different N_{eq} values.

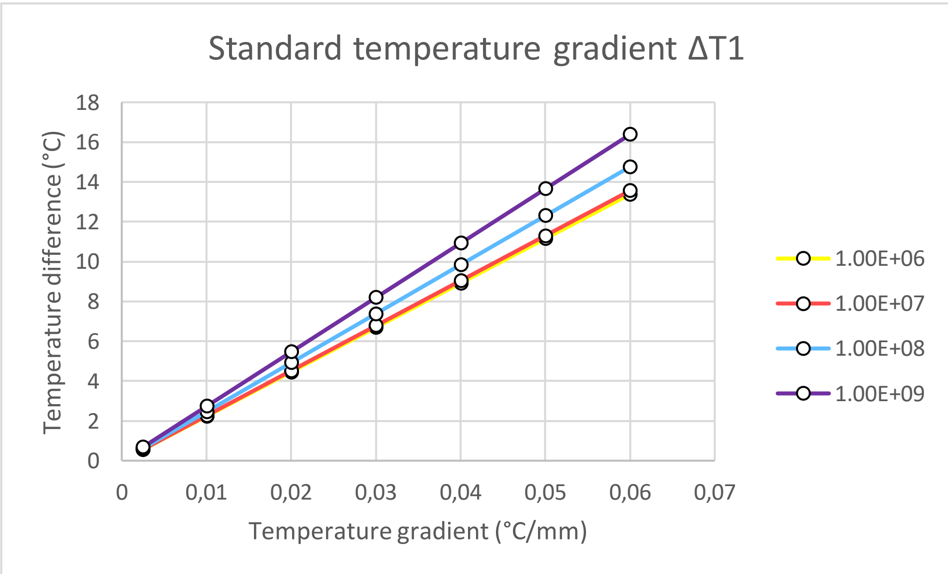


Figure 44 Temperature difference for standard temperature gradient ΔT1

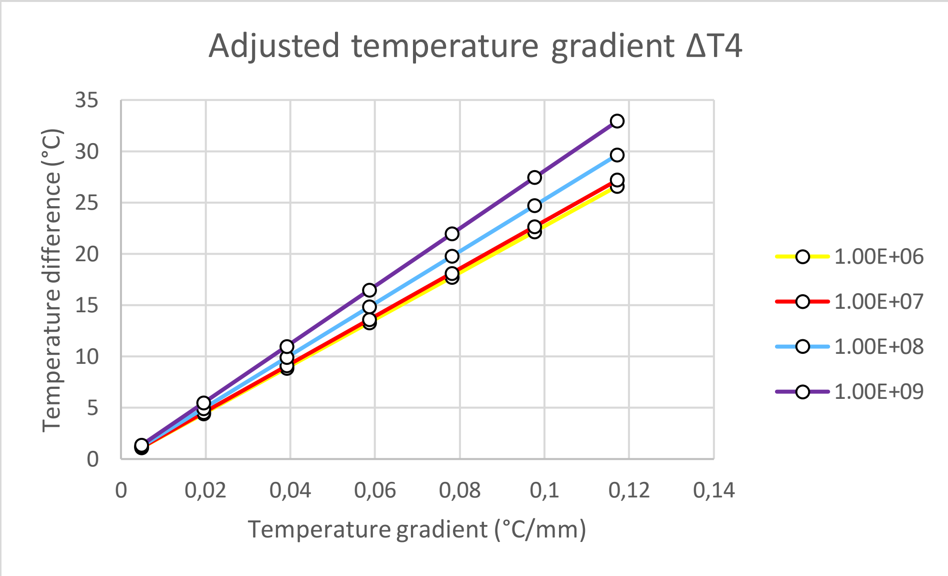


Figure 45 Temperature difference for adjusted temperature gradient ΔT4

Finally, a combination of both an increase of the temperature gradient and frequency distribution, Table 18 and Table 19, has been used as well. As expected, this results in a combination of the effects seen in Figure 42 and Figure 43, with bigger translations between the different settings used. These results have been given in Figure 46.

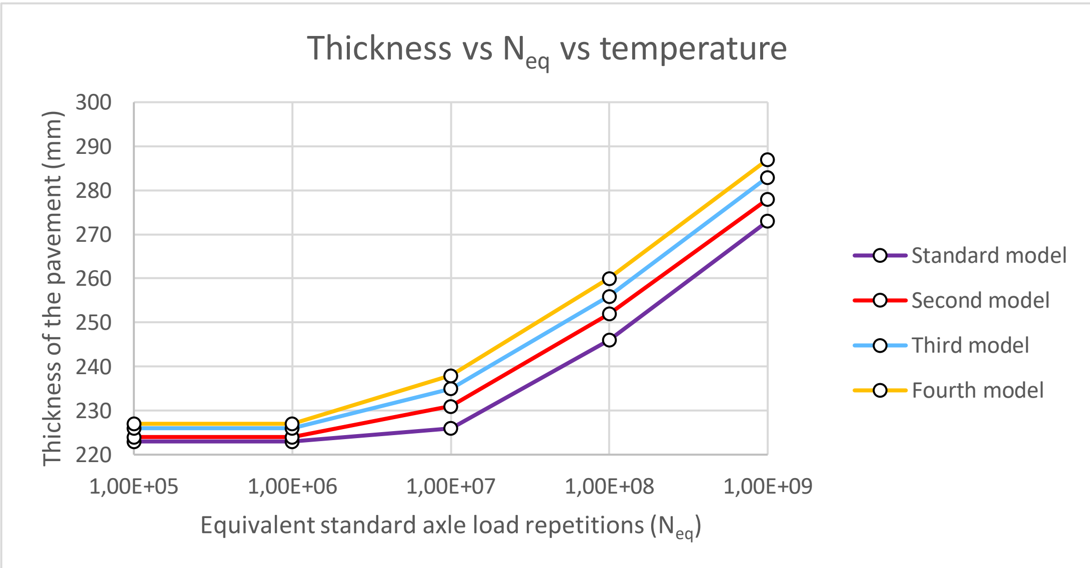


Figure 46 Thickness of the pavement for varying temperature gradients and frequency distributions

Finally, the prediction for the temperature fluctuation development over the years, made in *4.1 Climate*, is used to make a prediction about how pavement thickness will be influenced by the temperature gradient changing together with the temperature fluctuation in "de Bilt". The standard temperature gradient (provided in VENCON 2.0) is multiplied with factors specified in Table 11 for the temperature development after 30 years (2040-2050 in the table) and after 60 years (2070-2080). This was done for both situation 2 and situation 3. Using these temperature gradients, the pavement thickness is calculated and given in Table 20 and Table 21.

Situation 2 - current trend continues			
N_{eq}	year 0	year 30	year 60
1.00E+03	223	223	223
1.00E+04	223	223	223
1.00E+05	223	223	223
1.00E+06	223	223	223
1.00E+07	225	226	227
1.00E+08	245	246	246
1.00E+09	272	273	273

Table 20 Pavement thickness for adjusted temperature gradient – situation 2

Situation 3 - current trend accelerates			
N_{eq}	year 0	year 30	year 60
1.00E+03	223	223	223
1.00E+04	223	223	223
1.00E+05	223	223	223
1.00E+06	223	223	223
1.00E+07	225	227	227
1.00E+08	245	246	247
1.00E+09	272	273	273

Table 21 Pavement thickness for adjusted temperature gradient – situation 3

Additionally, the effective temperature difference between the top of the pavement and the bottom of the pavement is also calculated in Figure 47. This is done exclusively for situation 2, since situation 3 does not differentiate enough from situation 2 to yield any visible changes to the figure.

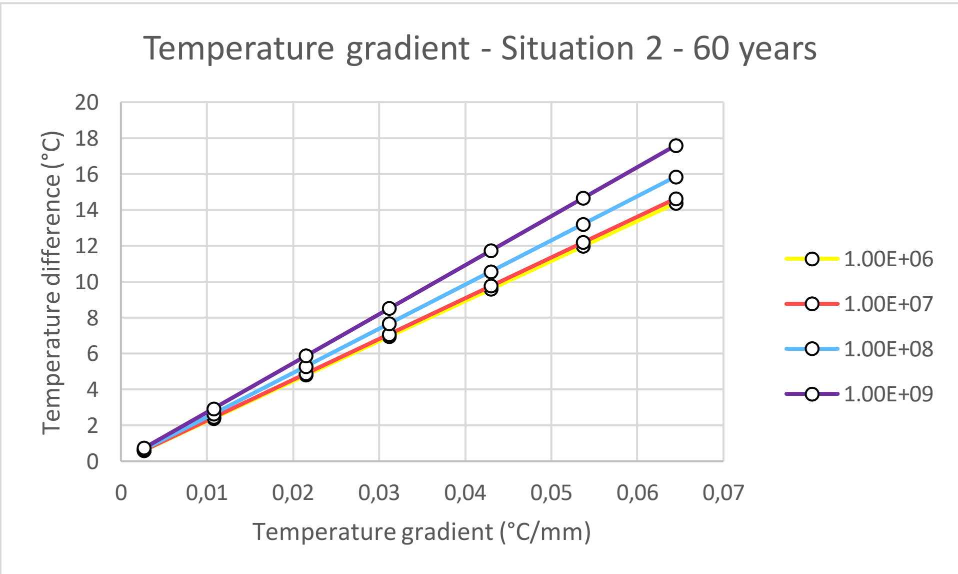


Figure 47 Temperature difference for situation 2 after 60 years

5. Case study

To validate the model developed during this thesis, the results for the pavement thickness are compared to the results from two separate cases, the project regarding the "Beverentunnel" and the case study provided by (Houben, 2007) in the background report for VENCON 2.0.

5.1 Beverentunnel

Phoenix Engineering was approached by Arcadis during the making of this thesis for the replacement of the concrete pavement in the "Beverentunnel", which is at the end of its lifetime. The "Beverentunnel" is located in the port of Antwerp and, due to that location, it is used primarily by heavy traffic. The tunnel crosses the "Waasland channel" and has a total length of around 1500 meters.

Figure 48 Entrance Beverentunnel (Febetra, 2019)

Arcadis had already designed a new concrete pavement using the VENCON 2.0 model, but needed a second opinion regarding this design. Phoenix Engineering gave this second opinion by proposing four new variants, which all had one aspect of the calculation changed. The first variant is a reduction of the traffic load on the edge of the pavement, which would prevent traffic from driving close to the edge. The second variant is changing the frequency distribution of the traffic wheel loads, since it was observed that more trucks were using the road than the "heavily loaded highway[13]" does take into account. The third variant adjusted the frequency distribution of the temperature gradients since it was assumed that the temperature gradients would be less severe inside a tunnel, due to the sun not being able to shine straight on the pavement. For the final variant, the number of trucks was adjusted based on the observations of the "Flemish traffic center" at the "Beverentunnel".

These variants together formed the proposed design by Phoenix Engineering. During a meeting between Arcadis and Phoenix Engineering, this proposed design was altered to just include the changed axle load frequency distribution and the changed frequency distribution of the temperature gradients. The second opinion itself is included in *Appendix D Second opinion Beverentunnel* and an example for the inputs (in the designed model) for the second opinion on the "Beverentunnel" is included in *Appendix E Input designed model for Beverentunnel.*

For this thesis, the output of the VENCON model, provided by Arcadis, was used to validate the output of the, during this thesis, designed model. The differences between the two models are presented in *Table 22*. The designed model gives pavement thicknesses that are on average 12-13 millimeter thicker than the VENCON model calculates. During this thesis the origin of this discrepancy could not be located.

Location	Plate 1		Plate 2		Plate 3	
	Model	VENCON 2.0	Model	VENCON 2.0	Model	VENCON 2.0
Transverse	175	167	226	224	183	196
Longitudinal	250	238	213	199	213	199
Free edge	250	238	-	-	250	238

Table 22 Validation with project Beverentunnel/VENCON 2.0

[13] This is one of the types of roads VENCON provides a standard frequency distribution for, see Table 2

5.2 Houben

In the background report about VENCON 2.0, made by (Houben, 2007), a case study is provided for a unreinforced concrete road. It concerns a 7,5 m wide, 2 lane provincial road, for which all the needed parameters are given. These parameters include: the properties of the foundation, load transfer on several locations in the pavement, the temperature gradient (standard gradient is used), and information about the traffic[14]. This information is used in the developed model, to check how closely the output matched the output given in the background report. In Table 23 the results of this validation are given, via the difference in pavement thickness (mm) between the model and the results that (Houben, 2007) provided. A positive value indicates a lower pavement thickness for the model, and a negative value indicates a higher pavement thickness for the model.

Concrete strength class	C28/35 (B35)						C35/45 (B45)					
Axle load freq. provincial road	Heavily loaded Provincial Road			Normally loaded Provincial Road			Heavily loaded Provincial Road			Normally loaded Provincial Road		
Number of heavy vehicles	10	100	1000	10	100	1000	10	100	1000	10	100	1000
Lifespan 20 yrs.	2	1	0	4	3	2	4	5	5	3	3	2
Lifespan 30 yrs.	2	0	-1	3	3	2	4	5	5	3	2	2
Lifespan 40 yrs.	1	0	-1	3	3	3	4	4	6	3	2	1

Table 23 Validation with background report

The results in Table 23 show that the model developed in this thesis calculates the pavement thickness almost exclusively within 5 mm of the calculated pavement thickness (Houben, 2007) provided.

6. Conclusions

This research report starts with a research question and seven sub questions in *1.7 Research question*, and throughout the report these sub questions have been (partially) answered. This chapter will go through the conclusions made for the topics discussed, and answers the research question itself.

The first sub question focuses on the expected temperature rise and expected temperature fluctuations in Western Europe. Weather trends in the Netherlands (de Bilt to be more precise) are used to answer this question. Using the data, retrieved from the KNMI, it is concluded that:

1) Temperatures are rising
2) Temperature fluctuations are increasing

The trend is analyzed in excel and described using three separate formulas (minimum/maximum temperature and temperature fluctuation). Using these formulas, three situations are considered;

1) the temperature will stay the same as in 2019,
2) the temperature will continue the established trends, and
3) the temperature trend will accelerate by a factor of 2.

This results in a prediction of the minimum/maximum temperature and temperature fluctuation for the next 60 years given in Table 9 and Table 10. Additionally, the increase of temperature variations is calculated, resulting in a factor that is used in this thesis, to adjust the standard temperature gradient used in the model. The factors representing the three situations are given in Table 11.

[14] Information about the traffic given is: traffic growth per year, days of usage per year, average number of axles for heavy traffic, tire types, and traffic distribution on key locations (on the transverse joints, exactly next to the longitudinal joints, exactly next to the free edge)

The second research topic concerns the different concrete pavements, and how continuously reinforced concrete pavements differentiates from them. These pavements are described in *2. Theoretical framework* and CRCP differentiates in; usage, managing forces, construction, and maintenance. First of all, CRCP is mainly used for highways, where riding comfort is important together with maintenance occurring as little as possible (especially for high intensity highways). The amount of maintenance needed for CRCP is significantly lower than the other types of rigid pavements due to the absence of joints. Secondly, CRCP also differentiates in managing forces due to the presence of reinforcement, which counteracts the tensile forces due to temperature and traffic loads, which are higher for CRCP than the other pavement types due to the length of the pavement. (a continuous pavement instead of a 4.5m plate or individual blocks) The reinforcement is used to control the width of the cracks, which is paramount to the lifespan of a CRCP. Finally, the construction is executed using machines (like a slipform paver) instead of by hand, additionally, more reinforcement is needed, and is placed differently, than with concrete reinforced slabs.

Three types of methods are analyzed for this thesis:

1) AASHTO 1993,
2) VENCON 2.0,
3) FLOOR 3.0

By doing an MCA (*4.2 Multi Criteria Analysis)* it is concluded that VENCON 2.0 is most applicable to calculating the pavement thickness for highways while taking into consideration stresses applied by temperature fluctuations. VENCON 2.0 and FLOOR 3.0 both use similar methods regarding traffic and temperature stresses, variables that AASHTO did not consider. However, FLOOR 3.0 is mostly focused on indoor constructions with static loads where VENCON 2.0 is primarily focused on traffic loads and their effect on the pavement. Additionally, FLOOR 3.0 also primarily focuses on unreinforced concrete plates, instead of CRCP, which VENCON 2.0 did take into account. VENCON 2.0 lacks the ability to sufficiently change material properties, which FLOOR 3.0 does allow its user to.

It isn't directly researched what is known of CRCP in other climates, however, it is possible to draw conclusions based on a study/model of the TU Delft, discussed in *2.4 Rigid pavements and climate*. Field experiments show that high temperatures during the day, after construction, accelerate the development of the tensile strength of a concrete pavement. When cracking does occur, the cracking width tends to be higher, due to the increased stiffness of the pavement and the higher stress needed to form cracks. Colder temperatures lead to a slower increase in tensile strength, which promotes cracking to occur.
Additionally, the field tests also show that bigger temperature drops overnight result in higher tensile stresses due to a temperature gradient, which promotes can promote cracking.
In August, these effects combine, leading to a stronger pavement (due to the high temperature during construction) and higher tensile stresses (due to a big drop in temperature overnight). In turn, this results in an increased crack width.
In colder temperatures, more cracking will occur, however, this does slow the tensile stresses increasing, leading to a smaller width of the cracks.

The variables needed to design continuously reinforced concrete pavements can be arranged in different categories:

- Traffic loads
- Traffic stresses
- Temperature stresses
- Materials
- Foundation

The traffic load is represented through equivalent standard axle load repetitions (N_{eq}). The N_{eq} represents how often a 100 kN axle load (traffic load) repeats itself during the entirety of the design lifetime of the pavement. This is calculated by taking into account several variables such as: the days of usage, traffic growth per year, the distribution of axle loads (dependent on the type of road), the average daily intensity, the positioning of the vehicles on the road, rutting, number of axles, design lifespan, etc. The number of *expected* axle load repetitions is then compared to the *allowable* number of axle load repetitions, calculated based on the stresses that the pavement is able to handle[15], the foundation of the pavement, and materials used. The number of *expected* axle load repetitions and *allowable* axle load repetitions are compared to each other using the Palmgren-Miner function. Finally, the reinforcement is calculated based upon the strain occurring on the topside of the pavement. The reinforcement serves to control the crack width in the construction, which should not exceed four millimeters. An overview of these calculations is given in Figure 29.

Temperatures are a factor, in the design of CRCP, due to the tensile stresses that occur because of it. These tensile stresses occur due to volume changes/temperature gradients in the concrete. The volume changes are either shrinkage (plastic/drying/autogenous/thermal shrinkage) or expansion due to temperatures increasing. Additionally, the moment of construction matters, since day-night fluctuations have been proven to be an important factor in the development of the cracking pattern, as was discussed in *2.4 Rigid pavements and climate*.
In the design, temperature gradients occurring from the top of the pavement to the bottom of the pavement are considered. In the designed model, several temperature gradients (which occur in different frequencies) are considered. These temperature gradients are used to calculate the tensile stresses occurring in the pavement. These tensile stresses are also dependent on other variables such as: pavement thickness, the linear thermal expansion coefficient, elasticity modulus, bearing constant, and dimensions of the plate. These tensile stresses are calculated for either a small temperature gradient or a big temperature gradient (for which the dimensions of the road change due to an arising curvature). The normative tensile stresses are finally used in the calculation of the allowable axle load repetitions.

The research question is:

What is the effect of increasing environmental temperatures and temperature variations in the next 60 years on the design of continuously reinforced concrete pavements used for constructing highways?

To answer this question, temperature gradients are analyzed, which consist of two parts
1) the temperature gradient
2) the frequency distribution.

Both of these factors have an impact on the pavement thickness. When adjusting the frequency distribution, the minimum thickness[16] does not change, however, when the calculated pavement thickness exceeds the minimum thickness, the pavement construction is 7-8 mm higher for the worst-case frequency distribution. This represents an average temperate gradient of 0,01654 °C/mm[17]. When adjusting the temperature gradient, the minimum thickness increases by 4 degrees in the worst-case scenario, and when the calculated pavement thickness exceeded the minimum

[15] Stresses are caused by a combination of wheel loads and temperature gradients
[16] Determined by dividing the maximum tensile stresses by the bending tensile strength, which cannot exceed 0,833.
[17] In case of $N_{eq} = 1*10^8$ → 253 mm * 0,01654 °C /mm = 4,18462 °C

thickness, the pavement height increased on average by 7-8 mm for the worst-case temperature gradient. This trend is shown in Figure 43.

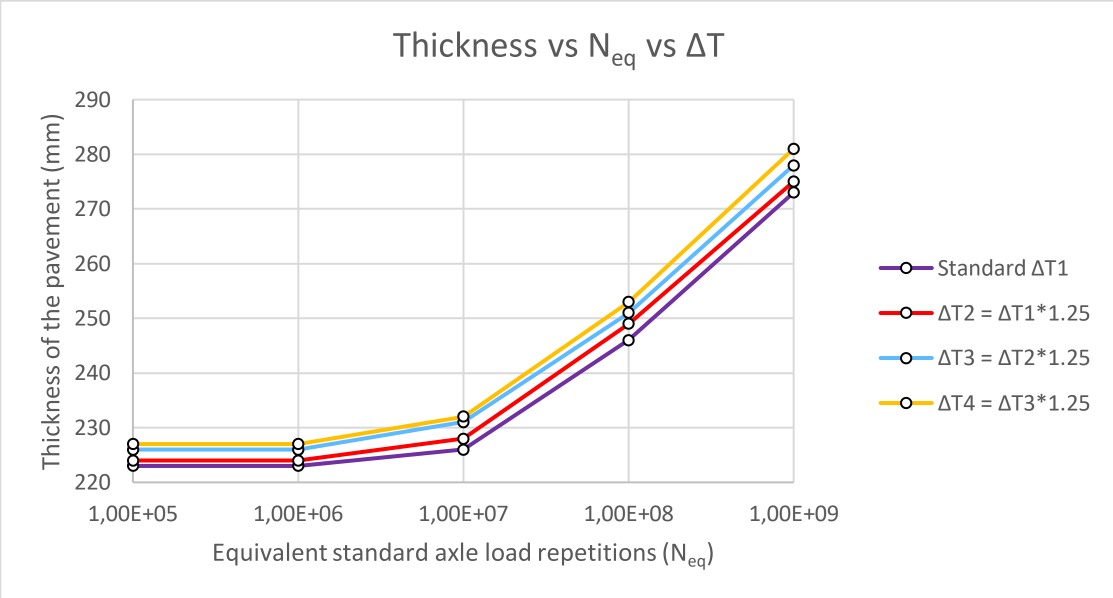


Figure 49 Thickness of the pavement for varying temperature gradients according to the developed model

Secondly, the standard temperature gradient can be multiplied by the same factor that the temperature variations are expected to increase by, according to the weather data gathered from the KNMI. These factors are calculated for both scenario 2 and 3, at 30 years from now and 60 years from now. As a result of these adjusted pavement gradients, the pavement thickness increases marginally, which is represented in Table 20 and Table 21.

Continuously reinforced concrete pavements are influenced by temperature gradients, however, according to the findings in this thesis, this does not significantly affect the designed pavement thickness. However, this conclusion depends on assumptions made during this thesis, which require further research to increase their probability of being realistic. This especially is the case due to temperature variations in theory being able to significantly affect continuously reinforced concrete pavements. Additionally, the month chosen to construct a continuously reinforced concrete pavement, can significantly affect the cracking pattern, and thus whether or not a pavement meets the requirements, such as the maximum crack width.

7. Discussion and recommendations

In this thesis there are several limitations which influenced the outcome of the research. In the discussion these limiting factors are discussed and recommendations are given for further research, to better understand/answer questions regarding the influence of temperatures (fluctuations) on concrete pavements.

Temperature gradient

The goal of the thesis is to analyze the influence of temperatures (fluctuations) on the design of concrete pavements, which is a relevant relationship because of the temperatures and temperature fluctuations increasing over the last 60 years. In this thesis the temperature trends at “de Bilt”, Netherlands, are described, however a concrete link between these temperature fluctuations and the temperature gradient in a concrete pavement has not been provided in this thesis. In this thesis adjustments have been made to show the development of pavement thickness as a result of both bigger temperature gradients in the pavement and an increased occurrence of the higher

temperature gradients in the standard temperature gradient (see Figure 42 and Figure 43). Additionally, the trend of the temperature fluctuations in "de Bilt" has been quantified into a factor, representing the growth of these fluctuations, which is used to modify the standard temperature gradient in the model. However, this last method relies on the standard temperature gradient being accurate in 2022.
Further research into the influence of temperature fluctuations on the temperature gradient is recommended together with research into which temperature gradient accurately represents the present.

Extreme combinations of traffic and temperatures
In this thesis a boundary condition is discussed, which adjusts the pavement thickness to certain extreme combinations of wheel loads and temperature gradients. This boundary condition is represented by the following equation:

$$0{,}5 \leq \sigma_{max} / f_{brm} \leq 0{,}833$$

A higher pavement thickness is need when this equation exceeds the limitation factor of 0,833. In practice, the calculated axle loads are a prediction of the traffic that will use the pavement during the entirety of its lifetime. However, it is not unlikely that there will be instances of extreme combinations occurring of temperature stresses and wheel loads that the pavement is not designed for. These loads will result in significant damage to the pavement construction, which the design tries to avoid. In this thesis, the instances of these extreme combinations occurring has not been explored.
Further analysis/experimentation is recommended, since these extreme combinations could lead to a significant reduction of the lifetime of a concrete pavement.

Reinforcement
The calculation of the reinforcement is considered during this thesis and is a factor in *4.2 Multi Criteria Analysis* where the winning variant (design method) is chosen. However, all methods lacked a way of describing the influence of the reinforcement on the pavement thickness. FLOOR 3.0 seems to take it into account in the program itself, but does not divulge any of the equations used in the background report. In VENCON 2.0 the pavement thickness is lowered by five millimeters when reinforcement is used, however, any argumentation for this action is missing.
In the background reports regarding FLOOR 3.0 and VENCON 2.0, there is information on the influence of temperatures on the quantity of reinforcement needed in the pavement. This information is not analyzed in this thesis, and it is likely that the temperature fluctuations[18] do influence the design of the reinforcement.
Further research into the relationship between reinforcement and the pavement thickness and most importantly the relationship between temperatures and reinforcement is recommended.

Input for pavement constructions
The calculation of pavement constructions is largely dependent on expectations, expectations of both the traffic loads and temperature loads are central in determining the design of a pavement structure. The accuracy of these predictions is often not verified and will affect the feasibility of the designed lifetime. For flexible pavements this is less of a problem than for rigid pavements since flexible pavements can deform because of their flexible nature. Rigid pavements, when exposed to higher traffic/temperature loads, can crack, resulting in a significant decrease in lifespan.
Additional research into both the accuracy of the predictions made for the input values and into the consequences of these input values being lower than what actually is occurring in practice is recommended.

[18] Temperature fluctuations cause tensile stresses in the pavement, for which (in CRCP) reinforcement is used to aid the concrete pavement, which

Case studies
In *5. Case study*, the developed model is validated using two cases, the “Beverentunnel” and the background report of VENCON 2.0. During the case study of the “Beverentunnel” it became clear that there is a discrepancy between the calculated thickness of the pavement of VENCON 2.0 and the developed model. This origin of this discrepancy was not located and, if found, would be something that would improve the developed model. Additional case studies would also benefit the accuracy of the model.
Further investigation into the discrepancy together with additional case studies would greatly benefit the accuracy of the developed model.

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9. Appendices

Appendix A Flow charts variants

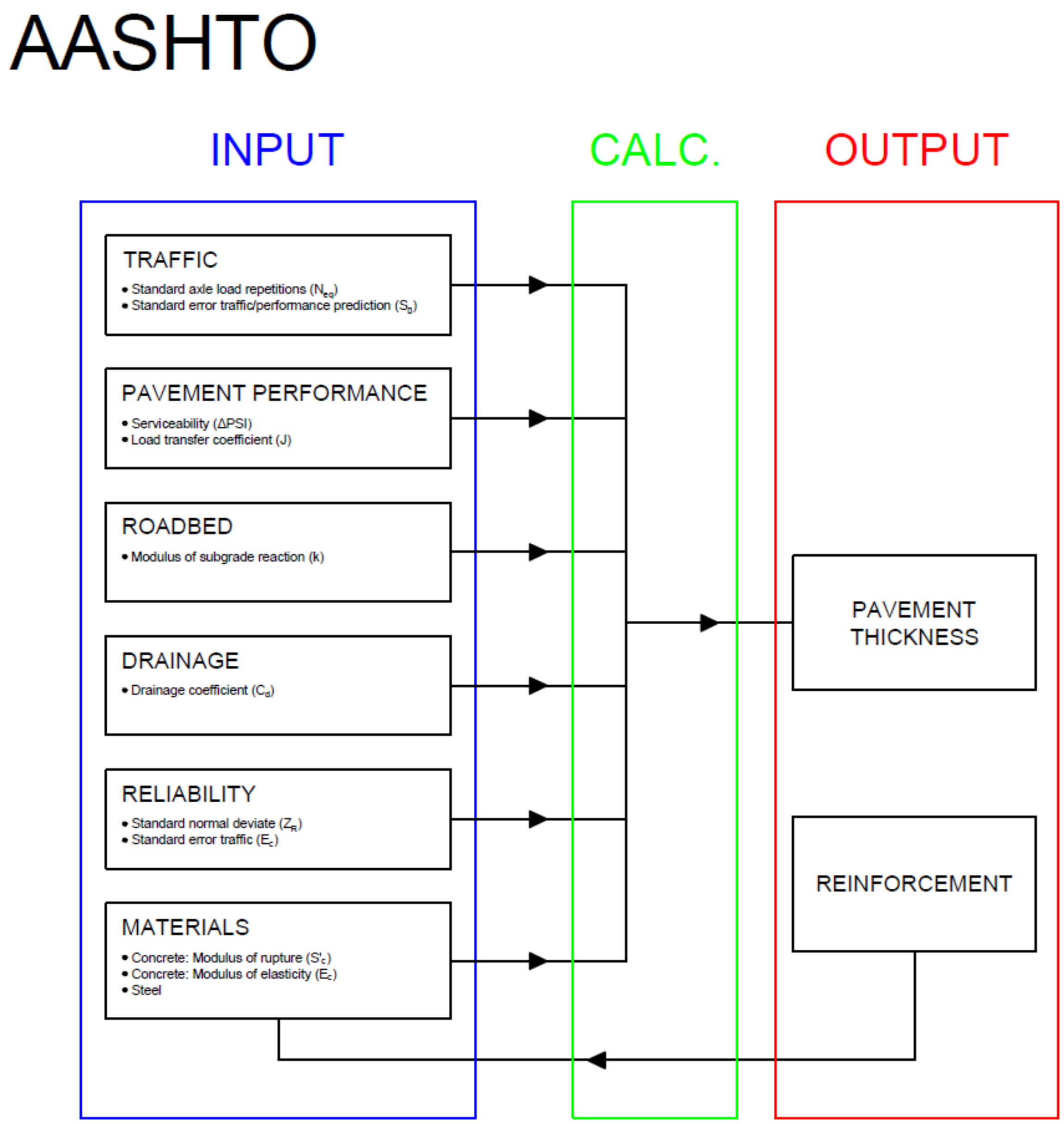

Figure 50 Enlarged flow chart of the AASHTO 1993 model

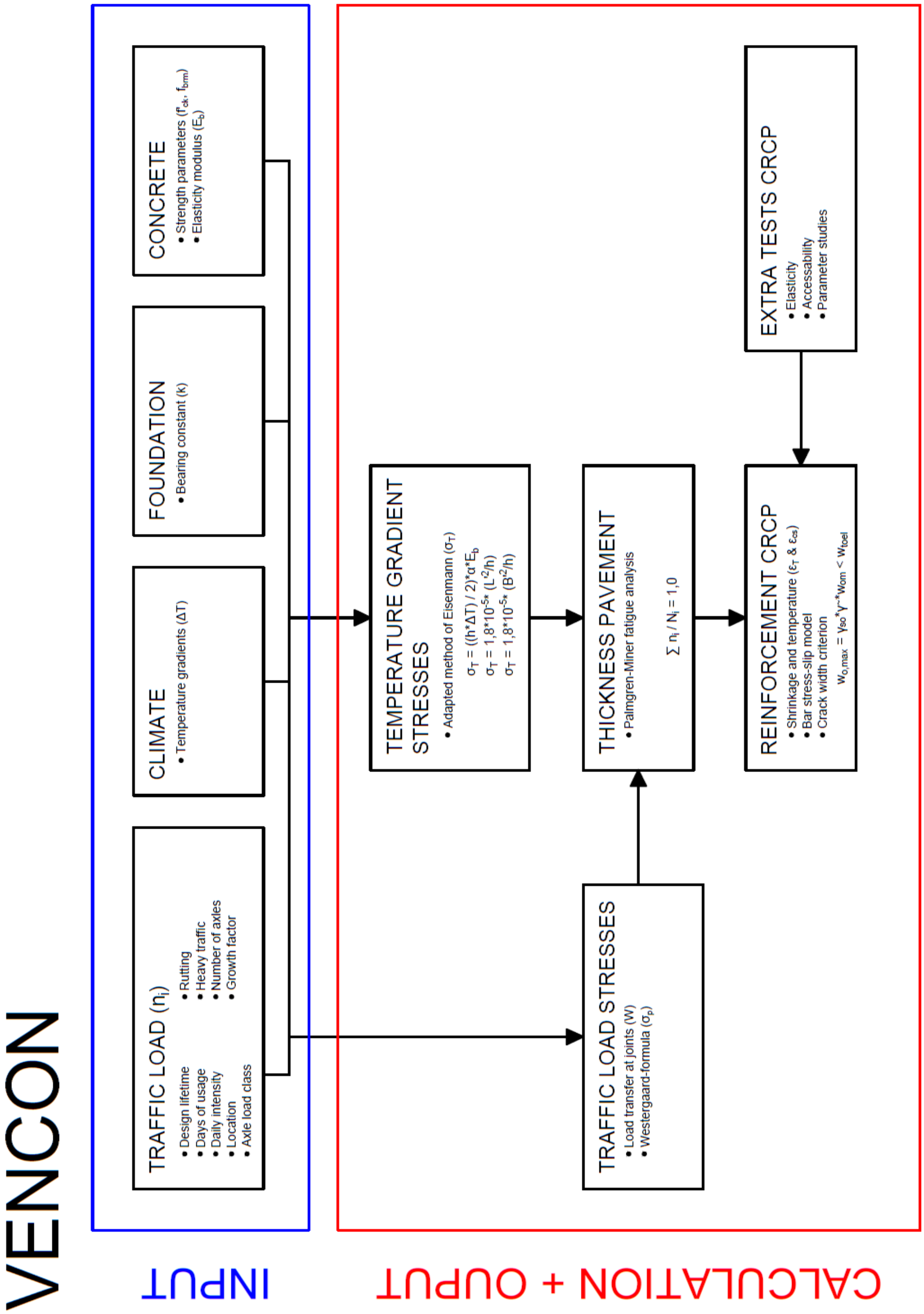

Figure 51 Enlarged flow chart of the VENCON 2 model

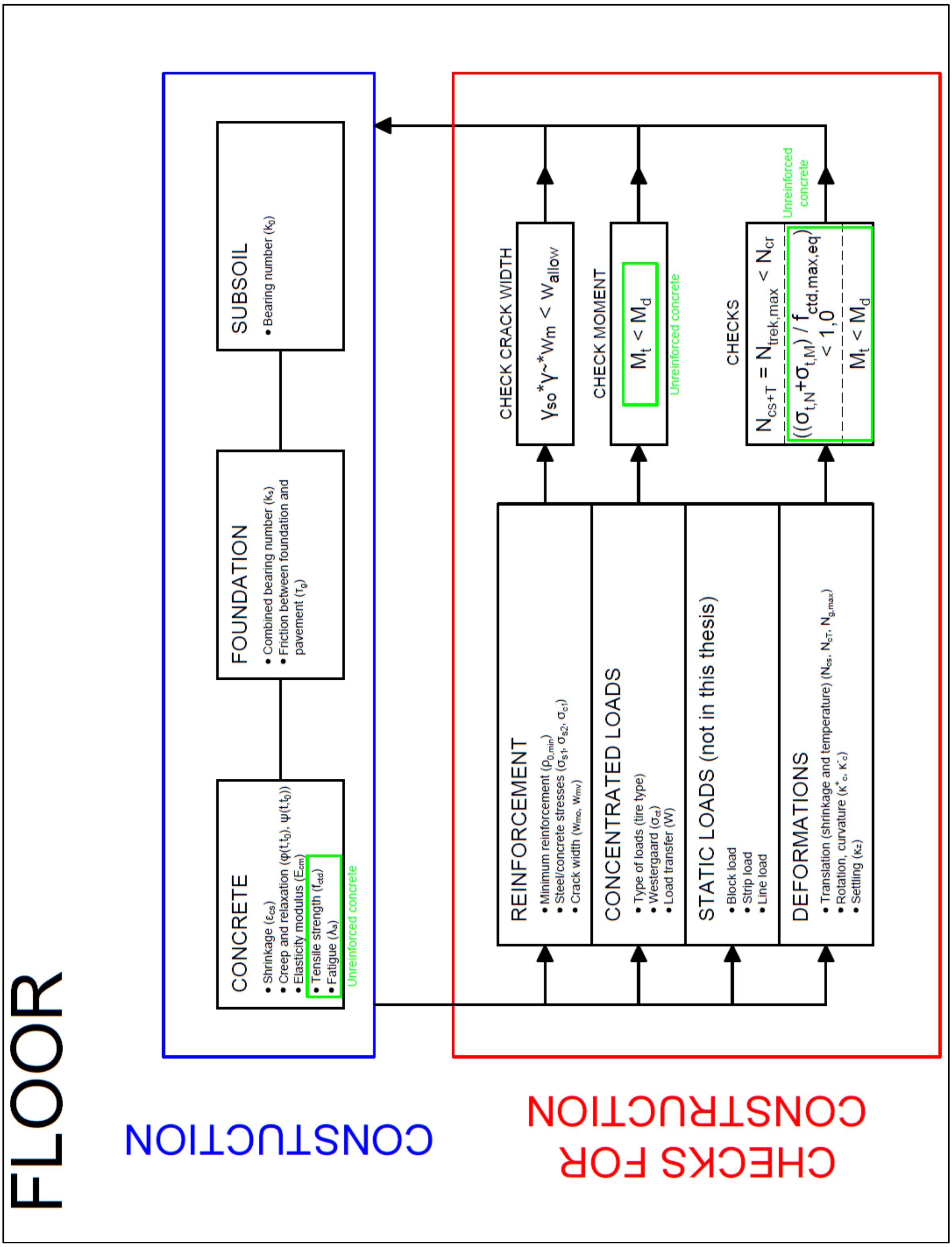

Figure 52 Enlarged flow chart of the FLOOR 3 model

Appendix B Model TU Delft

In *2.4 Rigid pavements and climate* there is an intermezzo regarding a model/study from the TU Delft regarding concrete pavements that were executed in different months. The pavements in August and November were highlighted due to their crack forming. The figures portraying tensile stresses and tensile stresses (Figure 18 and Figure 20) in that section were only the development of these tensile strengths and stresses when cracks occurred. However, there also was information about the tensile stresses if cracks did not occur, which are given in this appendix.

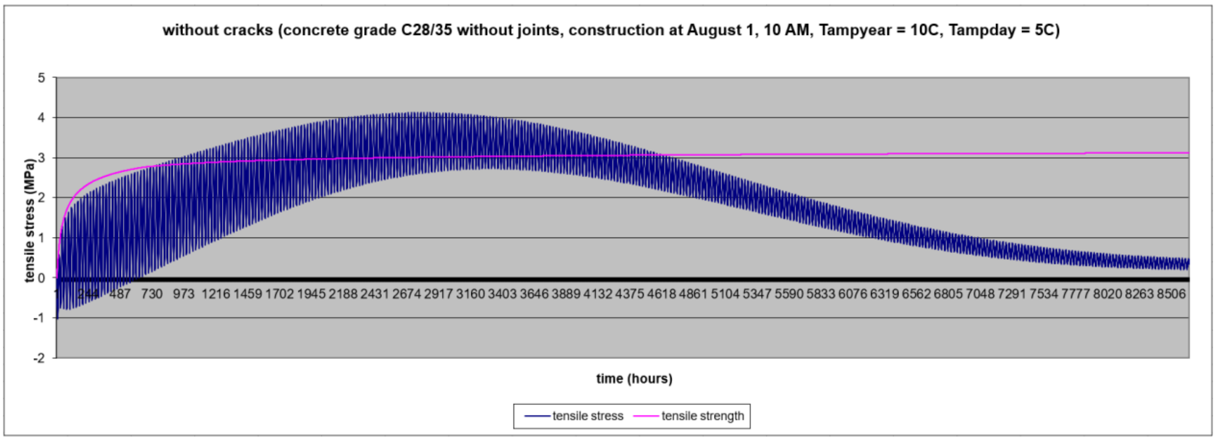

Figure 54 Tensile stresses and strength in August without cracks (TU Delft, 2007)

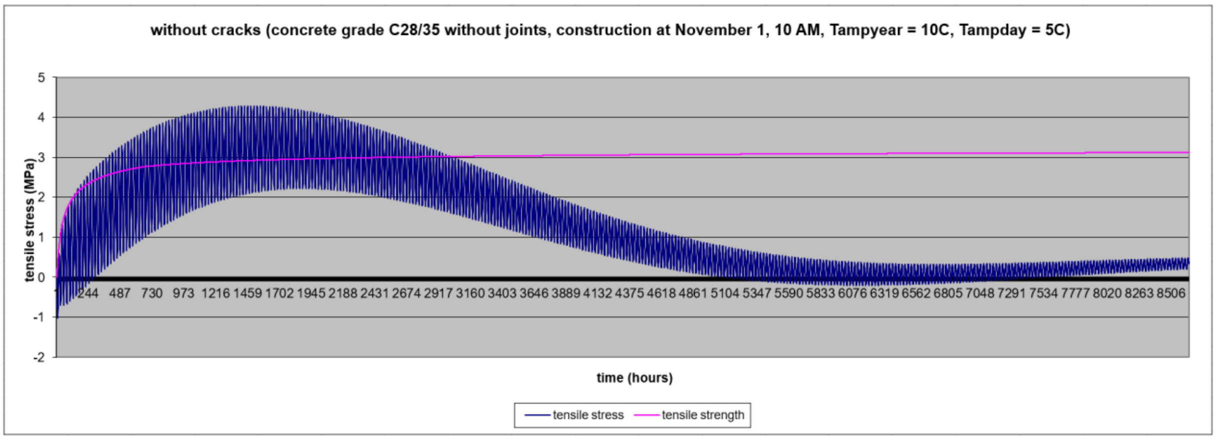

Figure 53 Tensile stresses and strength in November without cracks (TU Delft, 2007)

Appendix C Extra figures model

Not all figures that have been made using the developed model have been featured in the thesis. In this appendix two figures are given, the first one represents the differences in pavement thickness resulting from a change of concrete class. The second figure shows a comparison of the traffic stresses and the temperature stresses for different temperature gradients.

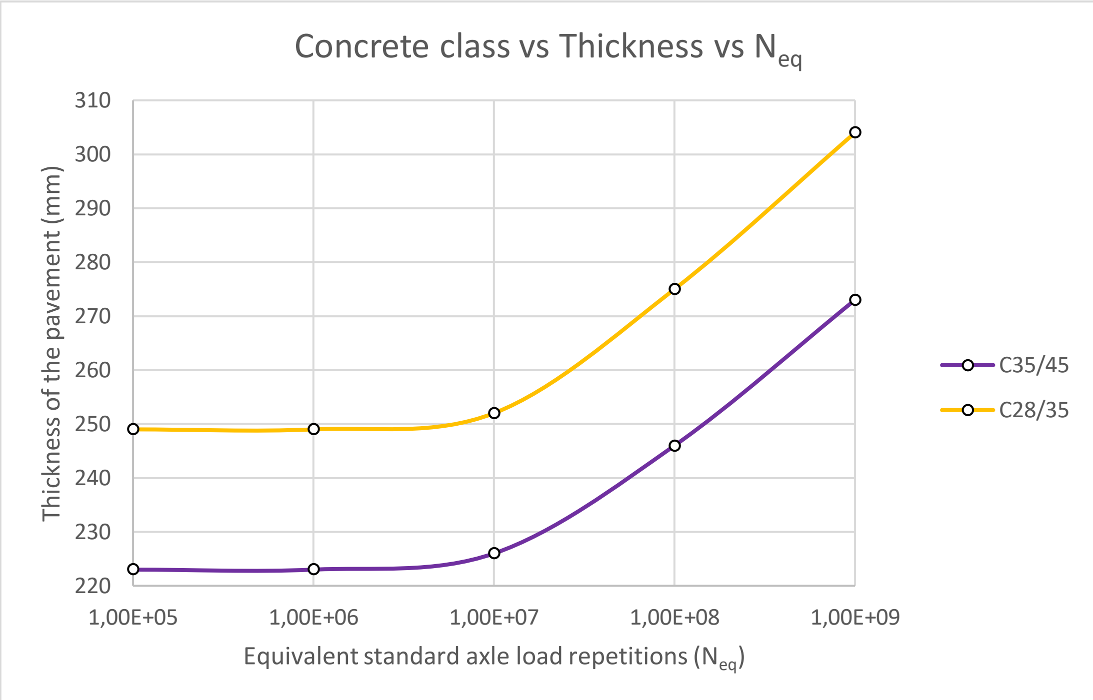

Figure 56 Influence of the concrete class on the thickness of the pavement

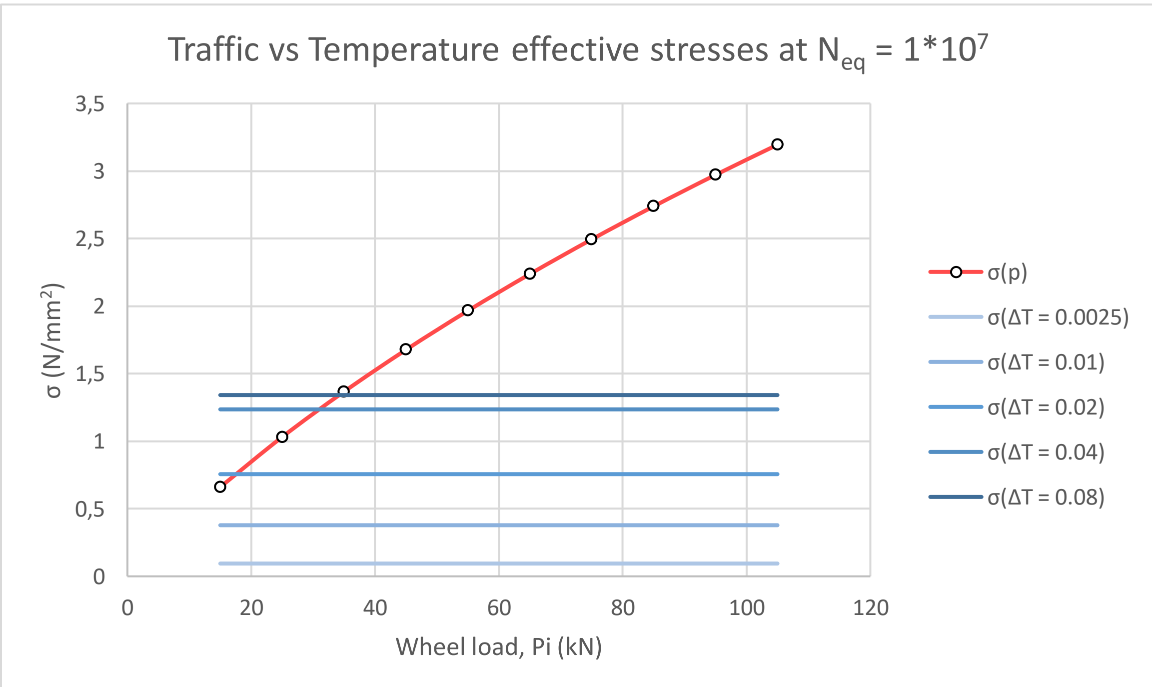

Figure 55 Traffic stresses compared to temperature stresses for $N_{eq} = 1*10^7$

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L.s,

Betreft: Second opinion rapportage Beverentunnel

1. Aanleiding

De aanleiding voor dit verslag is de vraag van Arcadis aan Phoenix Engineering voor een second opinion omtrent de vervanging van de cementbeton verharding in de Beverentunnel die aan het einde van zijn levensduur is. (1) Na vastlegging van de uiteindelijke constructie wordt een bestektekst conform de SS250 voorgesteld

2. Uitgangspunten

Bij het controleren van het ontwerp zijn behoudens 4 aanpassingen, dezelfde uitgangspunten aangehouden als vermeld in het rapport D10052682:21 d.d. 4 mart 2022 [1]. De uitgangspunten dewelke aangepast zijn, zijn:
1. de positie van de geleiderail.
2. de as verdeling.
3. de temperatuurgradiënten.
4. vrachtwagens per etmaal.

Het effect van de aangepaste varianten is beschreven in paragraaf 4, en de respectievelijke bijlagen.

1. Geleiderail plaatsen op de vluchtstrook

In het rapport [1] *1.8.2 B407 Geleideconstructie* werd gesteld dat “de locatie van de geleideconstructie en/of de barrier invloed heeft op de constructiedikte van de betonverharding.” Door het plaatsen van de geleideconstructie aan de rand van de constructie, op de vluchtstrook, rijdt minder verkeer net naast de vrije rand van de rijbaan. Dit effect leidt t.o.v. het huidige ontwerp tot 1 procent in plaats van de gebruikelijke 3 procent.

2. Aslastenspectrum aanpassen

Voor de aslastfrequentieverdeling is het type weg “ zwaar belaste autosnelweg” aangenomen in de uitgangspunten van het ontwerp. Echter rijden er significant meer vrachtwagens door de Beverentunnel dan dat dit type weg weergeeft. Procentueel gezien rijden minder assen van de aslastklassen 20-40 kN en 40-60 kN over de weg rijdt, en meer verkeer van de zwaardere aslastklassen. De verschillen tussen de aslastfrequentieverdeling van de “zwaar belaste auto-snelweg” en de aangepaste aslastfrequentieverdeling zijn in de tabel. weergeven:

Aslastklasse [kN]		Gemiddelde wiellast [kN]	Aslastverdeling	
begin	end		Zwaar belaste autosnelweg	Aangepaste verdeling
20	40	15	0.2016	0.1500
40	60	25	0.3056	0.2500
60	80	35	0.2606	0.3000
80	100	45	0.1254	0.1726
100	120	55	0.0651	0.0700
120	140	65	0.0271	0.0300
140	160	75	0.0100	0.0200
160	180	85	0.0031	0.0050
180	200	95	0.0012	0.0020
200	220	105	0.0003	0.0004

Table 24 Aangepaste aslastverdeling

3. Frequentie temperatuurgradienten aanpassen

In het ontwerp is het standaard temperatuurspectrum gebruikt, dit spectrum is afgeleid van de temperatuurgradiënt gemeten op de doorgaand gewapende betonverharding op de A12 (In Nederland, Tussen Den Haag en Arnhem) in 2000 en 2001. In een tunnel is de temperatuurgradiënt in een verharding echter kleiner, doordat o.a. de zon de verharding niet direct kan verwarmen. Om het klimaat in de tunnel beter te representeren is gekozen voor een aangepaste frequentieverdeling, weergeven in de tabel.

Temperatuurgradiënt klasse [°C/mm]		Gemiddelde temperatuur-gradiënt ΔT [°C/mm]	Frequentieverdeling [%]	Aangepaste verdeling [%]
0	0.005	0.0025	59.0	65.0
0.005	0.015	0.01	22.0	28.0
0.015	0.025	0.02	7.5	5.0
0.025	0.035	0.03	5.5	2.0
0.035	0.045	0.04	4.5	0.0
0.045	0.055	0.05	1.0	0.0
0.055	0.065	0.06	0.5	0.0

Table 25 Aangepaste frequentie verdeling temperatuurgradiënten

4. Vrachtwagens per etmaal aanpassen

In de uitgangspunten, afkomstig van AWV, is aangenomen dat het aantal voertuigen is 4360 (vtg/etmaal: vtg = vrachtwagens) voor de Noord-Zuid richting en 4772 (vtg/etmaal) voor de Zuid-Noord richting. Echter volgens de gegevens van het Vlaams Verkeerscentrum van de Vlaamse overheid m.b.t. de Beverentunnel in het afgelopen jaar is het jaargemiddelde van 5529 (vtg/etmaal) voor de Noord-Zuid richting en 5914 (vtg/etmaal) voor de Zuid-Noord richting. In bijlage B zijn de maandelijkse gegevens voor beide richtingen weergeven.

3. Methodologie

Voor het maken van de berekeningen omtrent de aangepaste uitgangspunten is een eigen model gebruikt. Dit model is gemaakt met als doel het analyseren van de invloed van klimaatveranderingen en materiaaleigenschappen op de dikte van cementbeton verhardingen. Voor het maken van het model is VENCON 2.0 2 gebruikt als basis, de uitkomsten van het (ontworpen) model vallen gemiddeld iets hoger uit dan de uitkomsten die volgen uit VENCON 2.0 2. Ter vergelijking is in bijlage C de maatgevende uitvoer gegeven van het model bij de oorspronkelijke uitgangspunten.

Voor de (aangepaste) uitgangspunten is de verhardingsdikte bepaald. De verkorte (maatgevende) invoer/uitvoer is weergeven bij de paragraaf 4, de resultaten.

4. Resultaten

Oorspronkelijke situatie volgens eigen model.

OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 250 mm + 10 mm
Ontwerpdikte	250	mm
Besteksdikte = 238 + 10 mm	260	mm
Samengesteld beddinggetal	0.160	N/mm3

Variant 8: Geleiderail plaatsen op de vluchtstrook

OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 240 mm + 10 mm
Ontwerpdikte	240	mm
Besteksdikte = 238 + 10 mm	250	mm
Samengesteld beddinggetal	0.16	N/mm3

Variant 9: Aslastenspectrum aanpassen

OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 254 mm + 10 mm
Ontwerpdikte	254	mm
Besteksdikte = 238 + 10 mm	264	mm
Samengesteld beddinggetal	0.160	N/mm3

Variant 10: Frequentie temperatuurgradienten aanpassen

OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 239 mm + 10 mm
Ontwerpdikte	239	mm
Besteksdikte = 238 + 10 mm	249	mm
Samengesteld beddinggetal	0.160	N/mm3

Variant 11: Vrachtwagens per etmaal aanpassen
OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 252 mm + 10 mm
Ontwerpdikte	252	mm
Besteksdikte = 238 + 10 mm	262	mm
Samengesteld beddinggetal	0.160	N/mm3

Variant 12: Combinatie van alle varianten
OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 235 mm + 10 mm
Ontwerpdikte	235	mm
Besteksdikte = 238 + 10 mm	245	mm
Samengesteld beddinggetal	0.160	N/mm3

Variant 12: Combinatie variant 9 en 10
OVERZICHT REKENRESULTATEN

Maatgevende plaat/strook	Besteksdikte	= 243 mm + 10
Ontwerpdikte	243	mm
Besteksdikte = 238 + 10 mm	253	mm
Samengesteld beddinggetal	0.160	N/mm3

5. Eisen standaard bestek 250

In het standard bestek SB250 versie d.d. juni 2021 in hoofdstuk 2 paragraaf is bepaald dat de verkeersklasse van deze weg een B1 type weg is. In hoofdstuk 6 van de SB 250 is een maximale dikte van 250 mm vermeld, echter op basis van onze berekeningen raden we een bestek dikte van 253 mm aan. Overige richtlijnen van het SB250 zijn verder van toepassing.

6. Conclusie en aanbevelingen

Conclusie

Op basis van een aantal gewijzigde uitgangspunten, met name de aanpassing van de positie van geleiderail constructie (gunstiger), temperatuurgradiënten (gunstiger) en aslastconfiguratie (ongunstig) is de conclusie dat de dikte van 250 mm voldoende is. Uit het rapport [1] is voldoende vrije ruimte met deze betondikte van 250 mm (idem als oorspronkelijke voorstel).

In de langsvoegen zijn deuvels (W=60%, W =overdracht van belasting bij de voeg) gebruikt, echter genieten ankerstaven (W=50%, minder gunstig voor betondikte) de voorkeur. Zo blijft de naad tussen de rechter- en linkerrijstrook dezelfde. Voor de dikte is de vrije rand immers maatgevend (ook bij 1%).

Voor het ballastbed stellen we voor om deze alsnog aan te passen, immers bij het slopen van de oude verharding en het afvoeren van het betonpuin kan er stof in het ballastbed terecht komen, dit kan de drainagefunctie van het ballastbed verminderen. De conditie van het ballastbed moet gecontroleerd worden na het transporteren van het puin en mogelijk toch vervangen worden om de goede drainerende werking van het ballastbed te garanderen.

Appendix E Input designed model for Beverentunnel

INPUT MODEL

ROAD LAYOUT:

Road type	: Lane tunnel
Pavement type	: Unreinforced concrete
Concrete strength class	: C35/45
Bonus on pavement thickness	: 10 mm
Distribution per lane (right to left)	
- Emergency lane	: 3.00 m
- Lane 1	: 3.65 m
- Lane 2	: 3.65 m
- Lane to the left	: 0.90 m
Width pavement	: 11.20 m

LONGITUDINAL JOINTS:

Joint 1, lane/emergency lane, position	: 2.65 m
- shrinkage joint with dowels, transfer	: 80.0 %
Joint 2, lane/lane, position	: 6.75 m
- shrinkage joint with dowels, transfer	: 80.0 %

TRANSVERSE JOINTS:

Spacing transverse joints	: 4.50 m
Type of joint	: shrinkage joint with dowels

SUBSOIL:

Subsoil	: Sand-gravel
- Bearing number subsoil	: 0.147 N/mm^3
- Elasticity modulus subsoil	: 500 N/mm^2
Foundation layer	: Concrete granulates 0 / 40
- Thickness foundation layer	: 250 mm
- Elasticity modulus foundation layer	: 800 N/mm^2
- Foundation layer bound	: no
Sort bituminous interlayer	: Asphalt
- Thickness bituminous interlayer	: 50 mm
- Elasticity modulus interlayer	: 7500 N/mm^2

FICTITIOUS TRANSFER FOUNDATION:

Transfer lane right	: 35.0 %
Transfer lane left	: 35.0 %

Traffic load:

Intensity traffic:	
- Design lifespan	: 30.0 year
- Number of usage days per year	: 270
- Average daily intensity	: 4772 vehicles
- Percentage traffic on one lane	: 95.0 %
- Heavy traffic according to spectrum	: 100.0 %
- Average number of axles per vehicle	: 4.00
- Traffic growth per year	: 2.00 %
Number of heavy vehicles per lane	: 4533.40 number/day

DISTRIBUTION TRAFFIC OVER LANES:

Traffic over plate 1	: 1.0 %
- rutting factor	: 100.0 %
Traffic over plate 2	: 95.0 %
- rutting factor	: 92.0 %
Traffic over plate 3	: 4.0 %
- rutting factor	: 92.0 %

TRAFFIC OVER LANGITUDINAL JOINT:

Traffic over edge right side	: 3.0 %
Traffic over longitudinal joint 1	: 10.0 %
Traffic over longitudinal joint 2	: 10.0 %
Traffic over edge left side	: 3.0 %

TEMPERATURE SPECTRUM:

Nr.	Gradient class [K/mm]	ΔT	perc. %
1	0.000 - 0.005	0.003	59.00
2	0.005 - 0.015	0.010	22.00
3	0.015 - 0.025	0.020	7.50
4	0.025 - 0.035	0.030	5.50
5	0.035 - 0.045	0.040	4.50
6	0.045 - 0.055	0.050	1.00
7	0.055 - 0.065	0.060	0.50

AXLE LOAD SPECTRUM, HIGHWAY HEAVY LOADED:

Nr.	Axle load class [kN]	Av. Axle load [kN]	Perc. %
1	20-40	30.0	20.16
2	40-60	50.0	30.56
3	60-80	70.0	26.06
4	80-100	90.0	12.54
5	100-120	110.0	6.51
6	120-140	130.0	2.71
7	140-160	150.0	1.00
8	160-180	170.0	0.31
9	180-200	190.0	0.12
10	200-220	210.0	0.03

TIRE SPECTRUM, AWV FLANDERS:

Nr.	Code	Description band	Perc.
1	WT	Wide tire	50.00
2	DA	Double air	25.00
3	SA	Single air	25.00