Bicycle friendly junction designs: a comparison between two junction types

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Summary

A bicycle friendly junction differs from regular junctions because it provides shorter average waiting times and high traffic safety for bicycles. The Dutch government has set itself the goal of increasing bicycle usage and reducing the number of traffic casualties and wounded.

In urban areas the largest share of delays, for any transport mode, occurs at junctions. A decrease in travel time makes cycling more attractive than other modes and can therefore help to increase the level of cycling in a city. Bicyclists make up more than half of all yearly traffic injuries. Out of the total number of traffic fatalities and injuries amongst bicyclists about two third happen at a junction. So increasing bicycle traffic safety at junctions is a vital step in achieving the set traffic safety goals. Because of their characteristics implementing bicycle friendly junctions helps to promote cycling and reduce the amount of traffic fatalities and injuries.

Two types of bicycle friendly junction designs are: the roundabout with priority for bicycles and the Simultaneous Green For Cyclists (SGFC) signalised junction. These two junction types can be used in roughly the same situations because both have a recommended maximum daily intensity of 25.000 Passenger Car Equivalents (PCE) per day. Road authorities therefore have to choose between these two design types.

To help road authorities decide which junction type best fits their needs several studies have compared the performance of roundabouts compared to signalised junctions. In addition guidelines are set up to guide road authorities in the decision making. There are however several shortcomings in the current knowledge. First of all many studies do not take bicycles into account. Given that in the Netherlands about 25% of all trips is done by bicycle, in urban areas this share is usually higher, ignoring cycling may not give an accurate representation of the Dutch situation. Secondly some guidelines only give qualitative instead of quantitative recommendations. A quantitative threshold would better inform road authorities and reduce the need for studies for each individual location. Finally the SGFC-junction isn't as widely used as other signalised junctions and therefore also isn't often included in scientific studies.

This leads to the following main research question:

"How do the roundabout with bicycle priority and the SGFC-junction perform over different traffic demand scenarios?"

There are several different ways in which the performance of a junction can be specified. Traditionally the junction performance is measured by its capacity and the resulting delays that road users endure. However since bicycle friendly junctions can be constructed in order to increase traffic safety it is also important to assess the junction performance for this indicator. Finally the junctions are assessed on their environmental impact as well, because one of the reasons to promote cycling is that it is a very environmentally friendly mode of transport. Because bicycle friendly junctions provide low waiting times for bicycles it is likely that these junctions lead to higher travel times for motorised traffic. This leads to more congestion for motor vehicles and thus higher emissions which could negate the initial positive environmental impact of increased cycling. It is therefore important to look for the bicycle friendly junction type with the lowest amount of motor vehicle emissions. This leads to the following three sub questions:

"What is the difference between the throughput occurring at the junction designs?"

"What is the difference in traffic safety between the junction designs?"

"What is the difference in environmental impact between the junction designs?

So the two junction types are evaluated on three criteria: throughput, traffic safety and environmental impact. To assess the throughput the indicator average travel time in seconds, specified for motor vehicles and bicycles, is used. Traffic safety is expressed as the number of conflicts, specified in conflicts involving: only motor vehicles, only bicycles and a bicycle and a motor vehicle. A conflict occurs when the Time To Collision (TTC), how long it would take for two vehicles to collide given their location and current speeds, is below a value of 1.5 seconds. To assess the environmental impact of the junction types three types of motor vehicle emissions are used: Total Carbon (TC) emissions, NO_x emissions and PM_{10} emissions.

By answering the research questions it will be known how each of the bicycle friendly junction types performs on the three types of performance criteria over a range of traffic demand scenarios. This research probably will not be able to categorically state which junction design is the best because that will likely depend on the relative weights that a road authority places on each of the three performance criteria. For instance what level of congestion increase is acceptable given a certain increase in traffic safety. However with the information about the junction performances road authorities will be able to better quantify these trade-offs and therefore make better informed decisions.

There is no dataset available with the performances of these two junction types. Therefore a simulation model is used to generate the necessary data. A micro-simulation model, SIAS Paramics, is used because this enables a detailed representation of the two junction types, especially concerning the traffic light control logic. Because S-Paramics itself only gives results regarding traffic flow additional software modules are used to generate safety data (SWOV TTC module) and emission data (AIRE software package).

To increase the reliability of the simulation results the simulation model is calibrated using video observations. For the SGFC-junction new video observations are made within this research. For the roundabout video observations from an existing study are used. Model inputs in the form of junction geometry traffic demand and gap distribution, and model outputs in the form of queue lengths, are extracted from the video. The visibility - and the gap acceptance parameter are adjusted in order to match the model generated queue lengths with the observed queue lengths. The one sample t-test or the Kolmogorov Smirnov test, in case that the queue lengths over the different model runs for one traffic demand scenario are not normally distributed, are used to check if the modelled queue lengths match the observed queues close enough. After calibration the model is validated with the same process using video observations from a different time of day. This resulted in a statistically validated model.

Junction performances are influenced by the amount and distribution of traffic. In this research traffic intensity, traffic distribution over main and side road and share of left turning traffic are varied to create a range of traffic demand scenarios.

For the majority of the tested traffic demand scenarios the roundabout offers lower travel times for motor vehicles. However at traffic demand scenarios with a high motor vehicle intensity of 3.000 mv/h the SGFC-junction would usually yield lower motor vehicle travel times. For the highest bicycle intensity of 1.800 bicycles / hour the SGFC-junction already offers better motor vehicle travel times at a motor vehicle intensity of 2.500 mv/h. Regarding bicycle travel time the roundabout scored better than the SGFC-junction for all tested demand scenarios.

Because roundabouts perform well when traffic demand is relatively balanced over the junction approaches further research should be done with asymmetrical traffic demand scenarios. Another limitation to this study is that the simulated network was too small. For a part of the demand scenarios not all motor vehicles could be added to the network at the intended time due to extensive queue forming. This means that not all travel time was recorded correctly. To solve this extrapolated motor vehicle travel times are used for the high motor vehicle demand scenarios. For future studies it is recommended to ensure that the size of the used simulation network allows all vehicles to be added to the network at the intended time during the complete simulation period.

For traffic safety the roundabout likely scores better than the SGFC-junction based on accident statistics for roundabouts and general signalised junctions. However the simulation results showed the opposite: the SGFC-junction scored best for all tested traffic demand scenarios. There are three reasons that could partly explain this difference. Firstly accident data studies usually group all signalised junctions together whilst the SGFC-junction is likely safer than the average signalised junction. Secondly, the simulated road users all comply to the traffic laws. It is possible that in reality traffic law infractions happen more often and/ or have a higher accident risk at one of the two junction types. Because of the strict rule compliance of simulated road users no accidents happen in the simulations. Therefore the safety evaluation in this research is based on a surrogate safety measure: the amount of TTC conflicts. It is likely that the relation between the simulated number of conflicts and the number of real world crashes is different for the roundabout compared to the SGFC-junction. This leads to an underestimation of the number of crashes at the SGFC-junction and thus to an overestimation of the traffic safety at the SGFC-junction. The third factor that could help explain the discrepancy between the results and other literature lies in the limitations of the number of TTC conflicts indicator. The exact relation between the amount of TTC conflicts and the amount of actual accidents is still unclear. It is likely that relation is different at a roundabout compared to a SGFC-junction. Furthermore the TTC indicator only uses conflicts between two vehicles. Single vehicle accidents are therefore not taken into account. Further research on the safety performance of SGFC-junctions using real world accident data is recommended.

Regarding environmental impact the ranking of both junction types probably is similar to the ranking for motor vehicle travel time. The junction type with the lowest travel time has the lowest amount of congestion and therefore likely the lowest amount of emissions. So for the majority of the tested demand scenarios the roundabout likely scores better than the SGFC-junction. At the high motor vehicle intensity of 3.000 mv/h the SGFC-junction likely yields lower emissions. At the highest bicycle intensity this tipping point is shifted to a motor vehicle intensity of 2.500 mv/h.

The emission results from this study could not be used to assess the environmental impact of the junction types because they seemed to unreliable. In some instances an increase in motor vehicle demand would lead to a decrease in total emissions.

Obtaining results from the emissions module turned out to be a difficult process. The AIRE software had difficulties processing S-Paramics output because a new vehicle type (the bicycle) had been added. After trying various ways the errors related to the bicycle vehicle type was solved. However the AIRE results still are not valid. A higher motor vehicle intensity sometimes results in lower total emissions and a reduction in average motor vehicle speed, which indicates more stop/go traffic, resulted in lower emission values as well. A check if the abnormal results could be explained by the low entry speed of motor vehicles into the network (15 km/h) proves that this is not the solution. Clearly further research is recommended.

Table 1 below gives an overview of the ranking of the junction types for each performance indicator for the majority of traffic demand scenarios i.e. those with: 3.000 mv/h, 200 bicycles/h and 60% main road motor vehicle traffic, all scenarios with a motor vehicle intensity below 3.000 mv/h except for the scenarios with 2.500 mv/h and 1.800 bicycles/h and 60% main road motor vehicle traffic.

General relative ranking of the junction type over the different performance indicators		
Junction performance type	**Junction design type**	
	SGFC-junction	**Roundabout**
Motor vehicle travel time	-	+
Bicycle travel time	-	+
Motor vehicle only conflicts	- *	+ *
Bicycle only conflicts	- *	+ *
Motor vehicle bicycle conflicts	- *	+ *
Total Carbon emissions	- *	+ *
Nox emissions	- *	+ *
PM10 emissions	- *	+ *
Applicable for all demand scenarios except scenarios with		
- 3.000 mv/h, 600 bicycles/h or more and 60% main road motor vehicle traffic		
- 2.500 mv/h, 1.800 bicycles/h and 60% main road motor vehicle traffic		
- 3.000 mv/h and 80% main road motor vehicle traffic		

Table 1: Overview of the relative ranking of both junction types over all junction performance indicators. The values marked with an * indicate values based on literature.

Table 2 below gives an overvies of the ranking of the junction tyeps for each performance indicator for the high demand traffic scenarios i.e. those with: a motor vehicle intensity of 3.000mv/h and 80% main road motor vehicle traffic, a motor vehicle intensity of 3.000 mv/h and a bicycle intensity of at least 600 bicycles/h and 60% main road motor traffic and those with a motor vehicle intensity of 2.500 mv/h and 1.800 bicycles/h and 60% main road motor vehicle traffic

General relative ranking of the junction type over the different performance indicators		
Junction performance type	**Junction design type**	
	SGFC-junction	**Roundabout**
Motor vehicle travel time	+	-
Bicycle travel time	-	+
Motor vehicle only conflicts	- *	+ *
Bicycle only conflicts	- *	+ *
Motor vehicle bicycle conflicts	- *	+ *
Total Carbon emissions	+ *	- *
Nox emissions	+ *	- *
PM10 emissions	+ *	- *
Applicable for demand scenarios with		
- 3.000 mv/h, 600 bicycles/h or more and 60% main road motor vehicle traffic		
- 2.500 mv/h, 1.800 bicycles/h and 60% main road motor vehicle traffic		
- 3.000 mv/h and 80% main road motor vehicle traffic		

Table 2: Overview of the relative ranking of both junction types over all junction performance indicators. The values marked with an * indicate values based on literature.

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I hope that the results of this report are used, either for direct application or as a basis for further research.

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Content

1. Introduction

In the Netherlands the bicycle is an important mode of transportation. Especially within urban areas, the bicycling levels are high (Fietsersbond, 2014). The Dutch national infrastructure and land use policy prescribes that all levels of government in the Netherlands promote the use of bicycles both as main mode of transport as well as part of a multimodal trip. This policy to stimulate cycling is supported with several arguments (Ministry of Transport, Public Works and Water Management; Fietsberaad, 2009):

- bicycling is sustainable because bicycles emit no substances and hardly any sound
- bicycling is healthy because bicyclists get exercise
- bicycling is cheap both for the bicyclists as well as for governments providing infrastructure
- bicycling enhances urban traffic circulation
- bicycling (and the associated lower car use) provides a better liveability in residential areas

Cycling thus has many advantages. Government bodies in the Netherlands can stimulate cycling levels in many different ways. Measures to increase bicycle use can be grouped into five categories (Trasporti e Territorio, 2010), these are:

- Engineering measures e.g. bicycle infrastructure and bicycle parking
- Education e.g. bicycle riding lessons and traffic safety campaigns
- Encouragement e.g. marketing advertisements and bicycle events
- Enforcement e.g. traffic laws supporting cycling and preventing bicycle theft
- Evaluation e.g. measuring effectiveness in modal share

Within the engineering category bicycle infrastructure is one of the main types of measures. There are many types of bicycle infrastructure such as: bicycle paths, bicycle bridges and bicycle traffic lights. The Dutch Ministry of Infrastructure and Environment set five criteria for bicycle infrastructure to ensure that it has the largest possible positive effect on bicycle use. Bicycle infrastructure should be: coherent, direct, attractive, safe and comfortable (Ministerie van infrastructuur en milieu, 2012). Specifically at junctions directness and safety are the most important factors to adhere to (Haan, Zeegers, & Linden, 2003).

Although there are many different types of bicycle infrastructure this research is specifically about bicycle friendly junction designs. At junctions there is a lot of potential to improve bicycle use by improving the infrastructure.

Firstly because of the delays that occur at junctions. In urban road networks the junction capacity is the most important factor in determining road capacity, not the link capacity. Because a junction has to handle conflicting traffic flows, which consist of different conflicting transport modes from different roads, the junction capacity is usually much smaller than the link capacity (OECD, 2007). Because the junction capacity is decisive in an urban network, junctions will also be the place at which most of the congestion and thus delay will occur. Junctions are therefore also the locations with the biggest potential to reduce delay and travel time and thus make bicycling more attractive. Delay is related to the directness (in time) of bicycle infrastructure.

The second reason to focus on junctions is traffic safety. The central and local governments have set the joint goal of reducing the annual number of traffic fatalities to 500 and the annual number of traffic injuries to 10.600 in the year 2020 (Schultz van Haegen, 2012). In the Netherlands the general traffic safety has improved over the last years. The annual amount of traffic fatalities has fallen from 791 in 2007 to 570 in the year 2013 (CBS, 2014). However the number of traffic fatalities amongst bicyclists has remained almost constant from 189 deaths in 2007 to 184 deaths in 2013 (CBS, 2014).

In recent years the amount of serious traffic injuries has increased from about 16.640 injuries in 2007 to about 19.200 in 2012 (Weijermars & Bos, 2014). There are no recent figures available, but in 2009 the share of bicyclists in the total number of traffic injuries was 58,1% (Reurings, Vlakveld, Twisk, Dijkstra, & Wijnen, 2012). Given that bicyclists make up a large share of traffic fatalities and traffic injuries, increasing traffic safety for bicycles is a vital step in achieving the set traffic safety goals for 2020. Measured over the period 2000-2009 about 60% of the accidents that caused traffic fatalities amongst bicyclists took place at junctions. Of all the accidents between a motor vehicle and a bicycle that resulted in serious injury for bicyclists 66% took place at a junction. Improving traffic safety at junctions can thus have a large impact on the overall safety of bicyclists and therefore also make an important contribution to achieving the planned reductions in traffic fatalities and injuries.

1.1. Bicycle friendly junctions

The previous paragraph established that by implementing bicycle friendly junctions cycling can be promoted. This paragraph gives an overview of the criteria which define the bicycle friendliness of a junction. First the waiting time, which falls in the category directness, is discussed. Secondly the traffic safety, specifically the safety for bicycles, is discussed.

1.1.1. Waiting time

One junction design aspect that affects the bicycle friendliness of a junction is the average waiting time for bicycles. The average waiting time consists of two factors and is calculated as follows:

$$Total\ average\ waiting\ time = stop\ probability\ \times average\ waiting\ time\ when\ stopped$$

- *stop probability* = the probability that an approaching bicycle has to stop for a red light or to give way
- *average waiting time when stopped* = the average time in seconds a bicycle has to wait before it can cross the junction in case it has been stopped

The total average waiting time is directly related with directness: a shorter waiting time at a junction leads to a lower travel time and thus results in a more direct bicycle connection. The average waiting time when stopped has an effect on traffic safety as well. A shorter waiting time when stopped is believed to lead to a higher red light compliance by bicyclists and thus to a higher traffic safety (Meel, 2013). A field study conducted at a junction in Grave in the Netherlands supports this idea. At this junction the amount of red light violations by bicyclists decreased between 23% and 78% depending on the exact light control configuration (Harms, 2008).

The stop probability influences the comfort for cyclists of the junction. Decelerating and accelerating again asks extra mental effort from cyclists because it is more difficult to balance a bicycle at low speeds. Furthermore accelerating after a stop requires a lot of extra physical effort. The energy required for accelerating again after a stop equals the energy needed for cycling up to 200 meters at cruise speed (Hendriks, 2010). Reducing the amount of stops bicyclist have to make will thus increase their comfort.

The CROW has produced a guideline to assess the bicycle friendliness of a junction based on the total average waiting time in their "Ontwerpwijzer Fietsverkeer" ('Design manual for bicycle traffic') (CROW, 2006). Figure 1 below visualises this guideline. The x-axis displays the stop probability for a bicycle approaching the junction. The y-axis displays the average waiting time when stopped. A junction is bicycle friendly when the average waiting time is below 15 seconds. This threshold is represented by the blue line. For any point on the blue line it holds that $stop\ probability \times average\ waiting\ time\ when\ stopped = 15$ seconds. An average waiting time of 20 seconds or higher is seen as bicycle unfriendly. When a junction scores an average waiting time between 15 and 20 seconds it is classified as moderately bicycle friendly.

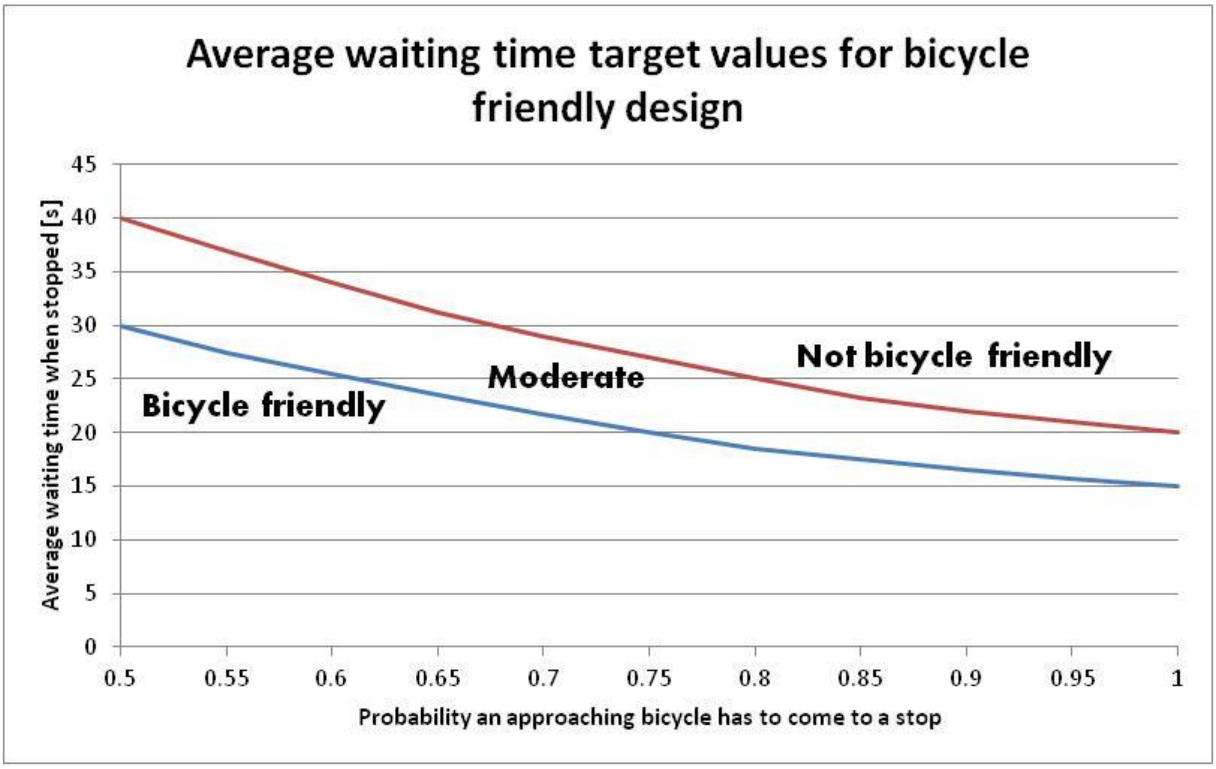

Figure 1: CROW average waiting time thresholds for bicycle friendly junctions

Signalised junctions
For signalised junctions the probability that a bicycle has to stop is calculated by dividing the average red time for cyclists by the average total cycle time over a representative period. For junctions with fixed time signals the average waiting time for a stop is simply half of the red time for cyclists assuming that bicyclists arrive randomly distributed over time. For junctions with variable signal timings the average waiting time when stopped depends on how the signal timings are set and on the traffic supply at opposing flows. These factors determine the minimum and maximum time between the detection and thus arrival of the bicyclist and the moment that the bicycle light turns green. The average waiting time when stopped lies exactly in the middle between this minimum and maximum time, under the assumption that bicyclists arrive randomly distributed over time.

Roundabouts
For roundabouts it is more difficult to calculate the stopping probability and average total waiting time for bicyclists. In the situations where bicycles have priority one may expect that there is no probability of stopping and (therefore) no waiting time. However occasionally motorists take priority forcing cyclists to slow down and/or stop. A study by Van Minnen and Braimaister, which comprised four roundabouts with bicycle priority, shows that on average 20% of bicyclists gave way to cars (Minnen & Braimaister, 1994). On average 10% of bicyclists had to come to a full stop at these four roundabouts. Table 3 below shows the average time lost per bicycle for crossing one roundabout arm against the total motor vehicle intensity on that arm. The study found no clear relation between time loss and motor vehicle intensity on the roundabout arm.

Average time loss for bicycles crossing one roundabout arm		
Roundabout ID	**Motor vehicle intensity [mv/h]**	**Average time loss [s]**
1	280	0
2	627	2.7
3	676	2.6
4	694	0.8

Table 3: Average time loss for bicycles crossing one roundabout arm against motor vehicle intensity

At roundabouts without priority for bicyclists, their total average waiting time is strongly related with motor traffic intensities. For low and medium motor traffic intensities on the roundabout the time losses for bicycles are low, but for higher intensities the waiting time can become substantial because it increases more than linear with motor vehicle intensity (Minnen J. v., 1998). In the same study Van Minnen constructed an empirical model predicting the average time loss for cyclists based on motor traffic intensity, see Figure 2. At very high intensities the model is overestimating the time loss for bicyclists. This is probably because motor traffic is partly congested then, which allows bicyclists to cross more easily between slow moving or stationary motor vehicles.

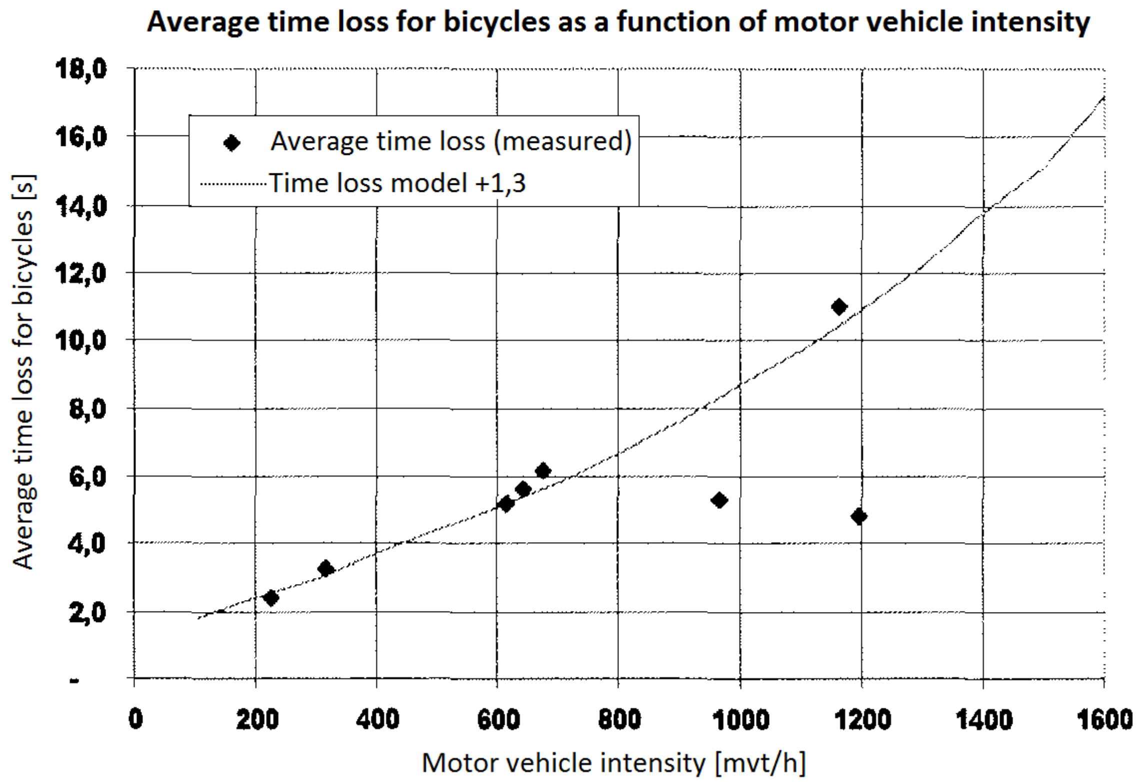

Figure 2: Average time loss for bicycles crossing a roundabout arm as a function of motor vehicle intensity.

Major / minor junctions

At major / minor junctions the waiting time for bicyclists at the minor road will depend directly on the motor traffic intensities of the major road as well. For bicyclists at the major road the waiting time will only be slightly influenced by the motor vehicle flow on the minor road, comparable to the effect on bicycle flow at roundabouts without priority for cyclists.

Uncontrolled junctions

At uncontrolled junctions three traffic laws regulate priority:

- drivers from the right have priority
- right turning drivers go before left turning drivers
- through going road users go before turning traffic

Depending on the direction a cyclist rides across the junction one or more of these three rules apply. These rules determine which traffic flow(s) have priority over the cyclist. The waiting time for cyclists will depend on the (combined) traffic intensity of the opposing traffic flows.

Grade separated junctions
For grade separated junctions the waiting times for cyclists will not be related to the motor vehicle intensity because these different transport modes are completely separated.

1.1.2. Safety

Regarding traffic safety for bicycles the "Ontwerpwijzer fietsverkeer" ('Design manual for bicycle traffic') does not have quantitative norms. Instead the CROW has formulated five qualitative recommendations (CROW, 2006):

- Minimise the number of conflicts with motorised traffic
- Conflicts should be bundled as much as possible to create a clear traffic situation
- At junctions at grade the speed differences between traffic flows should be minimised
- Bicyclists should be in the field of view of motorised traffic
- Junctions should be adequately visible during both day time as well as night time

The number of conflicts of bicycles with motorised traffic can be reduced in two ways. First by separating the bicycle traffic from motorised traffic in space for instance by creating a bypass for right turning bicycles. The other way is to separate bicycles and motor vehicles in time, for instance by using traffic lights with separate green phases for bicycles only. A lower number of conflict points leads to a lower amount of potential collision points and thus increases safety.

Bundling conflicts reduces the geographical spread of conflict points over the junction area. This gives drivers a clearer overview of the junction, and therefore not only increases safety but also the coherence of the junction. For example compared to a double lane roundabout a turbo roundabout limits weaving conflicts amongst motor vehicles to only the approaches instead of the approaches and the roundabout lanes. Therefore the turbo roundabout has a better safety record, than the double lane roundabout (CROW, 2008).

Minimising the speed differences at junctions increases traffic safety in two ways. Because of the lower speed differences drivers have more time to react to each other and avoid conflict by e.g. braking or swerving. Lower speed differences also mean that in case that a collision does occur the consequences will be less severe. So even if the amount of collisions remains constant the number of casualties and or wounded will be lower.

The last two recommendations about bicycle safety are aimed at improving the visibility of bicyclists. With a higher visibility the probability that motorists overlook a bicyclist decreases and thus leads to fewer collisions.

Signalised junctions
The main way signalised junctions increase safety is by separating conflicting traffic flows in time. The configuration of the traffic lights determines which traffic flows, if any, still conflict with each other and thus how many conflict points remain. By introducing right turn bypasses for bicycles the number of conflict points can be reduced by geographically separating traffic flows as well.

A common way to bundle traffic flows and thus reduce conflict points is to have cyclists perform a two stage left turn. For example a cyclist turning left from the South approach onto the West approach first has to cross the East approach just as a straight through going bicycle and in second instance cross the North approach together with cyclists going from East to West. This way conflict points of straight crossing bicycles and left turning bicycles are bundled. Another option is to introduce a dedicated green phase for bicycles whilst all motor vehicle is stopped. Implementing a Simultaneous Green For Bicycles (SGFC) phase reduces the amount of conflicts between bicycles and motor vehicles to zero, excluding red light violations.

Although red traffic lights slows down a part of the road users, the signalised junction does not reduce the speed differences between all road users. The location of the bicycle paths in relation to the motor vehicle lanes has a big impact on the visibility of cyclists.

On the junction approaches the bicycle path should be close to the motor vehicle lane so that bicyclists appear in the generally forward facing view of motor vehicle drivers. When crossing other junction arms the bicycle path should be located about five meters from the motor vehicle lane. This way a right turning motor vehicle crosses the bicycle path at an angle closer to 90 degrees and motorists do not have to turn their heads as far back. This increases the visibility of bicycles. Another important factor to increase the visibility of bicycles is to ensure that there are no obstacles between the bicycle path and the motor vehicle lane.

By providing adequate lighting the visibility of cyclists can be increased at night. Since this is possible at any junction type it will not create safety differences between junction types and is therefore not discussed further.

Roundabouts
At roundabouts right turning bicycles are completely separated from motor vehicles. Other bicycle flows are not separated from motor vehicles in time or space. Bundling of conflict points is a default bicycle safety characteristic of roundabouts. Straight through going and left turning bicycles cross the junction approaches in the same location. Another bicycle safety feature of roundabouts is that the speeds of all motor vehicles and thus three speed differences between road users is decreased. Regardless of the direction at which the motor vehicle crosses the junction it needs to slow down to make the curve around the central island. By leaving about five meters between the motor vehicle circle lane and the circular bicycle path the bicycle safety is increased. This distance ensures that motor vehicles cross the bicycle path at an angle closer to 90° which means that approaching bicycles are better visible.

Major / minor junctions
At major / minor junctions the major road usually has separated bicycle infrastructure. In order to decrease the accident risk between turning motor vehicles and straight through going bicycles this bicycle infrastructure should be placed at some distance, about 5 meters, from the roadway. Turning motor vehicles then cross the bicycle path at an angle close to 90 ° which increases the visibility of bicycles. To specifically reduce the speed of turning motor vehicles small corner radii can be applied.

Uncontrolled junctions
At uncontrolled junctions bicycle traffic usually shares the road with motor vehicles. The amount of conflict points for bicycles is therefore not reduced. To reduce speed differences between road users the junction can be constructed on a raised table. To specifically reduce the speed of turning motor vehicles small corner radii can be applied.

Grade separated junctions
Grade separated junctions ensure bicycle safety by separating motor vehicles and bicycles in space. Because the complete separation between the two modes bundling of conflicts, reducing speed differences and visibility of bicycles are no longer applicable.

1.2. Advantages and disadvantages of bicycle friendly junctions

The specific requirements for bicycle friendly junctions offer several advantages over more traditional junction designs. Some of these advantages are only beneficial for bicyclists, and may actually pose a disadvantage for others, others are beneficial for all road users.

The shorter average waiting times for bicyclists naturally is advantageous for bicyclists. As explained in paragraph 1.1 above this leads to shorter travel times and higher comfort for bicyclists. The increased red light compliance and resulting improvement of traffic safety is a benefit for all types of traffic on the junction. A lower amount of accidents results in a higher availability of the junction.

However shortening the average waiting time for cyclists at junctions has disadvantages for other road users. It means that the average waiting time of other road users increases. This results in longer travel times for motor vehicles and possibly for pedestrians as well. Especially for motor traffic the longer average waiting times result in longer queues. If these queues become very long they may block traffic flow at other nearby junctions. The effect of a certain junction design on motor traffic congestion is usually evaluated before implementing a new junction design.

These increased queues and congestion of motor vehicles will lead to increased emissions from motor traffic. This will lead to a decreased local air quality and accelerate climate change. This affects not only the road users at the junction, but also the local residents. This contrasts with some of the reasons to stimulate bicycle use mentioned in the introduction, specifically: bicycle use has zero emissions and the enhancing of livability. It is therefore important to investigate how the bicycle friendly junction performs regarding emissions. The magnitude of the effect of bicycle friendly junction designs on emissions is however not well researched. A study published by the CROW gives an estimate of the emissions impact of implementing a second bicycle green stage into a signal control plan, but it does not specifically include a SGFC signal control plan in their further comparison of emissions of junction designs (CROW, 2010).

The design recommendations for traffic safety (CROW, 2006), will make bicycle friendly junctions safer for bicyclists compared with traditional junction designs. The increased traffic safety is beneficial for all road users at the junction because it leads to a higher availability of the junction. The CROW recommendations to make junctions safer for bicyclists have however disadvantages as well. The amount of conflicts between bicycle traffic and motorised traffic can be reduced by separating these flows in time or in space. Traffic flows can be separated in time by adjusting the traffic signal timings. These adjustments however lead to longer traffic light cycle times and thus to longer wait times for all road users. Separating traffic flows in space, e.g. by building bypasses or a grade separated junction, increase the construction costs and use of space of the junction.

The recommendation to decrease speed differences between the different road users has disadvantages as well. When road users are forced to slow down they will have to accelerate again as well. Specifically for motor vehicles this will lead to more emissions which have a negative impact on the air quality. For bicyclists the main negative impact of extra accelerations is a greater discomfort. The traffic safety of different types of junction designs is usually evaluated as well. Dutch road authorities are mandated to guarantee an optimal traffic safety (Provincie Overijssel, 2012).

Implementing bicycle friendly junctions can have several advantages and disadvantages. It is therefore important for road authorities to know the performances bicycle friendly designs to be able to make well funded decisions.

1.3. Junction types under consideration

Several different junction design types have the potential to meet the requirements regarding waiting time for bicycles and traffic safety. Two common junction designs that are constructed in order to stimulate cycling are traffic light junctions with Simultaneous Green For Cyclists (SGFC) and roundabouts with priority for bicycles. Both the SGFC-junction and the single lane roundabout can be applied at locations with a daily intensity up to 25.000 Passenger Car Equivalents (PCE) (Eggen, Salomons, & Zeegers, 2003) and (CROW, 2012). Because these two junction types are both bicycle friendly junctions and both can be applied in the same motor traffic intensity range, road authorities often have to choose between these two designs. The differences between the performances of the SGFC-junction and the roundabout with bicyclist priority are not clear. To enable road authorities to make a well informed decision about which of these junction types fits best with their goals this research compares the performances of these two junction types. A detailed description of the two junction types included in this research is given in appendix A.

Apart from these two common designs there are more junction types that potentially meet the requirements for bicycle friendly junction designs. A grade separated junction offers low travel times and high safety for bicycles because they are completely separated from motor vehicles. These junctions however require a lot of space which makes them unfeasible in most urban locations where bicycle friendly junctions can provide the biggest benefits because bicycle use and time lost at junctions is higher in urban areas than outside urban areas. Uncontrolled junctions could also meet the bicycle friendly junction requirements, but they work only at locations with low motor vehicle intensities. Because they usually do not separate bicycle traffic from motor vehicle, higher motor vehicle intensities would lower bicycle safety too much.

1.4. Research questions

As reported in the introduction one of the measures road authorities can take to stimulate bicycle usage and reduce the number of traffic casualties amongst cyclists is implementing bicycle friendly junctions. The roundabout with priority for bicycles and the SGFC-junctions are two junction types that can fulfil the requirements for a bicycle friendly junction design. They can be applied in the same types of locations because they can both handle up to 25.000 PCE per day (Eggen, Salomons, & Zeegers, 2003) and (CROW, 2012). For road authorities making a well informed decision for either one of these junction types is important to ensure an efficient use of their resources.

Several studies compare the performance of roundabouts with signalised junctions. For instance the Dutch "Recommendations for urban traffic infrastructure" by the CROW announces the roundabout as the preferred junction type at crossings between two distributor roads unless restrictions regarding capacity, delay, use of space or costs exist (CROW, 2012). This indicates that usually roundabouts will be outperformed by other junction types at high traffic intensities. What kind of traffic demand scenario could cause these restrictions regarding capacity is however not explained. Several Dutch studies have found that roundabouts offer a higher traffic safety than signalised junctions, for example (Minnen J. v., 1995) and (Dijkstra, 2014). These studies however do not include signalised junctions with SGFC signal control. Regarding emissions Hyden and Várhelyi found that replacing signalised junctions with roundabouts reduces emissions of CO with 29% and NO_x with 21% (Hydén & Várhelyi, 2000). However the studied junction types did not have dedicated bicycle infrastructure which means that the results could be different for the Dutch situation.

On top of the current knowledge about the performances of different junction types Dutch road authorities could be helped by more detailed information about junction performances at specific traffic demand scenarios and by more knowledge specific to Dutch infrastructure where bicycles play an important role. This need for information leads to the main research question:

"How do the roundabout with bicycle priority and the SGFC-junction perform over different traffic demand scenarios?"

It is possible that the performance of one junction design type relative to the other design type differs for different traffic demands. It may for instance be that a roundabout scores lower congestion values than a signalised junction at low traffic intensities whilst at high traffic intensities the signalised junction scores better. Therefore several traffic demand scenario's are tested.

The junction performance is measured in three different fields: congestion, traffic safety and environmental impact. This leads to the following three sub research questions:

"What is the difference between the throughput of the junction designs?"

"What is the difference in traffic safety between the junction designs?"

"What is the difference in environmental impact between the junction designs?

1.5. Research goal

By answering the research questions it is known how each of the bicycle friendly junction types performs on three types of performance criteria over a range of traffic demand scenarios. This research probably will not be able to categorically state which junction design is the best because that will likely depend on the relative weights that a road authority places on each of the three performance criteria. For instance what level of congestion increase is acceptable given a certain increase in traffic safety. However with the information about the junction performances road authorities will be able to better quantify these trade-offs and therefore make better informed decisions.

Table 4 below shows what the results of the research could look like. The numbers are not indicative of any of the real results. In case that a road authority views traffic flow of motor vehicles as most important it should choose the SGFC-junction design because the travel time for motor vehicles is lower than the roundabout in this case. Traffic safety is expressed in the number of Time To Collision (TTC) conflicts. A TTC conflict occurs when the TTC value between two road users falls below a threshold value. The TTC value is defined as the time it would take before two road users collide given their current speeds and directions of travel. In case a road authority considers traffic safety for bicycle as most important it should construct the SGFC-junction in this case. If however low emissions have the absolute priority the roundabout is the best option. It may also be that with certain traffic demand scenarios one of the designs scores better in all aspects than the other. In those cases the conclusion to this research will naturally be to recommend that junction design type at that specific (range of) traffic demand scenarios.

Situation 1								
Car intensity **2500 PCE/h**	**Main road traffic car**	**80%**	**Left turn Car**	**5%**				
Bicycle intensity **1000 b/h**	**Main road traffic bicycle**	**80%**	**Left turn bicycle**	**5%**				
Junction design type	Travel time		Safety			Emissions		
	motor vehicles [s]	cyclists [s]	No. of motor vehicle only TTC conflicts [#]	No. Of bicycle only TTC conflicts [#]	No. Of motor vehicle bicycle TTC conflicts [#]	CO2 [gram]	NOx [gram]	PM10 [gram]
SGFC junction	10,000	5,000	300	80	20	13,000	5,800	11,500
Roundabout	12,500	4,000	220	150	200	12,000	5,000	11,000

Table 4: Example of what the simulation results could look like.

The remainder of this report is constructed as follows. Chapter 2 describes the research framework. It describes the main research steps and research constraints. Chapter 3 is about the preparation for the simulations. It starts with a choice for the simulation software package followed by the description of the calibration and validation of the software model. The last section of chapter 3 describes the preparations for the simulation runs, for example the used traffic demand scenario's, the length of the simulated time and the amount of runs per traffic demand scenario.

Chapter 4 describes the outcomes of the simulation model. The results are grouped into results about travel time, about traffic safety and about emissions. The end of chapter 4 gives a combined overview of the results over all three types.

Finally chapter 5 draws conclusions from the results presented in chapter 4. Chapter 5 also compares the results to other literature and it contains recommendations for further research.

2. Research framework

This chapter gives an overview of how the research questions are answered. Paragraph 2.1 gives an overview of the research: which steps are taken in the process of answering the research questions. The second part focuses on the research scope. Paragraphs 2.2 through 2.4 discuss some of the research constraints. Paragraphs 2.5 through 2.7 give a more detailed look on certain parts of this study.

2.1. Research overview

The goal of this research is to evaluate the performances of two bicycle friendly junction types. This section gives an overview of the research steps that are taken to achieve the research goal. A schematic overview of the research steps is presented in Figure 3.

Detailed data about the performances of the two junction types under different traffic circumstances are not readily available. There are two ways to get the needed data: take measurements at existing junctions or generate it by using simulations. In this research the latter method is chosen because: it gives the researcher exact control over which designs and traffic demand scenarios are investigated, it costs relatively little resources and because it offers flexibility in case changes in the (to be) investigated situations are needed. Because traffic simulation models typically do not generate information about traffic safety or the environmental impact of traffic it is necessary to use additional software models, these are called effect models. The effect models use information about the traffic flow generated by the traffic simulation model as input to calculate the traffic impact on their specific topic. Logically one of the first steps of this research is to set up the traffic simulation model.

An important part of the set up of the traffic simulation model is the model calibration. Because the research conclusions are based on the simulation results it is very important that the simulations accurately represent reality. For the model calibration data on the performances of existing junctions is needed. Because this information is not readily available it is necessary to conduct measurements at existing junctions. These measurements are done via camera observations. Camera observations allow the researcher great flexibility because the equipment is relatively easy to setup compared to for instance loop detectors. In addition camera observations can be used to extract a variety of different traffic flow variables. Furthermore camera observations give very reliable data because the images are stored and can be (re)analysed as many times as necessary. One SGFC-junction in the city of Enschede is observed. The cameras are provided by Keypoint Consultancy BV. No new roundabout observations are made. Instead videos shot for a previous research are used.

For the model calibration different types of data are extracted from the video observations. The first data type is the model input data. It consists of the geometrical layout of the junction, the traffic demand and or the SGFC-junction the general signal control logic. The second data type are model parameter values. This includes mean headway, headway distribution and critical gap values. The final data type is the calibration comparison data which consists of the minimum and maximum queue length per five minute interval and per lane if applicable.

For the model calibration the simulated maximum queue lengths per five minute interval are compared to the observed queue lengths. If the simulated queue lengths match the observed queue lengths close enough than the simulated junction processes vehicles at the same rate as the existing junction. A statistical test is used to check if the simulations match the observations close enough. In case the simulated queue lengths do not match the observed queue lengths the model parameters visibility and gap acceptance are adjusted. For the model validation the same process is repeated, only then with results camera observations from a different time period.

After model calibration and validation follows the second part of the simulation model set up: the definition of the simulation runs that are done. At a more macro level it is necessary to determine which situations are simulated and how many simulations are run per situation. For each simulation run the model needs different kinds of input. Firstly it needs a representation of the spatial layout of the junction and its approaches. Specifically for the SGFC-junction the traffic simulation model also needs a traffic light control logic. Secondly it needs traffic demand input. How many vehicles, of a certain vehicle type, want to travel from where to where over the network at what time. Finally it is necessary to define how much time should be simulated.

After the setup of the traffic simulation model all runs can be executed. Part of the traffic simulation outputs are then analysed in order to present the junction performances regarding capacity. Part of the traffic simulation outputs also serve as input for the effect models which can now be run. After running the effect models their respective outputs are also analysed to present the junction performances regarding safety and emissions.

For each of the three types of junction performances the analysis starts with a comparison between the two junction types. A statistical test is used in order to assess the significance of the differences between the two junction performances. Additionally the impact of the different traffic demand scenarios on the performance of an individual junction is analysed. This serves as an extra check for the reliability of the model outputs. It is for instance common sense that the level of emissions increases with an increase in motor vehicle intensity. Analysing the impact of the traffic demand variables also gives an insight in the usefulness of including that variable. If for instance the share of left turning bicycles hardly impacts the junctions performance than future studies do not have to take this variable into account.

The final research step is to combine the research results and give an overview of which junction type performs better at which performance type under which traffic demand scenarios.

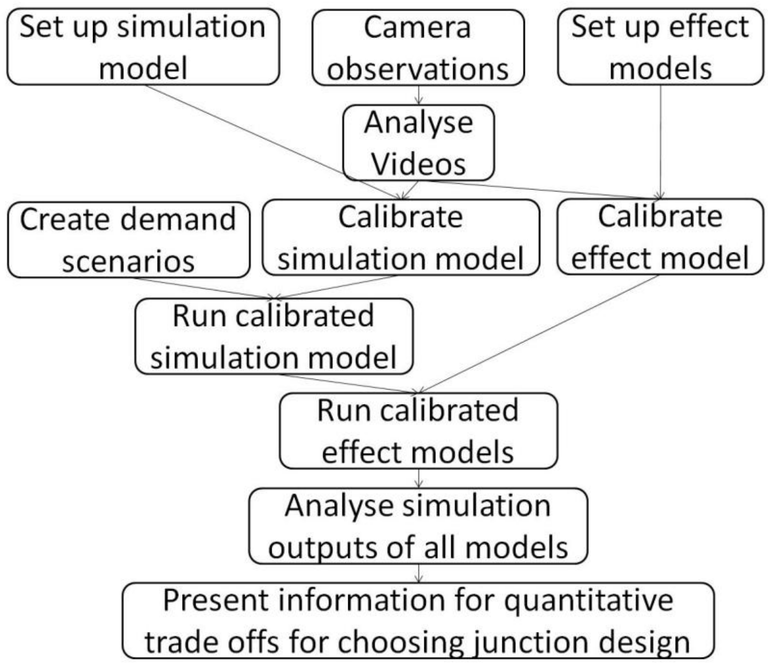

Figure 3: Schematic overview of the research steps

2.2. No network effects

Different junction designs not only influence flow characteristics of incoming traffic or traffic on the junction but also the outgoing flow. In urban areas, where junction density is generally high, the outflow characteristic of a junction can impact the performance of a downstream junction. This means that a design that leads to an optimal performance at the level of that one junction, may actually be suboptimal when including the surrounding network in the analysis. This would however add even more variables to the research making it more time consuming. Because of the time constraints for this research the influence of the characteristics of the out flowing traffic on the performance of downstream junctions will not be taken into account.

Different junction designs will result in different traffic flow conditions, for instance a higher average speed. In theory these changes in traffic conditions lead to people recalculating some of their trip decisions which could result in people changing their mode choice, route choice or departure time choice. These different decisions result in different traffic flow conditions which in turn result in different performances of junctions and a different environmental impact of traffic. This research will however only focus on the instantaneous differences in junction performance between junction designs and ignore the potential long term effects that may occur as a result of a change in people´s decision making. It is assumed that the changes in traffic conditions at one junction will not have a significant effect on people´s travel decisions and therefore not impact traffic demand.

This research investigates junctions in the urban area. Cycling levels are much higher in urban areas than in rural areas (Fietsersbond, 2014). Within the urban area cyclists generally have a higher priority at junctions compared to rural areas, see for instance the CROW recommendation to give cyclists priority at roundabouts within the urban area, (CROW, 2008). Because of the higher modal share of cycling in urban areas conflicts with other modes such as motorised traffic are more likely to occur here. Given the legal recommendations to prioritise cyclists and the higher cycling share in urban areas it is more likely and useful to construct bicycle friendly junctions in urban areas. Following these arguments this research is about bicycle friendly junctions in urban areas.

2.3. Traffic mode constraints

In this research two mode types are included: motorised traffic and bicycle traffic. These are the two most important modes looking at the modal split by number of trips. For 48% of the trips a car is used and for 26% of trips a bicycle is used (CBS, 2013). Several different modes are included under motorised traffic. These include: Heavy Goods Vehicles (HGVs), Light Goods Vehicles (LGVs) and passenger cars.

In this research dedicated public transport priority will not be included. Public transport has the potential to be less polluting than individual motor vehicles. Mainly for this reason public transport is provided with separate lanes, lights and priority over other modes at some junctions. Giving priority to public transport for instance by shortening the green phase of other modes of transport can have great impact on the junction capacity for those other modes, as well as on the environmental impact of the junction. So public transit can play an important role in junction design. The reason to not include it in this research is because the layout of dedicated public transport infrastructure is very location specific. This makes the effects of public transport priority very location specific as well whilst this research is trying to find generalised effects.

Pedestrians will not be included as a separate mode. In most of the Dutch junctions the pedestrian volumes are so low that they don't cause any capacity problems. Since they don't cause any environmental problems and hardly cause any traffic safety problems for others they are not considered in this research.

2.4. Simulation software constraints

In order to be able to process the large number of traffic demand scenarios this research will use a simulation model to evaluate the junction designs. For this research it is important that the amount of congestion is simulated in detail. After all one of the factors on which the junction designs are evaluated is the total time spent. In order to accurately represent the forming and dissipating of congestion a dynamic traffic model has to be used instead of a static model. A static traffic model assumes that conditions remain the same over time. This constitutes both the traffic demand and the driving conditions on links.

Because this research is about detailed differences in junction design it is necessary to simulate these junctions realistically and in great detail. For example the junction geometry has to be accurately represented and the traffic light cycle has to be simulated.

It is also important that the interactions between the road users of the two modes, motor traffic and bicycles, are simulated in detail. Because this research is about comparing bicycle friendly junction designs it is important to look at the effect of bicycle priority on motor vehicle traffic. To satisfy this need the research is executed with a micro-simulation software package instead of macro simulation software because in a micro-simulation software package the behaviour of individual vehicles is modelled.

2.5. Junction performance definition

Traditionally four different types of criteria are assessed when considering what type of junction should be build (CROW, 2008). These are: flow of traffic, traffic safety, use of space and construction costs. Next to these four the environmental impact of junction designs is gaining importance. In urban areas motorised traffic has a big negative impact on air quality (Groene Ruimte, 2013). Adjusting junction designs can positively contribute to keeping or getting local air quality to comply with European regulations (Koning, 2010). This research will only deal with the three traffic engineering related criteria: flow of traffic, traffic safety and environmental impact. The remainder of this paragraph explains why this research evaluates junctions on these three criteria.

The assessment criteria spatial constraints and construction costs will not be taken into account. Spatial constraints are location specific and there is no uncertainty about this criterion in junction type evaluations in practice. There is already a lot of practical knowledge about construction costs of roads and junctions which is expressed in key figures, for instance the costs of pavement per square metre. Examples include (but are not limited to): 'KengetallenKompas GWW 2013' (Vonk, Wilde, & Groot, 2013) and 'GWWkosten' (BIM Media B.V., 2015).

The traffic engineering related performances of the two junction designs have to some extend been researched already as well, the results of several researches are included in the detailed junction description in appendix A. Most of these studies however only assess the performance of different junction types on only one of the three criteria. This makes it difficult to get an overview of a junction type's overall performance because the different studies may use slightly different definitions regarding junction design and traffic demand scenarios. By evaluating the two junction designs on all three criteria in one study this research strives to overcome this problem.

Another reason to look into the three traffic engineering related criteria is that the influence of bicycle traffic on these criteria is not fully explored. Although the bicycle plays an important role in the Dutch transport system there is still a lack of knowledge about bicyclists' behaviour on the road and their effect on other road users (Brömmelstroet, 2014). Several studies from outside the Netherlands ignore bicycle traffic completely, because the bicycle only plays a tiny role in traffic in their respective countries. Other studies from outside the Netherlands do mention bicycles, but because the junction designs do not incorporate specific bicycle infrastructure these studies are still not comparable to the Dutch circumstances.

2.5.1. Junction performance indicators definition

In order to quantify junction performance in the three criteria traffic flow, traffic safety and environmental impact, it is necessary to choose at least one indicator per criterion. This paragraph describes which indicators are used for each of the three performance criteria.

For measuring the junction traffic flow the indicator total time spend is used because it can measure the full effect of the junction design on traffic flow. Although this indicator does not directly distinguish the state of the traffic flow, the effects of this state on road capacity are represented in the simulation and thus indirectly included in the measurements. Compared to using queue length as an indicator the total time spend does not have to be adapted to the used number of lanes. Another disadvantage of a queue length indicator is that it is necessary to define when a vehicle is queued. The chosen definition may have a big impact on the comparison between the junction types.

The two junction designs are intended to provide short waiting times for cyclists, which will likely result in longer waiting times for motor vehicles. In order to analyse these effects the total travel time is specified for both traffic modes. The fewer time is lost, the more efficient a junction is at handling the traffic flows.

Traditionally traffic simulation models have not been used to evaluate traffic safety. Because simulated road users adhere to traffic laws and never get distracted accidents never happen within the simulations. Because there are no accidents to record other, surrogate, safety measures have to be used. In general there are three types of surrogate safety indicators: exposure based, conflict based and expert judgement (Dijkstra, 2011). Exposure based indicators use traffic intensities and accident risks based on real world observations to determine traffic safety. Because the SGFC-junction is not so widely applied determining an accident risk based on real world accident data is not feasible. For the same reason expert judgement also is unfeasible to use. Therefore the assessment of junction safety is done with a conflict based indicator.

Out of the potential conflict based indicators the number of Time To Collision (TTC) conflicts is used in this study. The TTC is defined as the time it would take for two vehicles to collide given their current locations and speeds (Archer, 2005). When the TTC value drops below a certain threshold it is unlikely that a human driver would have time to react and avoid a collision. In this research the threshold is set at 1,5 seconds. By comparing the number of TTC conflicts of one junction type with the number of the other junction type it is possible to give an indication of their traffic safety relative to each other. The main advantage of using the number of TTC conflicts as an indicator is that it can be used to measure both rear-end conflicts on links as well as conflicts at an angle on intersections (Dijkstra, 2011).

A disadvantage of using the amount of TTC conflicts as a safety indicator is that the relation between the amount of conflicts and the amount of accidents is not clear (Archer, 2005). However several studies have demonstrated that there is a correlation between the number of simulated TTC conflicts and real world accidents (Saleem, Bhagwant, Shalaby, & Ariza, 2014), (Dijkstra, 2011) and (Astarita, Giofré, Guido, & Vitale, 2012). Because the research goal is to compare two junction types to each other and not to predict the exact number of accidents at a junction it is not a problem that the link between TTC conflicts and accidents is fully understood.

Another important limitation of conflict based indicators is that, because it requires a conflict between two road users, single vehicle accidents are not included. However from 2002 through 2006 about 46% of the total registered accidents with motor vehicles were single vehicle accidents

(Kuiken, Bolle, & Nägele, 2008). An older study specifically looked into single bicycle accidents and found that about 60% of the total number of bicycle accidents are single bicycle accidents (Schoon & Blokpoel, Frequentie en oorzaken van enkelvoudige fietsongevallen, 2000). It is not known whether this reported share of single bicycle accidents applies for accidents at junctions as well. At this moment simulation software is not capable to generate information about single vehicle accidents.

Bicycle friendly junctions should offer traffic safety specifically for bicycles. To be able to give a more bicycle specific safety assessment the number of TTC conflicts is specified in three categories: motor vehicle only conflicts, bicycle only conflicts and motor vehicle bicycle conflicts. The first category does not say much about bicycle safety specifically. The second category does directly relate to bicycles however a collision with only bicycles is unlikely to result in severe injuries or traffic fatalities. The third category arguably has the biggest impact on bicycle safety. Because of the differences in mass between a motor vehicle and a bicycle a collision between these two is likely to severely injure or kill the cyclist.

The evaluation of the junctions environmental performance the indicators Total Carbon (TC), PM_{10} and NO_x are used. These indicators each represent different kind of environmental impact. The TC indicator quantifies the junction's impact on global warming. The NO_x indicator describes a more regional environmental impact. NO_x emissions are one of the contributors to acid rain. Finally the PM_{10} indicator describes a local environmental impact. Small particulate matter causes serious health problems for humans because the particles are so small that they can enter the bloodstream through the lungs after inhalation.

Appendix B gives more details about the decision for the used variables.

2.6. Traffic demand scenarios

Because junction performance may be influenced by the traffic demand, this study tests the two junction designs over a range of traffic demand scenarios. This paragraph presents how these scenarios are set up.

The traffic demand scenario is described with three different types variables: hourly intensity, distribution of traffic over the main road and the side road and the percentage of left turning traffic. These three variables have been chosen because they all influence the amount of conflicts that occur at the junction. In turn the amount of conflicting traffic movements occurring at a junction impacts the junction performance. Each of the three demand variables is split into two parts: one that defines the value for motor vehicles and the other defines the value for bicycles. Both of the intensity variables have five levels. The other variables have only two levels mainly because the total number of traffic demand scenarios needs to be limited to make sure that the research can be executed within the time constraint. Table 5 below gives an overview of the amount of traffic demand scenarios that are simulated.

Set-up and amount of simulated traffic demand scenarios							
Levels	Traffic demand variables						Total
	Total motor vehicle intensity	Total bicycle intensity	Percentage of motor vehicles on main road	Percentage of bicycle on main road	Percentage of left turning traffic motor vehicles	Percentage of left turning traffic bicycles	Number of traffic demand scenarios
Levels per variable	5	5	2	2	2	2	400

Table 5: Overview of the used traffic demand variables and the total number of traffic demand scenarios

The values that are used for the traffic demand variables are displayed in Table 6 below. The motor vehicle intensity values are chosen to give a range from a relatively low value to just beyond the theoretical capacity of a single lane roundabout, at 2.700 PCE/hour (CROW, 2008). The bicycle intensity range is based on common values observed at different junctions in Enschede (Veenstra, Thomas, & Geurs, 2013). For the main-/ side road distribution two values between an equal distribution, i.e. 50% main road traffic, and the unrealistic maximum of 100% main road traffic are chosen. For efficient operation the SGFC-junction should be applied in locations with at least 10% left turning bicycle traffic (Eggen, Salomons, & Zeegers, 2003). The left turn shares are chosen below and above this value to check if this threshold is also decisive in comparison with the roundabout. Appendix H describes the decision making behind the traffic demand variable levels in more detail.

The values used for the traffic demand variables						
	Traffic demand variables					
	Total motor vehicle intensity [mv/h]	**Total bicycle intensity [b/h]**	**Percentage of motor vehicles on main road [%]**	**Percentage of bicycle on main road [%]**	**Percentage of left turning traffic motor vehicles [%]**	**Percentage of left turning traffic bicycles [%]**
Level values	1000	200	60	60	5	5
	1500	600	80	80	15	15
	2000	1000				
	2500	1400				
	3000	1800				

Table 6: The values used for the traffic demand variables

Several other factors need to be defined as well in order to fully describe the traffic demand scenario. These factors remain constant over all of the simulated demand scenarios because making them variable would lead to an unfeasibly great amount of traffic demand scenarios.
The first factor is the right turn share. This is set at 10% for both motor vehicles and bicycles. The second factor is the composition of motor vehicle traffic. It consists of four different vehicle types: cars, vans, lorries and articulated lorries. The shares of these types in the total number of motor vehicles is set in accordance with the video observation results. Finally the mean headway and headway distribution are also based on the observed values. These values are presented in paragraph 3.3 about the simulation run setup.

2.7. Set-up of the simulated junctions

Paragraph 1.3 has already defined which junction types are studied. This paragraph presents how these junction types are implemented in this study. Firstly the common characteristics of the junction networks are presented. Then each of the two junction types is discussed in more detail.

For both junctions the same approaches are modelled. These have a distance of 350 meters measured from the centre of the junction. These roads are modelled as distributor roads and are modelled with a speed limit of 50 km/h and a lane width of 3.2 meters in accordance with the guidelines (CROW, 2012). The bicycle paths are separated from the roadway, and are modelled at a width of 2,00 meters again in accordance with the CROW recommendations (CROW, 2012).

The roundabout is modelled following the CROW recommendations (CROW, 2012). In accordance with the recommendations there is a distance of 5 meter between the roundabout circulatory lane and the bicycle path, so that there is space for a car to give way to bicycles without blocking the roundabout. In addition the bicycle paths leaving the circulatory bicycle path disconnect as soon as far away from the next approach so that the probability that motor vehicles wait unnecessary on bicycles that turn out to leave the roundabout is low. Figure 4

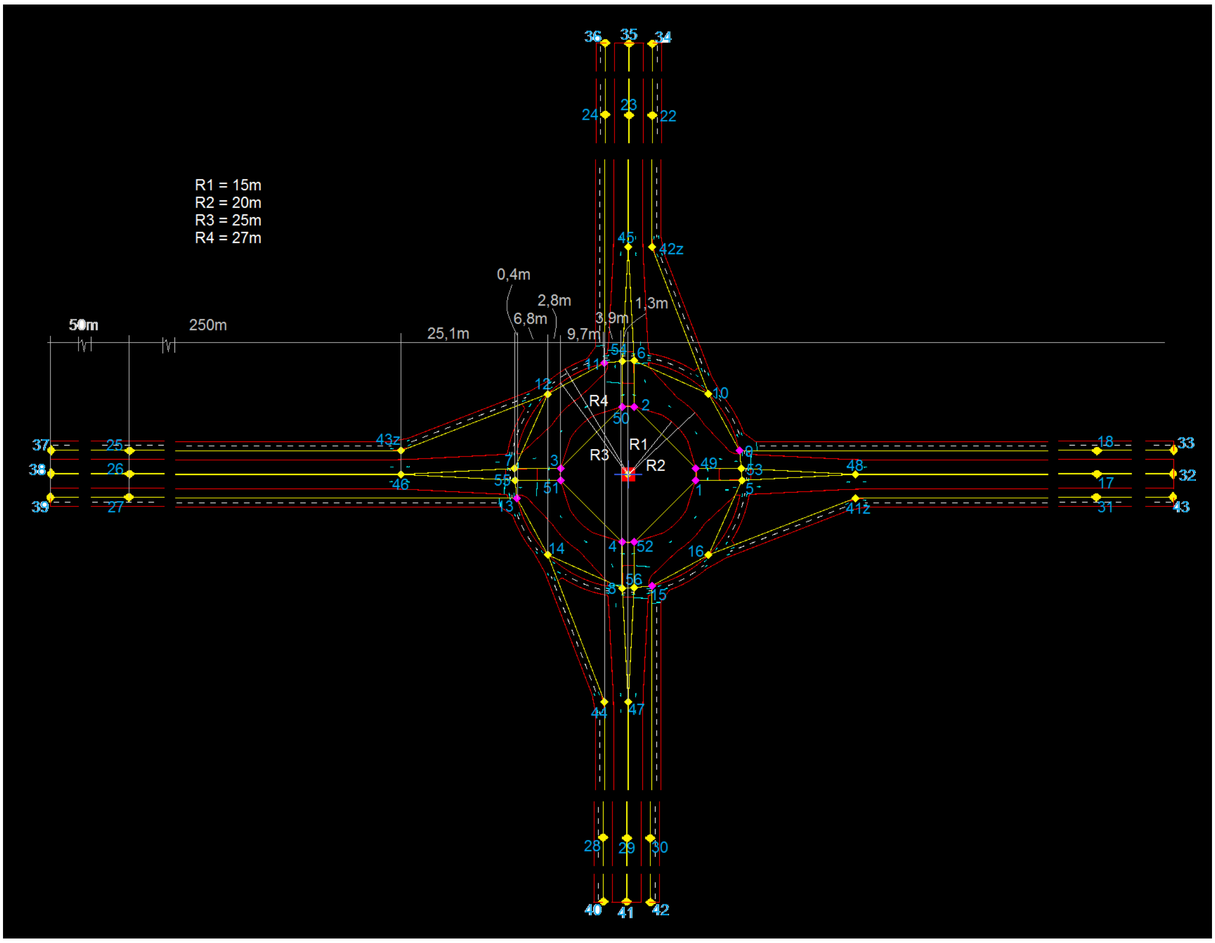

Figure 4: Overview of the roundabout as it is modelled in this study

The SGFC-junction is also modelled mostly in accordance with CROW recommendations (CROW, 2012). Most notable difference is that the bicycle lane is modelled three meters wide instead of two. This is to ensure that the bicycle outflow capacity during each green stage is high enough. The junction overview in Figure 5 shows that the main road (from left to right) has three lanes on the final junction approach and the minor road (from top to bottom) has one lane. The dedicated turn lanes are 50 meters long, measured from the centre of the junction. The amount of turn lanes depends on the traffic demand scenario . The size of the junction crossing area depends on the amount of turn lanes used.

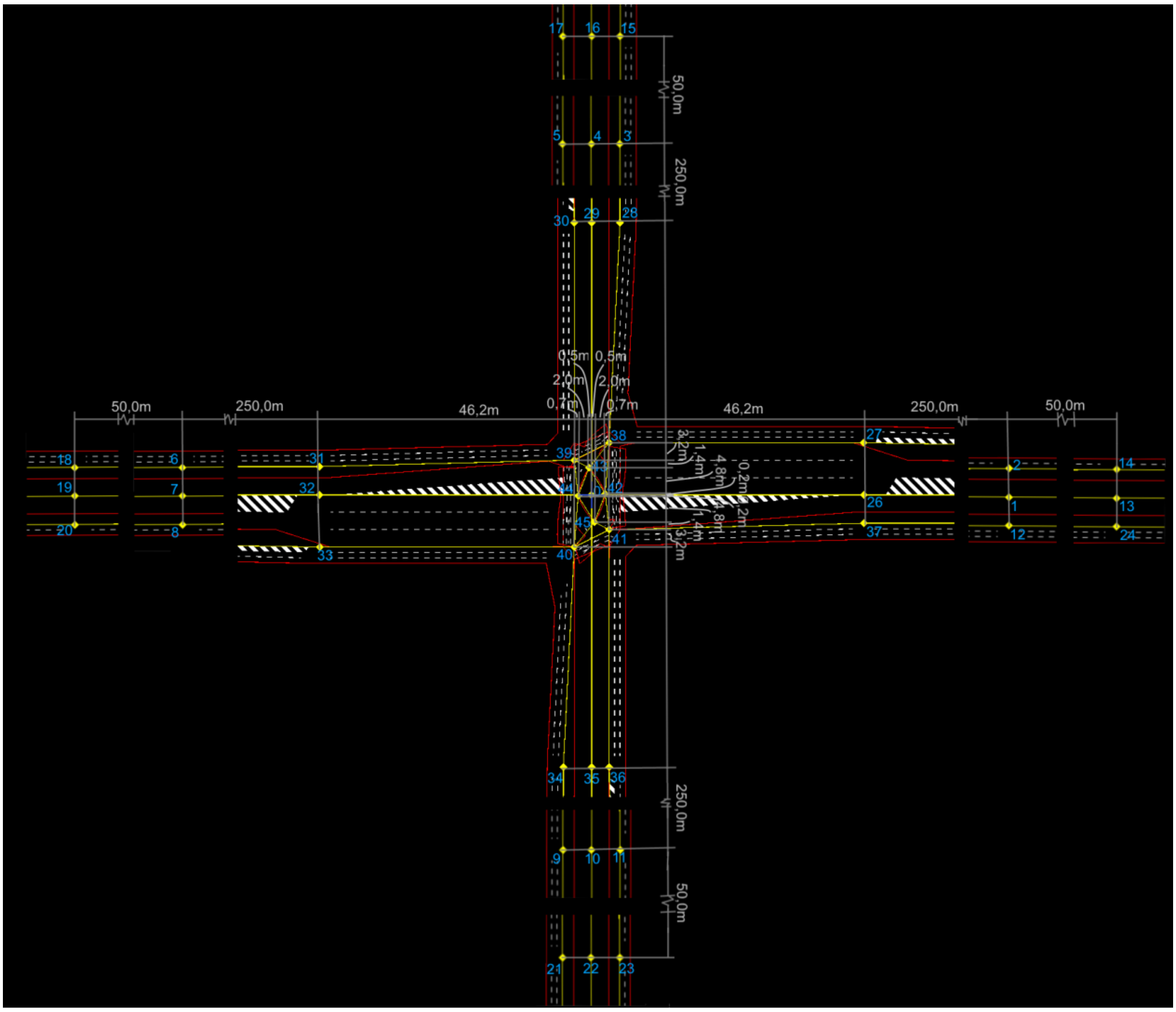

Figure 5: An overview of how the SGFC-junction is modelled in this study

The amount of turn lanes at the SGFC-junction is determined based on CROW design recommendations (CROW, 2012). This recommendation is represented in Figure 6 below (CROW, 2012). The letters I_a, on the x-axis, and I_o, on the y-axis, represent the intensity of the trough traffic. Each of the lines represent the percentage of left turning traffic from the total flow of I_a, denoted by L. If the point in the graph described by the two through traffic intensities lies to the right/above the line with the appropriate percentage of left turning traffic than a dedicated left turn lane is recommended. When this guideline recommends a left turn lane for a certain traffic pattern then it is applied in the simulation. The same guideline is applied to determine if a dedicated right turn lane is implemented.

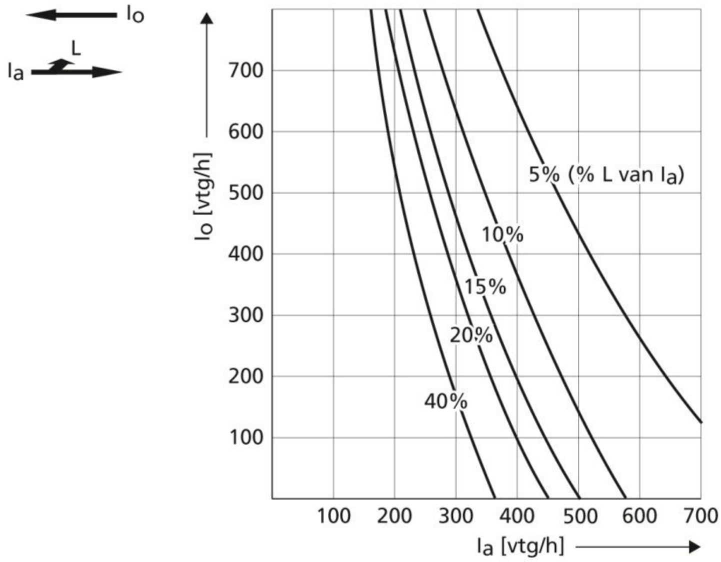

Figure 6: Design recommendation about left turn lanes. If the intersection point of the two through traffic flows is above/right of the line with the respective left turn percentage a dedicated line is needed.

Table 7 below shows which turn lanes, if any, are used for certain groups of traffic demand scenarios.

Overview of the used turn lane configuration over the different traffic demand scenarios					
Main road and left turn share combination	**Motor vehicle intensity**				
	1000	**1500**	**2000**	**2500**	**3000**
60% main road, 5% left turn	- - - -	- - - -	Left turn major Right turn major - -	Left turn major Right turn major - Right turn minor	Left turn major Right turn major Left turn minor Right turn minor
60% main road, 15% left turn	- - - -	- - - -	Left turn major Right turn major - -	Left turn major Right turn major Left turn minor -	Left turn major Right turn major Left turn minor Right turn minor
80% main road, 5% left turn	- - - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -
80% main road, 15 % left turn	- - - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -

Table 7: Overview of the used turn lane configuration over the different traffic demand scenarios

Appendix A gives a more detailed description of how both junction types are modelled in this study.

3. Preparation of the simulation model

This chapter describes all the steps that need to be taken before the simulation runs can be executed. Firstly paragraph 3.1 describes the choice of a traffic simulation software package and the effect models. After the simulation software is chosen the software has to be calibrated to ensure that it correctly represents the conditions at existing SGFC-junctions and roundabouts. After calibration the simulation software is validated against a different dataset. Paragraph 3.2 gives an overview of the process of how data is collected through camera observations, how the model is calibrated and how it is validated. Paragraph 3.3 describes the final steps that need to be taken before the model simulations can be run: determining the traffic demand scenarios to be investigated, the number of runs per situation and the simulated time per run.

3.1. Simulation software package

In paragraph 2.4 the type of simulation software to be used was already constrained to a micro simulation model. This paragraph explains why the S-Paramics 2011 micro simulation software is chosen for this research.

The comparison of the two bicycle friendly junctions results in several requirements for the used the simulation software. Especially the safety assessment requires specific model characteristics. In 2005 Archer highlighted the following model characteristics necessary for traffic safety modelling (Archer, 2005):

- the exact geometric layout of the road network has to be represented (lane widths, traffic islands, stop lines, etc)
- accurate representation of traffic signal control strategies including vehicle actuated signalling and co-ordinated signalling
- the interaction between vehicles and other classes of road users
- the precise representation of traffic flows, turning movements and traffic composition over time and specific to links
- accurate levels of speed and speed variation over time, and specific to certain links and turning manoeuvres.
- differences in vehicle characteristics between and within specific vehicle classes for example: length, weight, engine-power and braking and acceleration ability
- differences in behaviour and performance between and among different classes of road-users for example desired speed, headway and gap-acceptance

The arguably two most used micro-simulation tools in Europe are VISSIM and Paramics (Archer, 2005). Both of these software packages are capable of representing a road network in great detail. Both software packages enable the user to create their own signal control logic, which can incorporate information from the simulated traffic state via loop detectors. Both models allow the user to define different vehicle types with different characteristics, see (PTV Group, 2014) and (SIAS Limited, 2011). With a certain stochastic element both software packages enable the user to define a distribution of vehicle characteristics and driver behaviour within one vehicle type.

Because the S-Paramics software is able to meet the requirements and because it is available at the University of Twente this program is used. For this research S-Paramics version 2011.1 is used.

3.1.1. Modelling bicycles in Paramics

Bicycling is not included in Paramics by default. It is however still possible to simulate bicycling in Paramics provided that bicycles have their own dedicated infrastructure (Bertini, Lindgren, &

Tantiyanugulchai, 2002). Both junction design types investigated in this research have dedicated bicycle lanes so it is therefore possible to use Paramics simulation software.

Because bicycling is normally not included in Paramics it is necessary to create a new road section type and new vehicle type. The dimensions of the bicycle paths are taken from the 'design manual for traffic facilities in urban areas' (CROW, 2012). For uni-directional bicycle paths along distributor roads the minimum separation between the road and the bicycle path should be 0,35 meter. The minimum width of the bicycle path itself should be 2,00 meters.

To be able to simulate bicycles a new vehicle type has to be created in Paramics. For a vehicle type the physical properties and the dynamic capabilities have to be defined by the user and by implementing these appropriately it is possible to create bicycles (SIAS Limited, 2012). For road design purposes the CROW has defined the physical dimensions of a representative standard Dutch bicycle (CROW, 2012). The bicycle length is set at 1,95 meters, the width is 0,64 meters. The bicycle length value is directly inserted into Paramics. The value for width is adjusted to compensate for the sideways oscillation of bicycles. An important dynamic characteristic of bicycles is the sideways oscillation, so perpendicular to the direction of travel. This oscillation is necessary to balance the bicycle. Under normal weather conditions and at a cycling speed of 20 km/h this oscillation is 0,20m (CROW, 2006). It is not possible to set this oscillation factor in Paramics. On order to replicate this effect the width of the bicycle is increased. The bicycle width is set at 0,75m.

For the dynamic capabilities the desired speed, maximum acceleration and maximum deceleration have to be set in Paramics. For the probability distribution of the desired travel speed for cyclists in the Netherlands the 5th percentile is at a speed of 13 km/h and the 95th percentile lies at 16km/h (CROW, 2006). The average desired speed is therefore set at 14,5 km/h. The CROW states that for an acceleration from standstill the value lies between 0,8 m/s^2 and 1,2 m/s^2 (CROW, 2006). For this research a value of 1,0 m/s^2 will therefore be used. Regarding deceleration the CROW states a value of 1,5 m/s^2, for comfortable deceleration, and 2,6 m/s^2 for emergency braking. Because only the maximum deceleration can be set in Paramics a value of 2,6 m/s^2 is used.

3.1.2. Traffic safety module

Paragraph 2.5.1 explained that traffic safety is measured in the number of TTC conflicts. The SWOV has developed a module that calculates TTC values. This module uses output from micro simulation software to calculate the amount of times the Time To Collision is below a critical value. The input for this module consists of the location and the speed of each vehicle in the network for each simulation time step in Paramics. With this information the module can calculate the distance between adjacent vehicles and based on their speed differences it calculates the TTC. It can also calculate the TTC for vehicles that approach the conflict points on a junction. So it is not necessary that two vehicles are on the same link in order to calculate the TTC.

3.1.3. Emission effect model

Because the Paramics software itself does not generate output regarding emissions it is necessary to use a separate software module to evaluate junction performance regarding emissions. The AIRE (Analysis of Instantaneous Road Emissions) software module is specifically developed to estimate vehicle emissions based on micro-simulation output (Transport Scotland, 2011). The software uses three types of input: data about traffic flow from the Paramics simulation, data on the composition of the vehicle fleet and data about the emissions of different vehicles types (SIAS Limited, 2010). Each of these three are further explained in the paragraphs below.

The input data from Paramics simulations that the AIRE software uses consists of the speed and rate of acceleration of each individual vehicle per time step of the Paramics simulation, by default half a second (SIAS Limited, 2011). Stop and go traffic or driving at a slow but constant speed can result in the same average speed on a link. The realised emissions in the latter traffic state are however much lower than those realised in the former state. Because of the sampling interval of only 0,5 seconds AIRE is capable of taking the traffic state into account contrary to emission models based on average link speeds.

The fleet composition data is divided into four different sections: Fuel Split, Split by Vehicle Load, Split by Size and Split by EU Emission Standard. The fuel split describes per vehicle type the distribution of diesel and petrol powered vehicles. The split by vehicle load describes what percentage of vehicles drives around empty or fully loaded. Especially for lorries this has an influence on the emissions. The split by size describes different vehicle types by weight class / vehicle type. Examples are passenger cars, light commercial vehicles (vans), light goods vehicles (small lorries) and heavy goods vehicles (lorries). The split by EU emission standard describes the distribution of vehicles over the different emission norms. This is based on the construction year of the vehicle. For this study the vehicle fleet composition is based on the total amount of kilometres travelled of different vehicle types and vehicle ages found in CBS datasheets (CBS, 2014), (CBS, 2014) and (CBS, 2014).

The emissions per vehicle type lists average emissions for over 3.000 combination of the previously mentioned vehicle type variables and the driving variables. The average emission values are based on the output of the European Commission's Fifth Framework project ARTEMIS (SIAS Limited, 2010). This research program created emission models for different vehicles driving in Europe by measuring vehicle emissions for several standardised driving cycles (André, Keller, Sjödin, Gadrat, Mc Crae, & Dilara, 2009).

The AIRE software gives output about three different emissions: oxides of nitrogen (NO_x), particulate matter (PM_{10}) and total carbon (CO_2 equivalent = CO_2e) (SIAS Limited, 2010). These emissions can be summarised per vehicle trip, per link or for the complete network.

3.2. Calibration and validation of the traffic simulation model

To increase the reliability of the S-Paramics outputs the simulation model is calibrated with data gathered from camera observations. The purpose of the model calibration is to adjust the simulation model to accurately represent the behaviour of Dutch road users. The values for certain behaviour model parameters can be directly measured in the street, for example the mean headway between motor vehicles. Other model parameters, such as visibility, cannot be measured. The calibration process ensures that these parameters are set at values representative for Dutch traffic. The calibration process is divided into four steps: conducting the camera observations, analysing the video recordings, the calibration of the S-Paramics models and the validation of the S-Paramics models. This paragraph gives an overview of these steps.

3.2.1. Set up of camera observations

The camera observations are done to record the typical behaviour of road users at both junction design types. For each type one existing location is observed. Keypoint Consultancy B.V. provides the camera's for the observations. These camera's are mounted on lamp posts. This higher vantage point gives a good overview of the junction area and reduces the risk that the view is obstructed by passing lorries or vans. The cameras take a picture every second.

For the SGFC-junction type observations are conducted at the crossing of the Oldenzaalsestraat, the Lasondersingel and the Laaressingel in Enschede. An overview of this location including the placement of the cameras is presented in Figure 7 below.

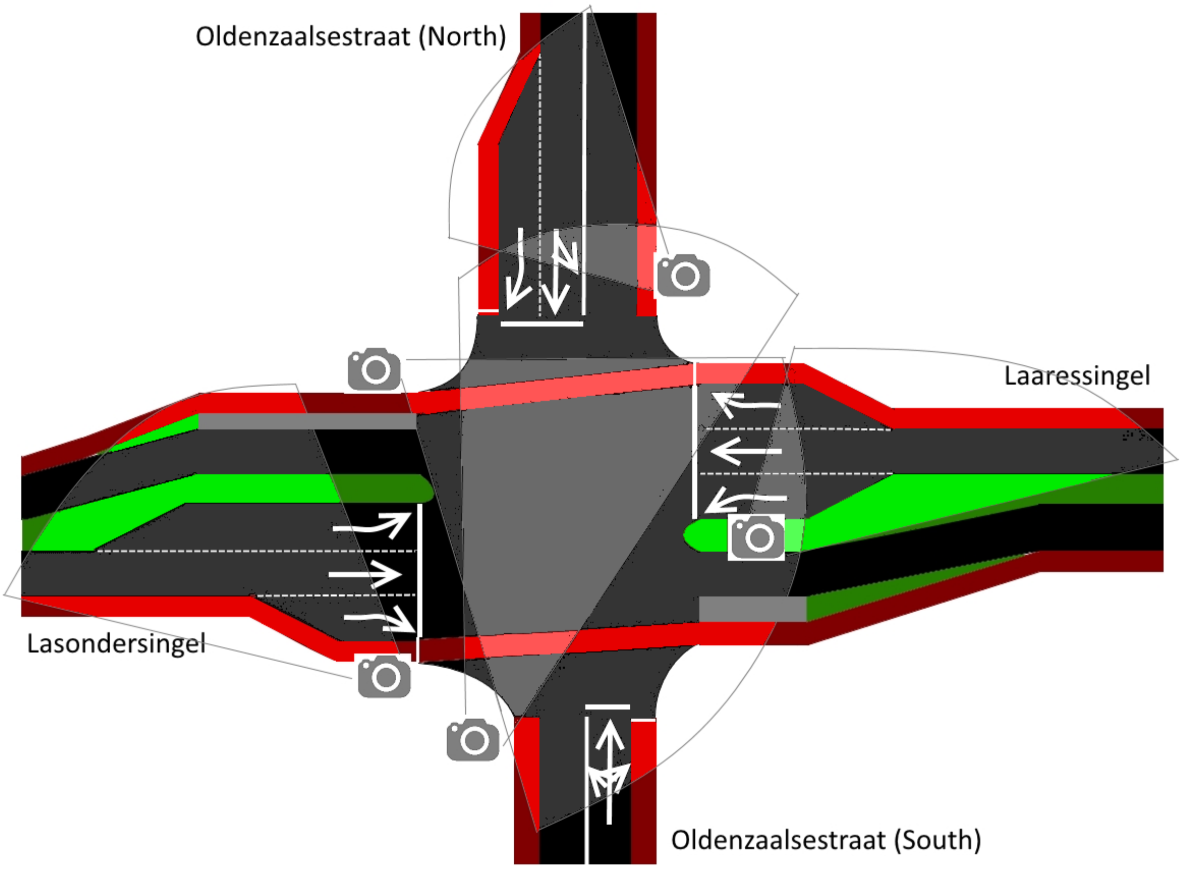

Figure 7: Schematic overview of the observed junction layout and camera placement

For the roundabout observations previously recorded for another research are used (Olijve, 2014). The used camera has a frame rate of 25 fps. These observations were taken at the intersection of the Europaweg, Admiraal Helfrichstraat and Bruchterweg in Hardenberg. A schematic overview of the observed location and camera placement is given in Figure 42 below.

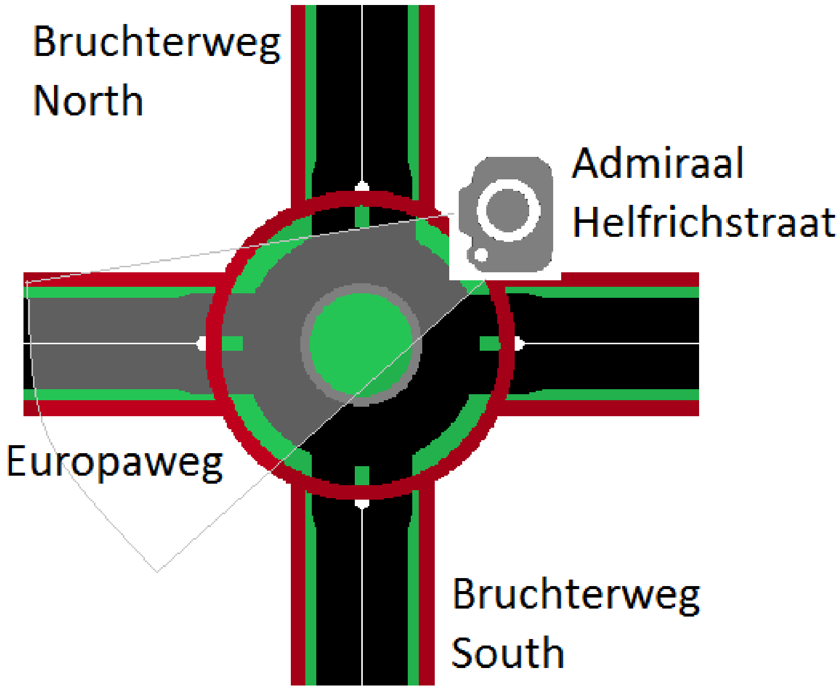

Figure 8: Schematic overview of the observed roundabout and camera placement

In order to determine the typical behaviour of road users at the junctions it is necessary to take sufficiently long observations. Therefore the observation interval is set at one hour. For the roundabout observations the observation interval had already been set at 50 minutes. To be able to validate the simulation model, data for different traffic conditions are needed. Therefore one observation from the peak period and one observation from the off peak period is analysed. More details about the camera observation set up are in appendix C.

3.2.2. Analyses of the camera observations

The analysis of the images will yield several types of data. The first type of data gives the values at which different user controlled traffic simulation model variables have to be set. Especially for variables related to bicycle traffic it is important to use observations because bicycle traffic is not included by default in simulation models. Therefore there is an extra risk that the default values of these variables differ from real conditions.

The other type of data is used for the model calibration process. This type is subdivided into model input and comparison observations for model output. The model input comprises of several data types:

- junction geometrical layout describing e.g. lane widths, lengths of dedicated turn lanes and dimensions of the crossing area
- specifically for the SGFC-junction the general signal control logic
- the traffic demand specified per turn direction, vehicle type and five minute interval
- the gap distribution of arriving traffic
- estimation of the critical gap

The model output comparison data consists of:

- The maximum queue length per five minute interval in number of vehicles
- The minimum queue length per five minute interval in number of vehicles

A detailed description of how the data types mentioned in this paragraph are extracted from the video observations are presented in appendix D. The results of the camera observations are presented in appendix E. The traffic demand profiles, gap distributions and critical gap estimation presented there are used as model input in the calibration process.

3.2.3. S-Paramics calibration

This paragraph gives an overview of the calibration process and presents how well the calibrated simulation models match the observations.

The goal of the calibration process is to make the simulated junction capacity correspond as closely as possible to the observed junction capacity. This is done by matching the simulated queue lengths with the observed queue lengths. For each five minute interval the minimum and maximum queue length are compared. In preparation for the calibration the S-Paramics model is set up according to the model input data extracted from the video observations. This way the starting conditions are similar to the observed situation. This means that the observed model input dattatraffic intensities per vehicle type, per direction per five minute interval are

If the ending conditions, in the form of the simulated queue lengths, are similar to the observations the simulated junction has handled traffic similar to the existing junction meaning that the model accurately represents the real junction. For both of the Paramics models the calibration is done based observations during a peak hour.

During the calibration process some model parameters are adjusted to increase the similarity between the observed queue lengths and the simulated queue lengths. In this case two model parameters are adjusted:

- Sightline
- Gap acceptance

The parameter sightlines in Paramics defines from how far away drivers can see other, potential conflicting, vehicles coming. This variable therefore impacts at what time drivers are able to detect a gap in the conflicting traffic flow and thus indirectly influences gap acceptance. The larger the sightline distance the sooner vehicles can detect a gap in the conflicting traffic flow increasing the probability that a potential gap will be used and thus influencing junction capacity.

The second variable is the gap acceptance. The gap acceptance behaviour has a big effect on roundabout capacity. The smaller the average gap in the roundabout traffic flow that drivers on the approach accept the more suitable gaps will occur and thus the higher the roundabout capacity. Even though the traffic lights separate several conflicting traffic flows in time there are still some conflicts left. In those cases the gap acceptance determines what time gap between conflicting vehicles is big enough to proceed and thus influences traffic flow and junction capacity.

To be able to define if the simulation outputs are similar enough to the observations a statistical test has to be used. For each model setting twenty runs are executed and the maximum queue lengths of these runs are compared to the observations. If the statistical test cannot prove that the simulated values differ significantly from the observed values than the model resembles the observations close enough meaning that the calibration is successful. Appendix F describes the calibration process in greater detail.

To prove that the roundabout model is successfully calibrated the Kolmogorov-Smirnov (K-S)test is used to compare the simulation queue lengths with the observed queue lengths in each five minute interval. For every interval the K-S test statistic lies below the threshold value. This means that the K-S test cannot prove that the simulated maximum queue lengths differ significantly from the observed values. Therefore the model calibration is successful. The K-S test results are presented in G.

For the calibration of the SGFC-junction the three observed approaches are individually compared to the simulations. So for each five minute interval three comparisons are made, instead of just one in the roundabout calibration process. Another difference is the used statistical test. Because the queue length data over the twenty runs was normally distributed the two sample t-test is used instead of the Kolmogorov-Smirnov test. For each of the three junction arms and over all five minute intervals the test statistic is below the thresholds value. The calibration is therefore successful.

Appendix G presents the calibration process and results in more detail.

3.2.4. S-Paramics validation

Both of the Paramics junction models are validated against observations done during an off peak hour. The comparison between observed queue lengths and simulated queue lengths is done in the same way as in the calibration process. For the roundabout the K-S test indicates that for each five minute interval the simulated maximum queue length matches the observed maximum queue length close enough. For the SGFC-junction the t-test indicates that for each of the three approaches over all the five minute intervals the simulated maximum queue length matches the observed maximum queue length close enough. For both the junction models the validation is therefore successful. Appendix G presents the validation process and results in more detail.

3.3. Set up of the simulation runs

The final step of the simulation model preparations is the set up of the simulation runs. As already noted in paragraph 2.1 several things need to be defined:

- How much time is simulated per run
- Definition of traffic demand scenario constants
- How many runs per demand scenario are simulated

For each simulation run traffic demand is defined for one hour. This will allow for shorter simulation times and eliminates the need to construct a traffic demand pattern over the course of a day. Because a wide range of hourly traffic intensities is simulated road authorities can still create a realistic intensity distribution over day matching their specific junction location by combining the hourly simulation results into a longer time period.

At the end of the simulated hour there will still be vehicles in the network. Differences in the number of vehicles still left in the network can be important for evaluating the junction design types. For instance the amount of congestion that is still left in the network at the end of the simulation period would increase the total time lost for a certain junction design if the simulation would have run longer. Two options to deal with this problem are: let the simulation run longer until all congestion has dissolved or estimate the not captured residuals by estimating how long it would have taken vehicles to leave the network based on queue length and road capacity (Dowling, Holland, & Huang, 2002). In this research the first option is applied; In total two hours are simulated. Because there is only a traffic demand in the first hour the additional simulated hour ensures that the simulation is kept running until all vehicles have left the network. This is in line with the recommendation that is made in the paper by (Dowling, Holland, & Huang, 2002). No new vehicles are entering the network after the first hour, unless these vehicles could not be released onto the network previously because it was completely congested. This second hour serves as a cool down period.

Measurements of junction performance take place over the complete two hour period. This means that there is no warm up period at the start of the simulation in which the empty network is gradually filled with vehicles. This may reduce the accuracy of the results because regardless of the traffic intensity the first measured vehicles always travel over an (almost) empty network. Because the used simulation network is small, it is filled with vehicles quickly and therefore this only has a very small impact on the simulation results.

The variables describing the traffic demand scenarios are already explained in paragraph 2.6. Other factors that describe the behaviour of road users are defined based on the camera observations. These are: the shares of different types of motor vehicles in the total group of motor vehicles, the mean headway and headway distribution and critical gap.

Share of different motor vehicle types			
Motor vehicle type	**Junction type**		**Used distribution**
	Roundabout	**SGFC-junction**	
car	84.490%	89.938%	**87.214%**
van	10.612%	0.000%	**5.306%**
lorry	3.878%	0.588%	**2.233%**
articulated lorry	1.020%	0.284%	**0.652%**
motorbike	0.000%	0.521%	**0.260%**
Total	**100.000%**	**100.000%**	**95.665%**

Table 8 below shows the used motor vehicle type distribution. It is the average of the observed distributions at the roundabout and the SGFC-junction.

Share of different motor vehicle types			
Motor vehicle type	Junction type		Used distribution
	Roundabout	SGFC-junction	
car	84.490%	89.938%	**87.214%**
van	10.612%	0.000%	**5.306%**
lorry	3.878%	0.588%	**2.233%**
articulated lorry	1.020%	0.284%	**0.652%**
motorbike	0.000%	0.521%	**0.260%**
Total	**100.000%**	**100.000%**	**95.665%**

Table 8: Share of different motor vehicle types used for all traffic demand scenarios

Table 9 below shows the used mean headway. It is the average of the observed distributions at the roundabout and the SGFC-junction.

Share of different motor vehicle types			
	Junction type		Used mean headway
	Roundabout	SGFC-junction	
Motor vehicles	3.35	3.69	**3.52**
Bicycles	3.07	3.32	**3.20**

Table 9: Used mean headways

Table 10 below shows the used headway distribution. In S-Paramics this is integrated in the aggression distribution parameter. The used distribution is the average of the observed distributions at the roundabout and the SGFC-junction.

Distribution of motor vehicle traffic over different gap sizes excluding gaps > 9.5 s									
Arms	**Relative importance of bins**								
Bin lower bound [s]	**0.5**	**1.5**	**2.5**	**3.5**	**4.5**	**5.5**	**6.5**	**7.5**	**8.5**
Bin higher bound [s]	**1.5**	**2.5**	**3.5**	**4.5**	**5.5**	**6.5**	**7.5**	**8.5**	**9.5**
Roundabout	21%	28%	16%	8%	9%	5%	5%	4%	4%
SGFC-junction	23%	19%	8%	7%	19%	4%	4%	4%	11%
Used distribution	**22%**	**24%**	**12%**	**8%**	**14%**	**4%**	**5%**	**4%**	**7%**

Table 10: Used distribution of motor vehicle traffic over different gap sizes

The S-Paramics simulation software is a stochastic program. This means that even with the same input the output from one run may differ from the output of a previous run. The software uses random number generators in both the releasing of vehicles onto the network as well as in the simulation of vehicle behaviour (SIAS Limited, 2012). The stochastic element is added to mimic the variation in behaviour of real world driver vehicle combinations. Both the interval between vehicle releases as well as the vehicle type released are subject to randomised functions. Naturally the probability distribution is set up to meet the average vehicle intensity per type that the user specified. However small variations from the specified number will occur. The vehicle behaviour, which for instance simulates decision making on overtaking or gap acceptance at junctions, contains stochastic parts as well.

These stochastic influences mean that a single run of a network is not necessarily representative of typical traffic conditions (SIAS Limited, 2012). Therefore it is necessary to conduct multiple runs of each situation and use the average results of these results for model calibration and analyses. For this research the number of runs per traffic demand scenario is set at 10. The results in chapter 4 show that for the vast majority of the comparisons the difference between the junction types is statistically significant. Therefore the amount of 10 runs per demand scenario is enough.

4. Analyses of simulation results

This chapter presents the simulation output from S-Paramics as well as the output from the SWOV TTC modules and AIRE. The first three paragraphs present and analyse the results regarding throughput, safety and emission respectively. The following section provides an analysis of the combination of results from the three performance fields.

For each junction type $5_{motor\ vehicle\ intensity\ levels} \times 5_{bicycle\ intensity\ levels} \times 2_{motor\ vehicle\ main-side\ road\ levels} \times 2_{bicycle\ main-side\ road\ levels} \times 2_{motor\ vehicle\ left\ turn\ levels} \times 2_{bicycle\ left\ turn\ levels} = 400$ traffic demand scenarios are simulated. Presenting all results would therefore require at least one table with 400 rows. The size of these tables makes it difficult to get an understanding of which junction type scores best at which traffic demand scenarios. Therefore not all results are presented in this report. Furthermore to make it easier to deduce the relative performances of the two junction types the results are presented in graphs instead of tables. Because it is unfeasible to present the effects of all six traffic demand variables in one graph, multiple graphs are required. Given that multiple graphs are needed, two dimensional graphs are chosen because they offer a good readability. Each graph has the same layout.

- The y-axis shows the junction performance, for instance the average travel time in seconds
- The x-axis displays the first traffic demand variable, which is always an intensity variable because it has five levels
- Two lines represent the SGFC-junction, coloured different shades of blue
- Two lines represent the roundabout, coloured different shades of red
- For each junction type the impact of the second traffic demand variable is represented in the difference between the darker and lighter shaded line

With this graph layout five different types of information can be extracted from each graph. These types of results and the way they are represented in the graphs is indicated in Table 11 below. Table 11 also shows the order in which the result types are discussed and if the results of a certain type are statistically tested.

Types of results extracted from each graph			
Order	**Result type**	**Manner of visualisation**	**Statistical test**
1	Impact of junction design on junction performance	Differences between the dark blue and the dark red line and the light blue and light red line	Yes
2	Impact of second traffic demand variable on SGFC-junction	Differences between the dark blue and light blue line	No
3	Impact of first traffic demand variable on SGFC-junction	Paths of the individual blue lines	No
4	Impact of second traffic demand variable on the roundabout	Differences between the dark red and light red line	No
5	Impact of first traffic demand variable on the roundabout	Paths of the individual red lines	No

Table 11: The different types of results and the order and way in which they are extracted from the result graphs

Table 11 indicates that only the results regarding the impact of junction design are submitted to a statistical test. Because this is the result type needed to answer the research questions it is necessary to submit them to a statistical test. The other result types give an insight in how traffic demand variables impact the performance of an individual junction type. These results are noteworthy since a change in junction performance can change the outcome of the final comparison between the two junction designs. It can also direct further research because if one traffic demand variable turns out to hardly impact junction performance other studies can exclude this variable.

For the comparison between the performances of the two junction designs either the Mann-Whitney u-test (M-W u-test) or the two sample t-test are used. The two sample t-test is preferred because it generally has a bigger statistical power than the M-W u-test (Lehmann, 1999). An important prerequisite of the two sample t-test is that the data in both samples is normally distributed. To clarify: the data of the ten different runs per traffic demand scenario and junction type should be normally distributed around the average over these ten runs. The normality of the data is checked with the Shapiro Wilk (SW) test for normality. In case the data for both junction types is normally distributed, the next step is to determine if the variances of the two data sets is equal. The equality of the variances determines which variant of the two sample t-test is used. The F-test for equal variances is used to determine if the variances of the two samples are similar. In case the data is not normally distributed the M-W u-test is used to compare the junction performances. The statistical tests comparing the two junction designs are all conducted at a 0,05 significance level. Appendix J shows the Matlab script that is used to automate this process of comparison between junction types.

Because each graph can illustrate the effect of two traffic demand variables, the other four variables have to remain constant to ensure fair comparisons of the results in the graphs. The levels at which these variables are set should ideally be an average value because then the graph shows the average impact of the traffic demand variables that are included in the graph. However the main-/ side road distribution and left turning share variables only have two levels and therefore no average value. For these variables the value that leads to the highest amount of potential conflicts is chosen because it is more likely to expose the differences between both junctions. Table 12 below shows the default levels for each traffic demand variable in case they are not included in a graph.

Values for traffic demand variables not included in a graph		
Traffic demand variable type	Mode type	
	Motor vehicles	Bicycles
Traffic intensity	2.000	1.000
Main-/ side road distribution	60	60
Left turn share	15	15

Table 12: Default values for traffic demand variables not displayed in a result graph

Because the great number of simulated traffic demand scenarios it is not feasible to present all results in the report. Appendix I demonstrates that the impact of the variables bicycle main-/ side road distribution and bicycle left turn share is small. Therefore graphs for these variables are not presented in this chapter. In addition appendix 15 demonstrates that the separate effects of the motor vehicle main-/ side road distribution and the motor vehicle left turn share are relatively small. Therefore the combined effect of these variables is expected to also be small and not interesting to show in this chapter. Table 13 below gives an overview of the traffic demand scenarios for which the results are presented in this chapter.

Variable displayed with different lines		Variable on the x-axis					
		intensity		main-side road distrib.		left turning share	
		motor v	bicycle	motor v	bicycle	motor v	bicycle
intensity	motor vehicle	x	x	x	x	x	x
	bicycle	1	x	x	x	x	x
main-side road distrib.	motor vehicle	2	3	x	x	x	x
	bicycle	4	5	6	x	x	x
left turning share	motor vehicle	7	8	9	10	x	x
	bicycle	11	12	13	14	15	x

Table 13: Traffic demand scenarios for which results are displayed in this chapter.

Table 13 shows that graph types 1, 2, 3, 7 and 8 are presented in this chapter. The result graphs are always presented in the same order of traffic demand scenario variables over the different junction performance indicators

- motor vehicle intensity on the x-axis and bicycle intensity in the lines (graph type 1)
- motor vehicle intensity on the x-axis and motor vehicle main-/ side road distribution in the lines (graph type 2)
- bicycle intensity on the x-axis and motor vehicle main-/ side road distribution in the lines (graph type 3)
- motor vehicle intensity on the x-axis and motor vehicle left turn share in the lines (graph type 7)
- bicycle intensity on the x-axis and motor vehicle left turn share in the lines (graph type 8)

4.1. Results concerning travel time

The average travel time per vehicle is used to compare the two junction designs regarding throughput. This is the time it takes a vehicle to get from their point of entry into the network to their point of exit out of the network which is about 625 meters. For comparison: the free flow travel time through the network for a motor vehicle, assuming the speed limit of 50 km/h, is $\frac{625}{(50 \div 3.6)} = 45$ seconds. For a bicycle, assuming the desired speed of 14 km/h, the free flow travel time is $\frac{625}{(14 \div 3.6)} = 161$ seconds. The following sub-paragraphs each discuss the impact of two of the variables used to describe the traffic demand scenarios. Each sub-paragraph first shows a graph with the results regarding average travel time for motor vehicles followed by a graph with the average travel time for bicycles.

4.1.1. Impact of motor vehicle intensity and bicycle intensity

For this comparison the motor vehicle intensity and bicycle intensity are variable. The share of traffic allocated to the main road is kept at 60% for both motor vehicles and bicycles for all these simulations. The share of left turning traffic is set at 15% for both modes. The average travel times for motor vehicles over different motor vehicle and bicycle intensities are shown in Figure 9 below.

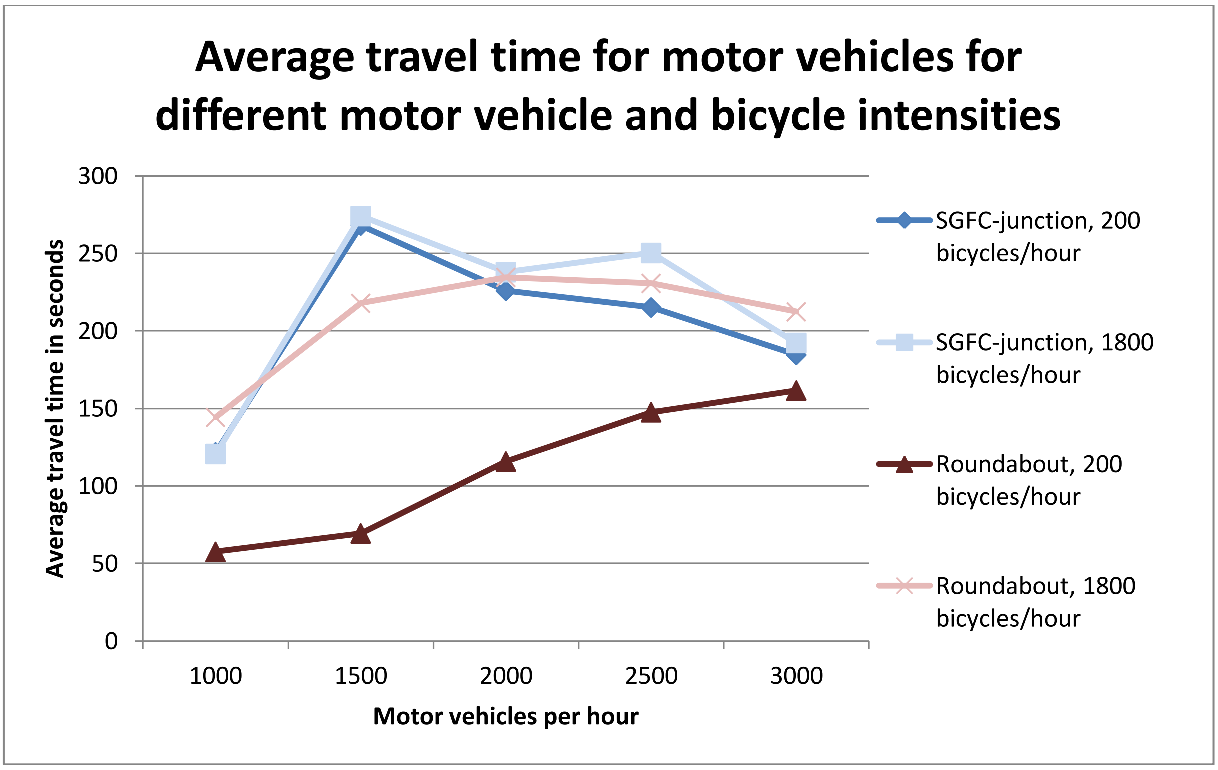

Figure 9: Comparison of average travel time for motor vehicles over different motor vehicle and bicycle intensities and the two junction types.

SGFC-junction and roundabout comparison at 200 bicycles per hour
For all these traffic demand scenarios the simulation data is normally distributed therefore the two sample t-test is used to check if the performances of the SGFC-junction and the roundabout are significantly different. The comparison between the SGFC-junction and the roundabout for the demand scenarios with 200 bicycles/hour shows that the roundabout always offers statistically significant shorter average travel times for motor vehicles. Apart from the lowest motor vehicle intensity level the difference between the SGFC-junction performance and roundabout performance decreases as motor vehicle intensity increases. If this trend continues the SGFC-junction will likely offer lower average motor vehicle travel times than the roundabout from about 3.500 mv/h and upward.

SGFC-junction and roundabout comparison at 1800 bicycles per hour
For the traffic demand scenarios with 1.800 bicycles/hour the results are not as straightforward. At a motor vehicle intensity of 1.000 MV/h the SGFC offers a significantly lower average travel time than the roundabout. At a motor vehicle intensity of 1.500 MV/h the roundabout offers a significantly lower average travel time. At the following two motor vehicle intensity levels the roundabout offers lower travel times but the differences with the SGFC-junction are not statistically significant. Finally at a level of 3.000 MV/h the SGFC-junction offers significantly lower travel times than the roundabout. There is no clear trend behind the differences between the junction types performances.

SGFC-junction lines
Both SGFC-junction graphs have similar values. This indicates that the bicycle intensity only slightly affects the motor vehicle travel time. This is caused by the setup of the SGFC traffic light control. It takes only one bicycle on one arm to implement the bicycle green phase. The maximum traffic light cycle length is set at 120 seconds and the bicycle phase has a fixed green time of 10 seconds which means that the car green phase(s) in-between two possible bicycle green light phases take up to around 45 seconds. So if at least one bicycle arrives at the junction per 45 seconds the bicycle phase

is applied twice per traffic light cycle which, assuming an even distribution, already happens at an intensity of $3600 \div 45 = 80$ bicycles per hour. In the simulations the arrival of bicycles is not evenly distributed over time. Therefore the dedicated bicycle green phase is still skipped sometimes at bicycle intensities higher than 80 bicycles per hour although the probability that a green stage is skipped decreases with increasing bicycle intensity. For the first three motor vehicle intensity levels the difference between the SGFC-junction lines increases with an increase in motor vehicle intensity. As motor vehicle intensity increases the amount of motor vehicles that arrive at the junction during a bicycle green stage increases. A difference in the amount of realised bicycle green stages caused by a different bicycle intensity level therefore affects more motor vehicles at higher motor vehicle intensity levels. This results in a higher probability of queue forming because of a bicycle green stage which explains the trend of an increasing difference between the two SGFC-junction lines. At an intensity level of 2.500 motor vehicles per hour the SGFC line for 1.800 bicycles per hour deviates from this trend because it is higher than expected. Given that the difference between both SGFC lines is smaller at the motor vehicle intensity level of 3.000 mv/h than at the level of 2.500 mv/h is another indication that the SGFC line for 1.800 bicycles per hour is erroneous at the latter point.

The path that both SGFC-junction lines follow is not completely as expected at first sight. The first two points do follow the expected path: as motor traffic intensity increases queues at the junction will increase resulting in higher average travel times for motor vehicles. The last three points on the graph show a downward trend and thus contradict the general theory. This can however be explained by the fact that at these higher intensity levels dedicated turn lanes are implemented at the SGFC-junction, following the CROW recomendations, making the designs more realistic. The increase in motor vehicle intensity level from 1.500 mv/h to 2.000 mv/h leads to both a right and a left turn lane on the major road approaches, see table Table 7 in paragraph 2.7. The introduction of a left turn lane on the major road also means that a dedicated left turn green phase is implemented. This reduces the maximum green of the right turn/ through green phase of the major road. Apparently these added turn lanes increase the junction capacity to such extend that this overcompensates the expected longer queues as a result of the higher motor vehicle intensities.

The increase of motor vehicle intensity from 2.000 mv/h to 2.500 mv/h leads to the introduction of a left turn lane on the minor roads, see table Table 7 in paragraph 2.7. This does not change the number of green phases. The introduction of a left turn lane increases junction capacity and can therefore cause a change in the slope of the graph. The fact that the SGFC line for 200 bicycles per hour has a downward slope whilst the line for 1.800 bicycles per hour has an upward slope between the motor vehicle intensity levels of 2.000 mv/h and 2.500 mv/h is not expected. Given that the motor vehicle intensities do not differ between the two lines and that the other points in the graph show that bicycle intensity level only has a small impact on junction performance, it would be logical if both lines had a slope in the same direction. Because the previous introduction of other turn lanes caused a reduction in average travel time it seems that the SGFC line for 200 bicycles per hour follows the logical path. The increase that the SGFC line shows thus seems to be erroneous. It is unknown what has caused this deviancy.

The increase of motor vehicle intensity from 2.500 mv/h to 3.000 mv/h leads to the introduction of a right turn lane on the minor road, see table Table 7 in paragraph 2.7. This does not alter the amount of green stages. The introduction of the turn lane again overcompensates for the increase in motor vehicle intensity because both SGFC lines have a downward slope.

Roundabout lines

The roundabout line for traffic demand scenarios with 1.800 bicycles per hour always lies well above the roundabout line for demand scenarios with 200 bicycles per hour. This indicates that bicycle intensity has a relatively big impact on motor vehicle travel time. As the bicycle intensity increases the probability that a motor vehicle has to stop to give way when entering or exiting the roundabout increases and thus the average travel time for motor vehicles increases. Furthermore this higher stopping probability of motor vehicles also increases the probability of queues and congested traffic conditions on the roundabout itself and on roundabout approaches. Congested traffic conditions have a negative effect on the capacity of the roundabout circulating lane and the roundabout approaches. An increase in motor vehicle intensity also increases the probability that a motor vehicle has to stop and give way to a bicycle and thus also increases the probability of queues at the junction. As the motor vehicle intensity level increases it is therefore expected that the difference between the two roundabout lines increases. For the step from the first to the second motor vehicle intensity level this trend is followed. For the other motor vehicle intensity levels this expected trend does not show. This is because the roundabout line for 1.800 bicycles per hour levels out at the higher motor vehicle intensity levels. This is likely caused by a limitation of the simulation model, see the explanation in the paragraph below.

As noted before increases in motor vehicle and bicycle intensity lead to a higher probability of motor vehicles having to stop and queue forming. Each roundabout line is therefore expected to show an upward and increasing slope. The roundabout line for traffic demand scenarios 200 bicycles per hour has an upward slope, but the slope decreases with motor vehicle intensity. This could be because the size of the simulated road network is limited. At an extremely high motor vehicle intensity the queue length on the approaches quickly equals the length of the modelled approach meaning that it is not possible to add more vehicles to the network. The time between the moment of intended release onto the network and the actual release is not registered as vehicle travel time and therefore the case where each motor vehicle is in queued condition from its release onto the network until passing the roundabout results in a maximum travel time per vehicle. In case all motor vehicles experience this maximum queued condition the maximum measurable total travel time in that specific simulated network, and therefore maximum average travel time per vehicle, is reached. The decrease in the slope of the graph indicates that the maximum measurable travel time is approached. The CROW guidelines estimate the capacity for a single lane roundabout at 2.500 mv/h (CROW, 2008). Given that the highest simulated intensity is 3.000 motor vehicles per hour it is very likely that this is maximum measurable queue length is approached.

For the bicycle intensity level of 1.800 bicycle per hour the roundabout line initially follows the upward, but decreasing, slope just as the graph for an intensity of 200 bicycles per hour. From a motor vehicle level of 2.000 MV/ h onward the graph however displays a downward trend. This cannot be explained by the existence of a maximum measurable travel time in a simulation model. Because there is no known explanation for why the line deviates from the expected these results are unreliable and should not be taken into account for the final conclusions.

The average travel time for bicycles over the same motor vehicle and bicycle intensity levels is shown in Figure 10 below.

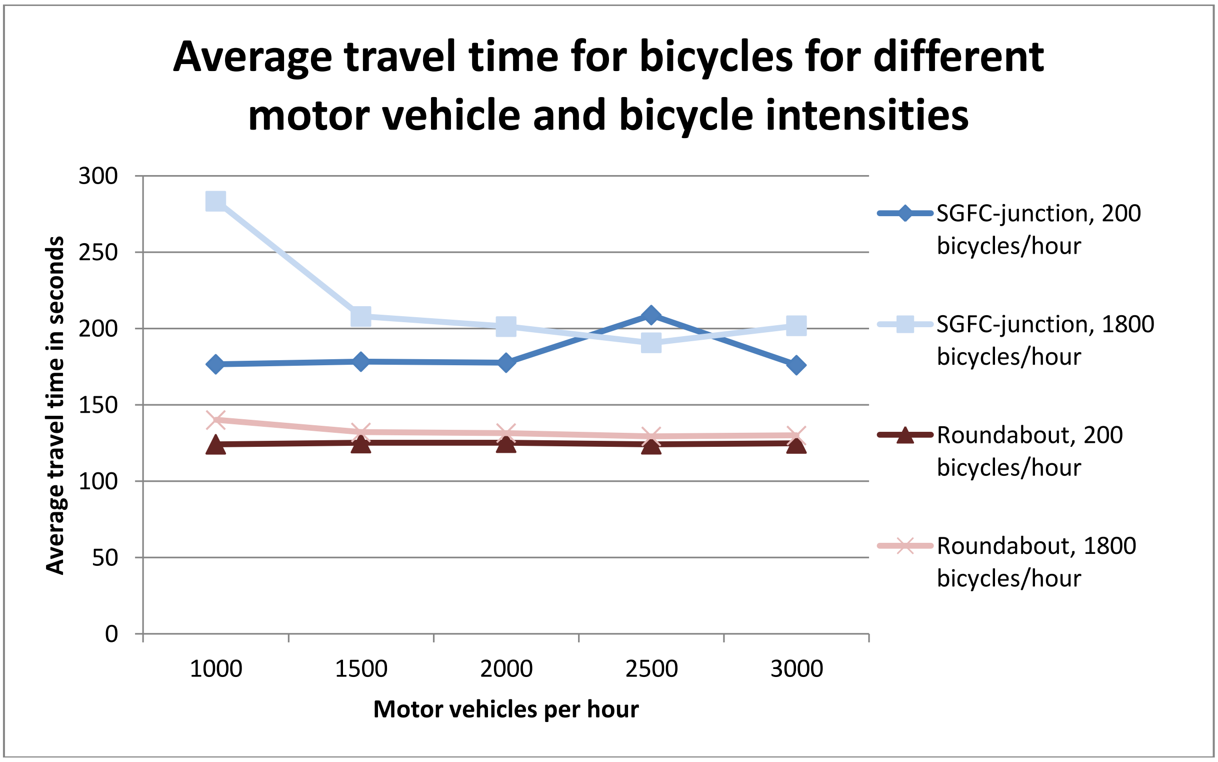

Figure 10: Comparison of average travel time for bicycles over different motor vehicle and bicycle intensities and the two junction types.

SGFC-junction compared to roundabout

Just as for the average motor vehicle travel time all the average bicycle travel time values of the different runs per traffic demand scenario are normally distributed meaning that the two sample T-test is applied for checking the statistical significance of the differences between the SGFC-junction and roundabout performance. Both at the 200 bicycles per hour and the 1.800 bicycles per hour intensity levels the differences between the two junction types are statistically significant at all five motor vehicle intensity levels. For the traffic demand scenarios with 200 bicycles per hour the roundabout offers a lower average travel time for bicycles compared to the SGFC-junction. The comparison between the two lines representing a bicycle intensity of 1.800 bicycles per hour gives the same outcome. These outcomes are expected given that bicycles never have to stop for motor vehicles at the roundabout whilst some bicycles will have to stop for a red light at the SGFC-junction.

SGFC-junction lines

In general the SGFC-junction line for 1.800 bicycles per hour lies above the SGFC line for 200 bicycles per hour. As bicycle intensity increases longer queues form during the time that the bicycle traffic light is red. Longer queues lead to higher bicycle travel times, as is generally indicated by the SGFC lines. As motor vehicle intensity increases the probability that green phases for motor vehicles are extended increases meaning that the time between two bicycle green phases increases. At a higher bicycle intensities this leads to more queued bicycles than at low bicycle intensities. Given the capacity drop that occurs with congested traffic conditions the difference between the two SGFC lines would have to increase with increasing motor vehicle density. The fact that this does not happen in Figure 10 can be explained by assuming that the length of the green phases for motor vehicles is already maximised at the lowest motor vehicle level and therefore does not increase with increasing motor vehicle intensity.

The SGFC line for traffic demand scenarios with 200 bicycles per hour follows an almost horizontal path, except for the fourth data point. This indicates that the motor vehicle intensity hardly impacts the travel time for bicycles. The only explanation for this is that the length of the motor vehicle green phases is already at its highest at the lowest motor vehicle intensity and can therefore not increase with increasing motor vehicle intensity. Given that this is not only an explanation for the horizontal path of the SGFC line for 200 bicycles per hour, but also explains the relatively constant difference between the two SGFC-junction lines it is likely that the length of motor vehicle green phases indeed remains constant over different motor vehicle intensities. This indicates that the signal control logic could have been set up more efficient for low motor vehicle intensities by reducing the number of extra seconds of green for each motor vehicle detected on the approach during that motor vehicle green phase. This would reduce the time between two bicycle green phases and thus reduce the average travel time for bicycles. Given that bicycles at roundabouts never have to give way to motor vehicles it is not expected that this would change the ranking of the two junction types.

Relative to the other four points on the SGFC line for 200 bicycles per hour the point at a motor vehicle intensity level of 2.500 mv/h breaks with the trend of the line. Not only does this point lie higher than the other four points it is also higher than the SGFC line for 1.800 bicycles per hour. The high average travel time at this point is therefore unrealistic: a higher bicycle intensity should never lead to a lower average travel time for bicycles. Over the ten runs for this data point the standard deviation is only 3,65 at an average travel time of 208,8 seconds. Compared to the average value the standard deviation is small. This means that all ten runs consistently score unexpectedly high values and the error is not caused by a model failure in one of the runs. It is unclear what has caused this outlier.

The SGFC-junction line for a bicycle intensity of 1.800 bicycles per hour also has one outlying value, in this case the value at a motor vehicle intensity of 1.000 mv/h. The first value of the graph lies a lot higher than the other points. The resulting downward slope of the line from the first to the second point of the line is illogical since an increase in motor vehicle intensity can never benefit bicycle travel time. Measured over the ten runs behind this data point the standard deviation is 19,4 which is relatively low given the average travel time value of 283,4 seconds. So again all ten runs produce unexpectedly high values and the outlier is not caused by a model failure in one run. It is not known what has caused this outlying data point.

Roundabout lines

The two lines representing the roundabout are almost the same which means that the bicycle intensity only marginally influences the travel time for bicycles. This indicates that the roundabout is operating far below it's bicycle capacity. Bicyclists only have to give way when entering the circulatory bicycle path. The bicycle intensity at each of the four conflict points is relatively low. At each of the two main road arms $60\% \div 2 = 30\%$ of the total bicycle intensity approaches the junction. This flow conflicts with three other flows:

- left turning bicycles from the opposite main road approach equaling $30\% \times 0{,}15 = 4{,}5\%$ of the total bicycle intensity
- left turning bicycles from the adjacent minor road approach, equaling $20\% \times 0{,}15 = 3\%$ of total bicycle intensity
- straight through going bicycles from the adjacent minor road approach, equaling $20\% \times 0{,}75 = 15\%$ of the total bicycle intensity

Therefore the conflict intensity is $(30\% + 4.5\% + 3\% + 15\%) \times 1.800 = 945$ bicycles per hour. The CROW design manual ASVV does not give the bicycle conflict capacity of roundabouts. It does give the conflict capacity for a single lane roundabout regarding motor vehicles at 1.500 motor vehicles per hour (CROW, 2012). Given that the capacity of a 1.80 meter wide bicycle path lies much higher than the capacity of a single lane road, about 4.700 bicyclists per hour (CROW, 2006) compared to about 2.000 cars per hour, it is likely that the bicycle conflict capacity for the roundabout is at least more than double that of the conflict capacity for motor vehicles. This means that the occurring conflict intensity of 945 bicycles per hour lies far below bicycle conflict capacity. Over the different motor vehicle intensity levels the difference between the roundabout lines remains almost constant. This indicates that the motor vehicle intensity does not impact bicycle travel times, which is logical given that bicycles have priority over motor vehicles.

Both the roundabout lines follow a horizontal path. This means that the motor vehicle intensity hardly influences the average travel time for bicycles. This is in line with expectations because bicycles have priority over motor vehicles.

Conclusions
The most noteworthy conclusions from Figure 9 about motor vehicle travel time are:

- At the low bicycle intensity level the roundabout always offers significantly lower motor vehicle travel times than the SGFC-junction
- At the high bicycle intensity the SGFC-junction has the lowest travel times at the lowest and the highest motor vehicle intensity, the roundabout has lower travel times at the other motor vehicle intensity levels
- Generally the differences between the two SGFC-junction lines are very small meaning that bicycle intensity hardly impacts motor vehicle travel time
- The erratic path of the SGFC-junction graph lines is caused by adding turn lanes to junction approaches at different motor vehicle intensity levels
- The motor vehicle travel time at the roundabout line for 1.800 bicycles is always higher than the line for 200 bicycles per hour and generally the difference increases with motor vehicle intensity
- Due to limitations to the simulated road network the upward slope of the roundabout lines levels out with increasing motor vehicle intensity
- For the roundabout line for a bicycle intensity of 1.800 bicycles per hour the point at the two highest motor vehicle intensities are lower than the point at the middle motor vehicle intensity and are therefore erroneous and thus not taken into account

The most noteworthy conclusions from Figure 10 about bicycle travel times are:

- The roundabout always offers better travel times for bicycles than the SGFC-junction
- At the SGFC-junction a higher bicycle intensity leads to only slightly higher bicycle travel times indicating that bicycle intensity hardly influences bicycle travel time
- In general both SGFC-junction lines follow an horizontal path indicating that motor vehicle intensity hardly influences bicycle travel time
- The fourth point of the 200 bicycles per hour and the first point of the 1.800 bicycle per hour SGFC lines are inexplicably high and should therefore be ignored
- At the roundabout a higher bicycle intensity leads to only slightly higher bicycle travel times indicating that bicycle intensity hardly influences bicycle travel time
- Both roundabout lines follow an almost horizontal path indicating that motor vehicle intensity hardly influences bicycle travel time

4.1.2. Impact of motor vehicle intensity and motor vehicle main and side road distribution

For this comparison the amount of motor vehicle traffic allocated to the main road is varied, as well as the motor vehicle traffic intensity. The bicycle intensity is set at 1.000 bicycles per hour. The share of left turning traffic is set at 15% for both modes. The average travel times for motor vehicles over different motor vehicle intensities and main and side road distributions are shown in Figure 11 below.

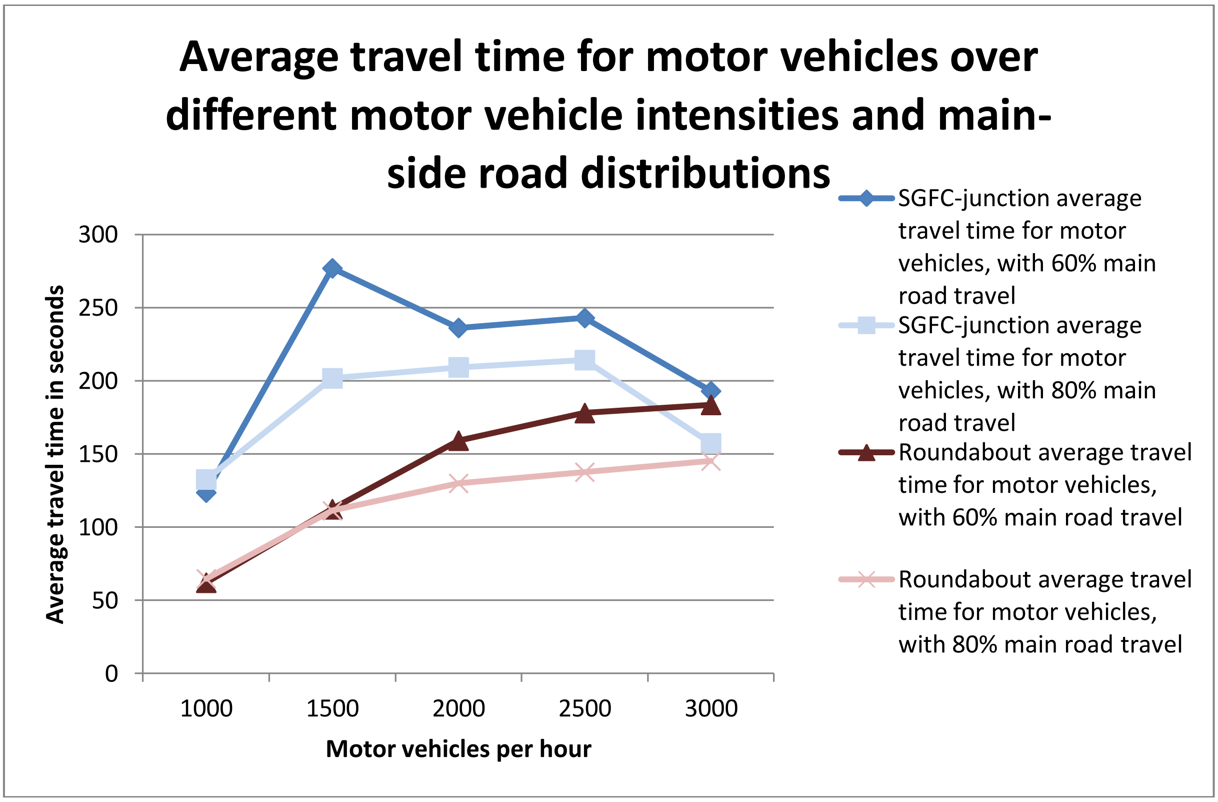

Figure 11: Average travel time for motor vehicles over different motor vehicle intensities and main- / side road distributions

SGFC-junction and roundabout compared at 60% main road travel

Comparing the two graphs for the situation of 60% motor vehicle traffic on the main road shows that the roundabout scores the lowest average travel time for motor vehicles for all motor vehicle intensities. The two sample t-test indicates that the differences between the two lines all are statistically significant. The turn lanes that are added to the SGFC-junction at motor vehicle intensity levels of 2.000 mv/h, 2.500 mv/h and 3.000 mv/h keep reducing the difference with the roundabout performance. Because one of the prerequisites for the SGFC-junction is that it needs to be relatively compact, so that the clearance time after the bicycle green stage does not get too long, more than three lane per approach is unwanted. This means that the maximum amount of turn lanes for a SGFC-junction is already applied at a motor vehicle intensity level of 3.000 mv/h, an SGFC-junction needs to be relatively compact, the average motor vehicle travel time for the SGFC-junction should increase for higher motor vehicle intensities.

SGFC-junction and roundabout compared at 80% main road travel

The comparison of the two lines for 80% main road travel results in the same conclusion: the roundabout always has the lowest travel times and all differences between the roundabout and the SGFC-junction are statistically significant. The difference between the SGFC-junction and the roundabout remains relatively constant over the different motor vehicle intensity levels except at the highest intensity level.

SGFC-junction lines

The motor vehicle travel time at the SGFC-junction generally is higher for the traffic demand scenarios with 60% main road motor vehicle traffic compared to the demand scenarios with 80% main road traffic. Because the amount of traffic on the side road is lower there are fewer conflicts with main road traffic which explains the lower travel times for the traffic demand scenarios with 80% main road traffic. Except for the first two intensity levels the difference between the SGFC lines remains relatively constant. At an intensity level of 1.500 mv/h the difference between the lines is more than twice as big. This is caused by a difference in turn lane configuration. The traffic demand scenario with 80% main road traffic has more left and right turning motor vehicles than the respective threshold values for dedicated turn lanes already at a total intensity of 1.500 mv/h. For the traffic demand scenario with 60% main road motor traffic this does not happen until a total intensity of 2.000 mv/h. At the lowest intensity level both SGFC-junction lines lie much closer together compared to the other intensity levels and the order of the lines is switched indicating that at least one of the two points on the graph is erroneous. For the traffic demand scenario with 60% main road motor vehicle traffic the standard deviation over the ten runs at the intensity level of 1.000 mv/h is 5,25 and for the 80% main road traffic demand scenario this is 5,97. The small size of the standard deviation indicates that the seemingly erroneous values in the graph are not caused by an error in one of the runs. It is therefore still unknown what caused this unexpected outcome and therefore these results should only be copied with care.

The graphs for the SGFC-junction initially follow an upward trend: with increasing motor vehicle intensities congestion and thus average motor vehicle travel time increases. The fact that the graph levels out and eventually decreases at the high motor vehicle intensities is explained by the dedicated turn lanes that are used in these cases, see table Table 7 in paragraph 2.7 for a turn lane overview. Apparently the extra junction capacity that comes with these turn lanes overcompensates for the higher average travel time that would be caused by higher motor vehicle intensities.

Roundabout lines
At the lowest two motor vehicle intensities there hardly is a difference between the two roundabout lines. For the other intensity levels the demand scenarios with 60% main road traffic lead to higher average travel times, just as was the case for the SGFC-junction.

The graph for the roundabout shows an upward trend as expected. The decrease in the upward slope is likely explained by the fact that the maximum measureable average travel time in the simulation model is approached, see the explanation below Figure 9.

Conclusions
The most noteworthy conclusions from Figure 11 about motor vehicle travel time over different motor vehicle intensities and main side road distributions are:

- The roundabout offers lower average motor vehicle travel times than the SGFC-junction
- In general the traffic demand scenarios with 80% main road motor vehicle traffic offer lower average motor vehicle travel times at both junction types, because a higher share of main road traffic leads to less conflicts with side road traffic.
- Both roundabout lines level out at high motor vehicle intensities caused by modelling limitations

The average travel times for bicycles over different motor vehicle intensities and main and side road distributions are shown in Figure 12 below.

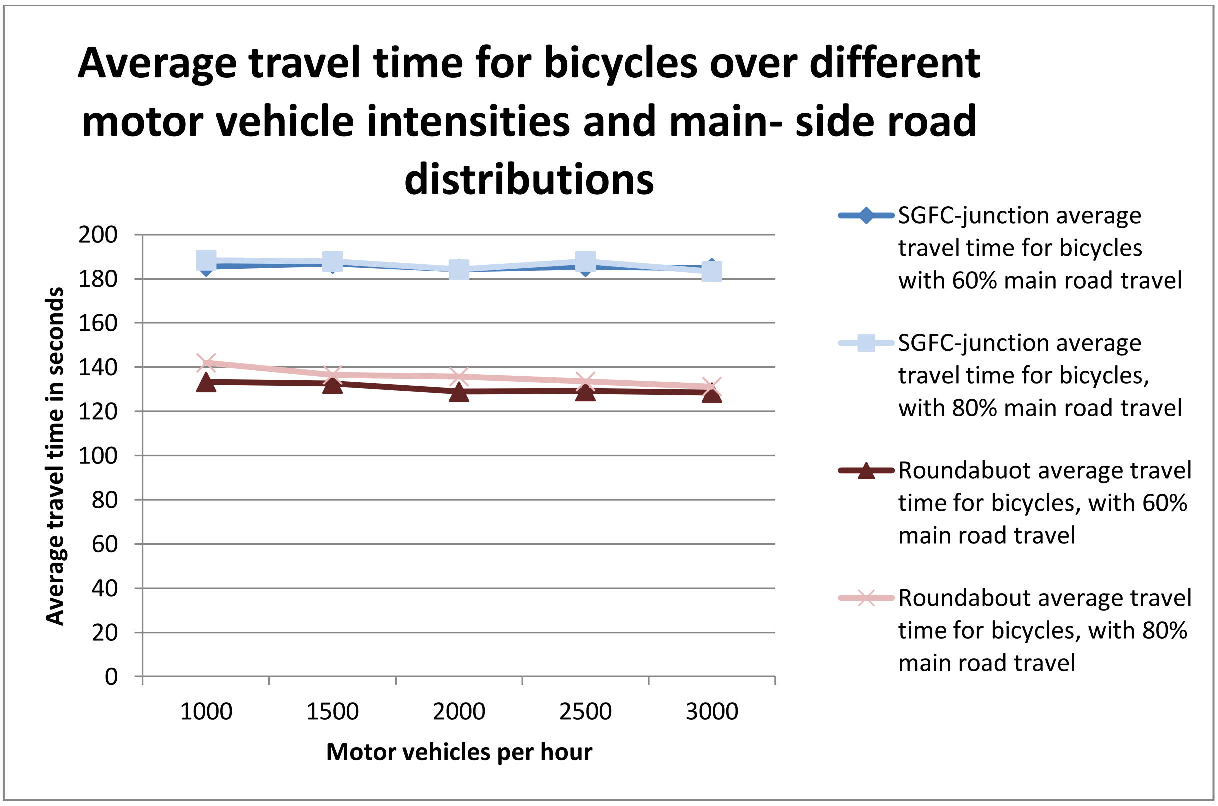

Figure 12: Average travel time for bicycles over different motor vehicle intensities and main- side road distributions

SGFC-junction and roundabout compared
Figure 12 shows that The roundabout has lower average travel times for bicycles than the SGFC-junction at any of the motor vehicle intensity and main/side road distributions. The difference between the junction types is statistically significant according to the two sample t-test for all displayed traffic demand scenarios.

The fact that both SGFC lines show almost the same values indicates that the distribution of motor vehicle traffic over the main and side road hardly impacts average bicycle travel time. A redistribution of motor vehicle traffic over the main and side roads leads to a proportionate redistribution of green time over the different motor vehicle green phases. The total amount of green time allocated to motor vehicles remains constant and therefore the average bicycle travel time remains the same.

SGFC-junction lines
Both SGFC-junction lines follow a horizontal path meaning that motor vehicle intensity on average does not impact bicycle travel times. The lack of this impact can be explained by the assumption that the green time for motor vehicles was already extended to the maximum values at the lowest motor vehicle intensity level, as already explained below Figure 10.

Roundabout lines
Both roundabout lines have similar values, again indicating that the motor vehicle main-/ side road distribution does not impact the travel time for bicycles. The average travel time for bicycles at the roundabout remains the same because motor vehicles have to give way to bicycles, regardless of the arm on which they approach the roundabout.

Conclusions
The most noteworthy conclusions from Figure 12 about bicycle travel time over different motor vehicle intensities and main side road distributions are:

- The roundabout offers better average travel times for bicycles than the SGFC-junction
- At both junction types the motor vehicle main-/ side distribution does not impact average bicycle travel time because the share of bicycle green time out of the total cycle time remains constant (SGFC-junction) and bicycles have priority over motor vehicles regardless of junction arm (roundabout)
- Motor vehicle intensity hardly impacts average bicycle travel time as already argued below Figure 10

4.1.3. Impact of bicycle intensities and motor vehicle main- / side road intensities

For this comparison the amount of motor vehicle traffic allocated to the main road is varied, as well as the bicycle intensity. The motor vehicle intensity is set at 2.000 motor vehicles per hour. The share of left turning traffic is set at 15% for both modes. Figure 13 shows the average travel time for motor vehicles over different bicycle intensities and main-side road distributions.

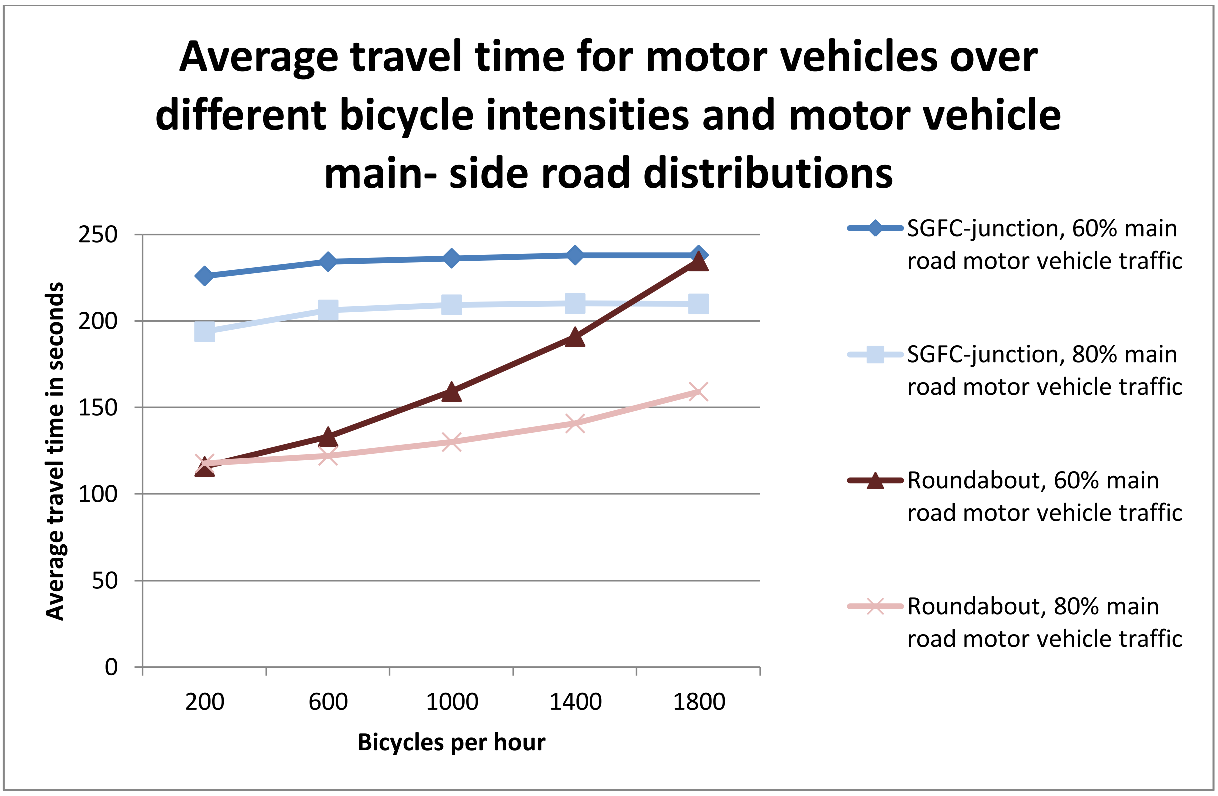

Figure 13: Average travel time for motor vehicles over different bicycle intensities and main- side road distributions

SGFC-junction compared to roundabout at 60% main road motor vehicle traffic

Figure 13 shows that for the situation with 60% main road motor vehicle traffic the roundabout has lower average travel times for motor vehicles than the SGFC-junction at all bicycle intensity levels. The t-test indicates that these differences are statistically significant for all except the highest bicycle intensity level. The difference between the two junction types decreases exponentially with increasing bicycle intensity. As bicycle intensity increases the amount of bicycle green time hardly does, see the explanation below Figure 9, meaning that the hinder of motor vehicles also remains constant. For the roundabout increasing bicycle intensities do lead to longer average motor vehicle travel times because the probability that motor vehicles have to give way to bicycles increases with bicycle intensity. If the displayed trend continuous than the SGFC-junction will offer better motor vehicle travel times for bicycle intensities higher than 1.800 bicycles/hour.

SGFC-junction compared to roundabout at 80% main road motor vehicle traffic

For the situation with 80% main road motor vehicle traffic the roundabout always scores better than the SGFC-junction as well. In this case however the differences are statistically significant at all bicycle intensity levels. The difference between both junction types progresses in the same manner as between the 60% main road motor vehicle traffic lines. Only the rate of decrease is lower meaning that it would take a higher bicycle intensity before the SGFC-junction would offer better travel times than the roundabout compared to the traffic demand scenarios with 60% main road motor vehicle traffic.

SGFC-junction lines
Just as in Figure 11 the SGFC-junction line for traffic demand scenarios with 60% main road traffic lies above the SGFC line for 80% motor vehicle traffic. The difference between the two SGFC lines remains relatively constant over the different bicycle intensities which is caused by the fact that the bicycle intensity has little effect on the amount of green time allocated to bicycles, see the explanation below Figure 9.

Both of the SGFC-junction lines follow an almost horizontal path which means that the average motor vehicle travel time is hardly affected by the bicycle intensity. This occurs because the dedicated bicycle green phase is already implemented twice per traffic light cycle from a relatively low bicycle intensity, see the explanation under Figure 9.

Just as for the SGFC-junction the roundabout line for 60% main road motor vehicle traffic generally lies above the line for 80% main road traffic. At the lowest bicycle intensity the roundabout lines are almost equal, but with increasing bicycle intensity the differences between the lines increases as well.

Roundabout lines
The roundabout lines show an positive exponential trend which is expected. As the bicycle intensity increases the probability that a motor vehicle has to give way to a bicycle when entering and exiting the roundabout increases which results in a higher average travel time.

Conclusions
The most noteworthy conclusions from Figure 13 about motor vehicle travel time over different bicycle intensities and motor vehicle main side road distributions are:

- The roundabout offers lower average travel times than the SGFC-junction
- A higher share of motor vehicle traffic on the main road leads to lower travel times on both junction types
- Bicycle intensity hardly impacts motor vehicle travel time at the SGFC-junction
- At the roundabout an increasing bicycle intensity results in higher motor vehicle travel times

Figure 14 below shows the average travel time for bicycles for the same set of traffic demand scenarios.

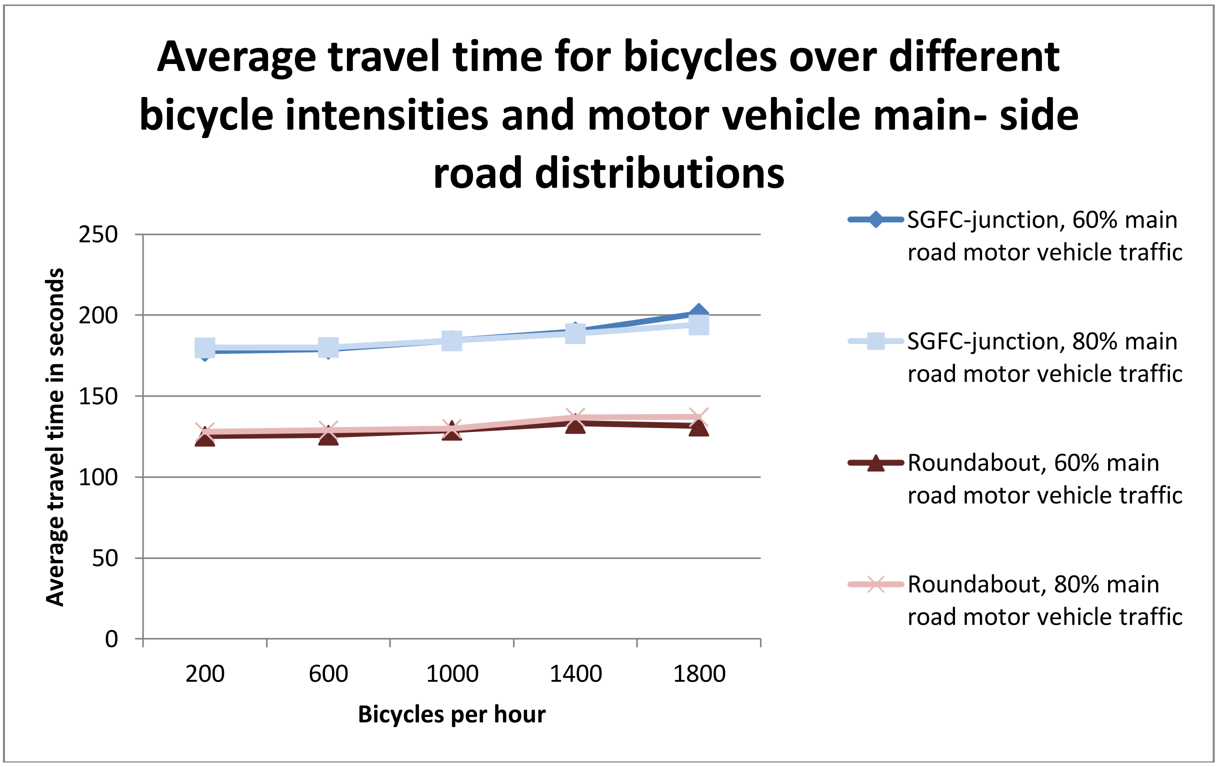

Figure 14: Average travel time for motor vehicles over different bicycle intensities and main- side road distributions

SGFC-junction and roundabout compared

Figure 14 shows that for both the 60% and the 80% main road motor vehicle traffic the roundabout scores lower average bicycle travel times for all bicycle intensity levels. The two sample t-test indicates that all differences between the junction types are statistically significant.

SGFC-junction lines

Consistent with Figure 12 both SGFC-junction lines have almost the same values meaning that the motor vehicle main-/ side road distribution does not impact bicycle travel times.

Both lines of the SGFC-junction show a slight upward trend which is expected. As the bicycle intensity increases the queue lengths on the approach increase. Furthermore during the bicycle green phase bicycles have a slightly higher probability that they have to give way when crossing the junction. Both effects lead to an increase in average bicycle travel time.

Roundabout lines

Consistent with Figure 12 both roundabout lines have almost the same values meaning that the motor vehicle main-/ side road distribution does not impact bicycle travel times.

Both the roundabout lines follow an almost horizontal path. As indicated before under Figure 10 the roundabout capacity for bicycles is likely much higher than the currently used bicycle intensities. Therefore the higher simulated bicycle intensities hardly increase the travel time for bicycles.

Conclusions

The most noteworthy conclusions from Figure 14 about bicycle travel time over different bicycle intensities and motor vehicle main side road distributions are:

- The roundabout offers lower average bicycle travel times compared to the SGFC-junction
- At both junction types the motor vehicle main-/ side road distribution hardly impacts bicycle travel time

- At the SGFC-junction increasing bicycle intensity leads to slightly increasing average bicycle travel times because an increase in bicycle queue lengths
- At the roundabout an increase in bicycle intensity hardly impacts bicycle travel time because the tested intensity values are likely far below the roundabout capacity

4.1.4. Impact of motor vehicle intensity and the share of left turning motor vehicles

For this comparison the motor vehicle intensity and the share of left turning motor vehicles are variable. The bicycle intensity is set at 1.000 bicycles/hour. The share of traffic allocated to the main road is kept at 60% for both modes. For bicycles the share of left turning traffic is set at 15%. The average travel times for motor vehicles over different motor vehicle intensities and left turning shares are shown in Figure 15 below.

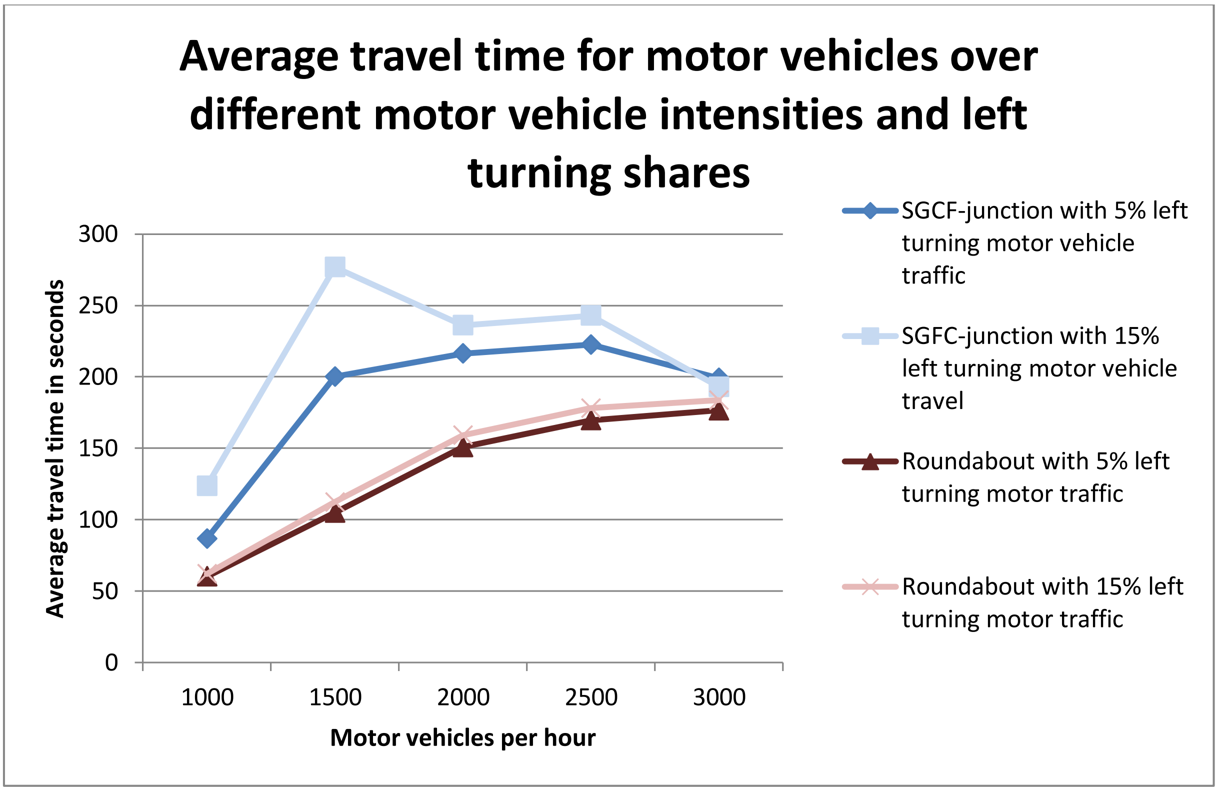

Figure 15: Average travel time for motor vehicles over different motor vehicle intensities and left turning traffic shares

SGFC-junction and roundabout compared at 5% left turning motor traffic

Comparing the two graphs for the situation with 5% left turning motor vehicle traffic shows that the roundabout always offers the lowest average travel time for motor vehicles. All these difference are statistically significant according to the two sample t-test. Initially the difference between the SGFC-junction and the roundabout increases with increasing motor vehicle intensity. However from an intensity of 2.000 mv/h the difference decreases again due to the stepwise implementation of turn lanes at the SGFC-junction.

SGFC-junction and roundabout compared at 15% left turning motor

The comparison between the two graphs for the situation with 15% left turning traffic shows similar results. For all compared situations the roundabout offers the lowest average travel times for motor vehicles and all these differences are statistically significant.

SGFC-junction lines

Of the two SGFC-junctions the graph for the situation with 15% left turning motor vehicle traffic generally has the highest average travel time for motor vehicles. This is to be expected. Out of the possible movements at a junction the left turn movement conflicts with the most other movements. An increase in the amount of left turning vehicles will therefore result in higher average travel times. The fact that at a motor vehicle intensity of 3.000 mv/h the situation with 15% left turning motor vehicle traffic scores lower contradicts this theory. This can however be explained by the setup of the signal plan. Each signal stage has a minimum green time. At the situation with only 5% left turning motor vehicle traffic the amount of left turning vehicles was high enough to justify a dedicated left turn lane and thus a left turn signal stage on the main road. However the amount of left turning vehicles was still so low that this minimum green time was likely not fully used. At an intensity of 3.000 mv/h and 60% main road traffic and 5% left turning motor vehicles each one of the main road junction approaches handles $\big((3.000 \times 60\%) \times 5\%\big) \div 2 = 45$ motor vehicles per hour. Given that the maximum traffic light cycle is two minutes there are at least 30 cycles per hour. This means that on average $45 \div 30 = 1.5$ left turning motor vehicle per arm has to be processed during the dedicated left turn green phase. Given the minimum green time of 5 seconds this means that the green time is not used very efficiently at these left turn intensities. By increasing the number of left turning vehicles the left turn stage was used more efficiently resulting in a more efficient signal control plan and thus lower average motor vehicle travel times overall. This efficiency of the left turn green phase also explains why the 15% left turn line decreases from the intensity level of 1.500 mv/h to 2.000 mv/h whilst the 5% line increases even though the same amount of turn lanes is added for both situations.

The levelling out, and eventually decrease, of the SGFC-junction lines can again be explained by the introduction of dedicated turn lanes at higher intensities.

Roundabout lines
The roundabout line for 15% left turning traffic lies above the roundabout line for 5% left turning traffic. Just as at the SGFC-junction the left turn movement conflicts with most other traffic flows. An increase in the share of left turning motor vehicles therefore results in a higher probability that approaching vehicles have to give way resulting in a higher average travel time.

Both roundabout line follow similar paths as in Figure 11. The lines follow an upward path at the lower motor vehicle intensities and level out at the higher intensities likely caused by a modelling limitation.

Conclusion
The most noteworthy conclusions from Figure 15 about motor vehicle travel time over different motor vehicle intensities and left turn shares are:

- The roundabout offers lower average motor vehicle travel times than the SGFG-junction
- Generally a higher motor vehicle left turn share increases motor vehicle travel time at the SGFC-junction however a certain minimum number of left turning vehicles, greater than the minimum used for adding a left turn lane, is needed in case the left turn has a dedicated green phase
- At the roundabout an increase in the motor vehicle left turn share only slightly increases motor vehicle travel time
- At high motor vehicle travel times the roundabout lines level out because of a modelling limitation

The average travel times for bicycles over different motor vehicle intensities and left turning shares are shown in Figure 10 below.

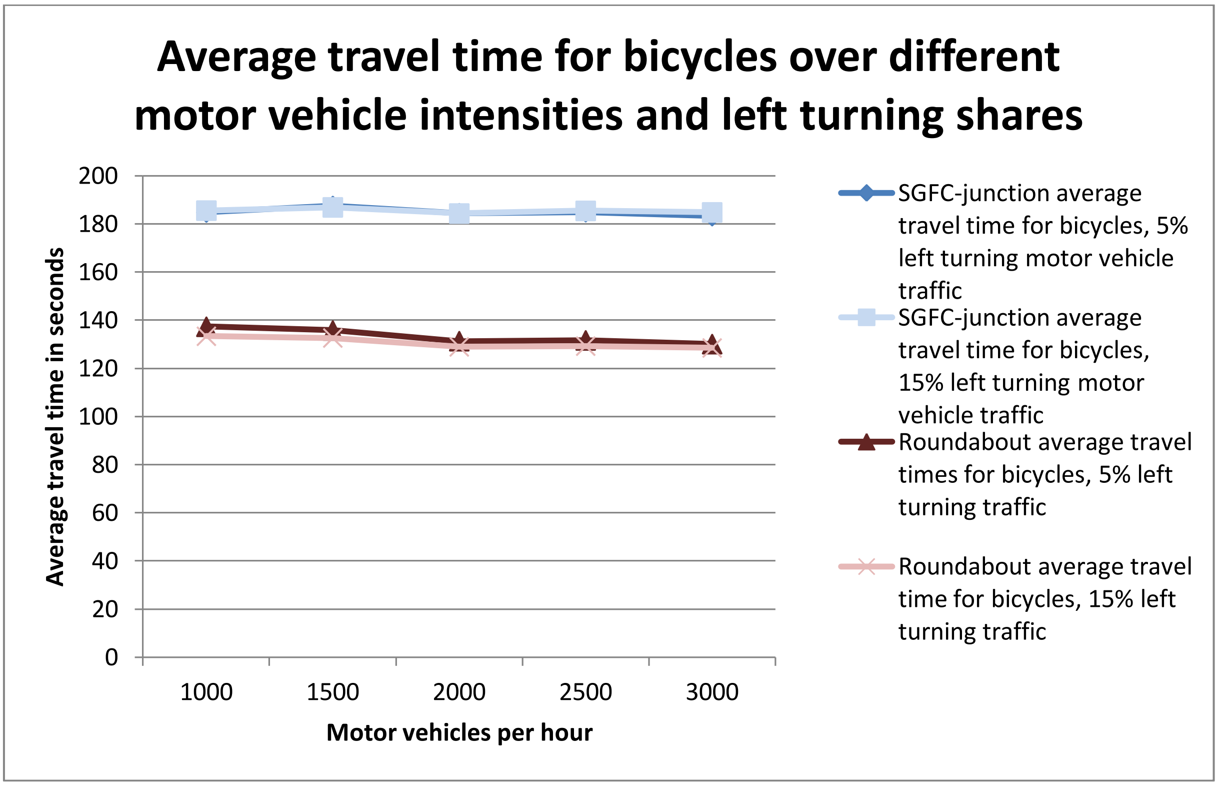

Figure 16: Average travel time for bicycles over different motor vehicle intensities and left turning motor vehicle shares

SGFC-junction and roundabout compared

Comparing the roundabout graphs with the respecting SGFC-junction graphs in Figure 16 above shows that the roundabout always offers lower average travel times for bicycles than the SGFC-junction. All of these differences are statistically significant at a significance level of 0,05 according to the two sample t-test.

SGFC-junction lines

The SGFC lines are almost the same, indicating that motor vehicle left turn share does not impact bicycle travel time. A change in the left turn share of motor vehicles leads to a redistribution of green time allocated to different motor vehicle green stages, but the total green time allocated to motor vehicles does not change, which explains why both lines contain similar values.

Both SGFC-junction lines follow a horizontal path indicating that motor vehicle intensity does not impact bicycle travel time. This is consistent with the results in Figure 12.

Roundabout lines

There is almost no difference between both roundabout lines indicating that the motor vehicle left turn share does not impact bicycle travel time. Both roundabout lines follow a horizontal path indicating that motor vehicle intensity does not impact bicycle travel time. Both indicated characteristics are explained by the fact that bicycles have priority over motor vehicles regardless of where they enter or exit the junction. This is in line with the results in Figure 12.

Conclusions

The most noteworthy conclusions from Figure 16 about bicycle travel time over different motor vehicle intensities and left turn shares are:

- The roundabout offers lower average bicycle travel times than the SGFC-junction
- At the SGFC-junction both motor vehicle intensity and left turn share do not impact average bicycle travel time

- At the roundabout both motor vehicle intensity and left turn share do not impact average bicycle travel time

4.1.5. Impact of bicycle intensity and left turning motor vehicles

For this comparison the bicycle intensity and the share of left turning motor vehicles are variable. The motor vehicle intensity is set at 2.000 motor vehicles/hour. The share of traffic allocated to the main road is kept at 60% for both modes. For bicycles the share of left turning traffic is set at 15%. Figure 17 shows the average travel times for motor vehicles over different bicycle intensities and motor vehicle left turning shares.

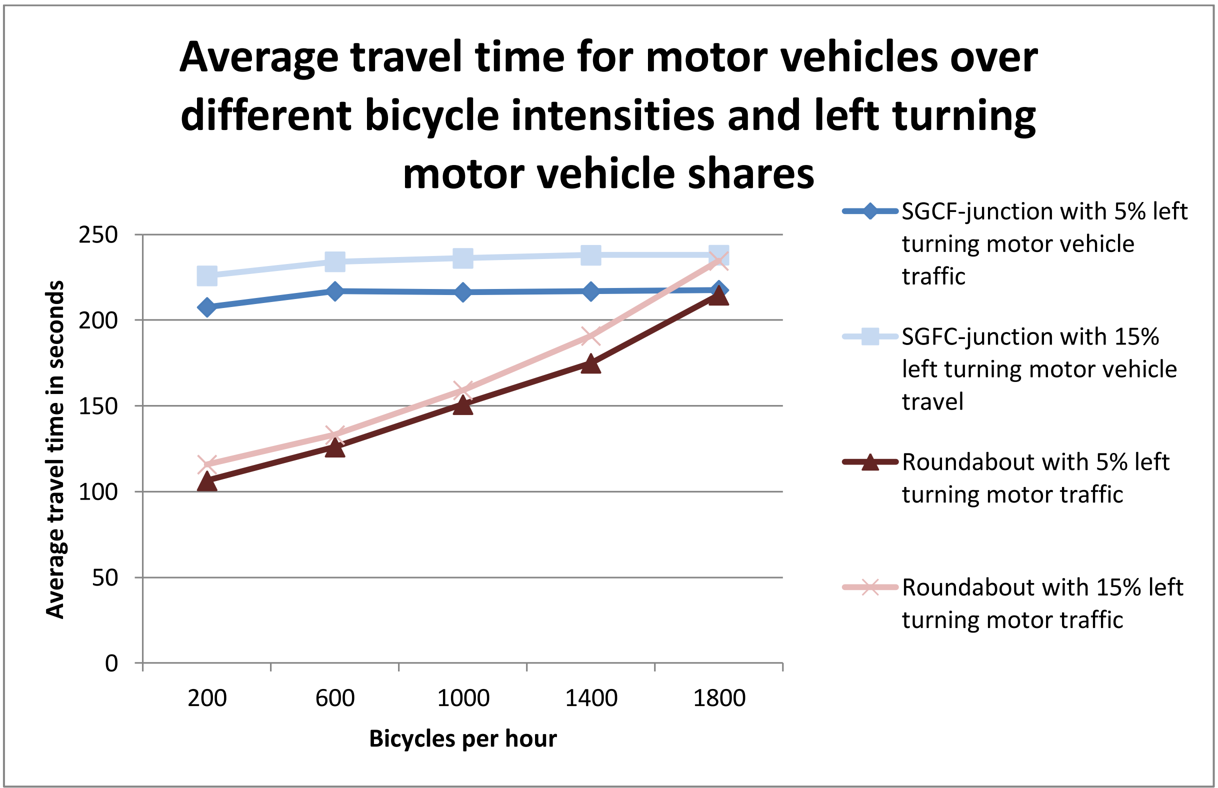

Figure 17: Average travel time for bicycles over different bicycle intensities and left turning motor vehicle shares

SGFC-junction and roundabout compared

For both left turn motor vehicle shares the roundabout scores lower average motor vehicle travel times for all bicycle intensity levels compared to the SGFC-junction. The two sample t-test shows that the differences between the junction types are statistically significant except for the highest level of bicycle intensity. As bicycle intensity increases the difference between the junction types gets smaller. It is very likely for higher bicycle intensities than tested here the roundabout would score higher travel times than the SGFC-junction.

SGFC-junction lines

At the SGFC-junction higher left turn motor vehicle share leads to a higher motor vehicle travel time because the left turn movement conflicts with a lot of other traffic flow directions. This is consistent with the results in Figure 15. The difference between both SGFC-junction lines remains constant over different bicycle intensities. This is because the amount of green time allocated to bicycles only slightly increases with bicycle intensity, as was explained below Figure 9.

Just as in Figure 13 before, the SGFC-junction lines follow a horizontal path meaning that bicycle intensity does not influence motor vehicle travel time.

Roundabout lines
The roundabout line for the demand scenarios with 15% left turning traffic lies above the roundabout line for demand scenarios with 5% left turning traffic. Left turning motor vehicles have to cross two junction approaches compared to only one for straight through traffic so an increase in the left turn share increases the amount of potential conflicts and thus the average travel time.

The roundabout lines again show an increasing trend indicating that as bicycle intensity increases the probability that motor vehicles have to give way and thus increase the average travel time for motor vehicles. This is consistent with the results presented earlier in Figure 13.

Conclusions
The most noteworthy conclusions from Figure 17 about motor vehicle travel time over different bicycle intensities and left turn motor vehicle shares are:

- The roundabout offers lower average motor vehicle travel times than the SGFC-junction
- At both junction types an increase of the left turn motor vehicle share leads to an increase in motor vehicle travel time because the left turn movement conflicts with the most other movements
- At the SGFC-junction an increase in bicycle intensity hardly affects motor vehicle travel time because the amount of green time allocated to bicycles only marginally increases
- At the roundabout an increase in bicycle intensity leads to an increase in motor vehicle travel time because motor vehicles have to give way to bicycles.

In Figure 18 below the average travel time for bicycles is shown for the same traffic demand scenarios.

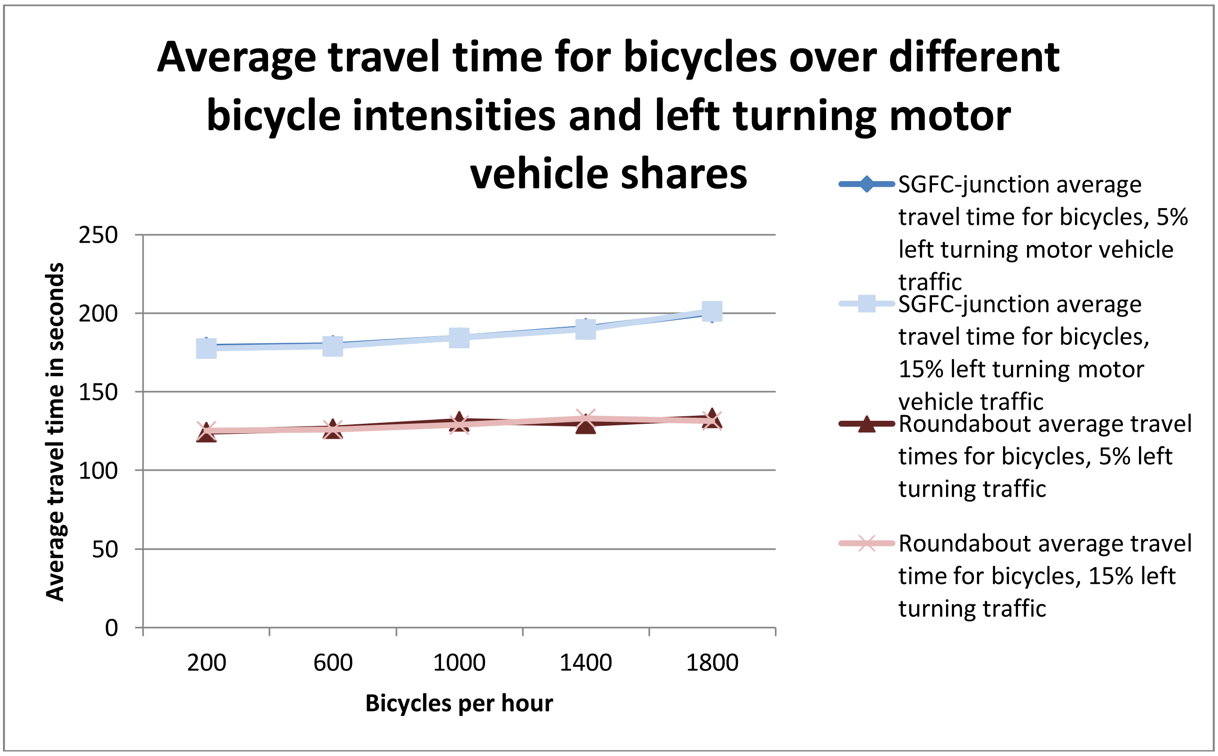

Figure 18: Average travel time for bicycles over different bicycle intensities and left turning motor vehicle shares

SGFC-junction and roundabout compared
Figure 18 shows that the roundabout scores lower average bicycle travel times for both left turning motor vehicle shares and all bicycle intensities. The two sample t-test indicates that all these differences between the two junction types are statistically significant.

SGFC-junction lines
The motor vehicle left turn share does not impact bicycle travel time at the SGFG-junction. An adjustment to the motor vehicle left turn share leads to a redistribution of green time between the main road left turn phase and the main road right turn / straight through phase. The distribution of green time between all motor vehicle phases and the bicycle phases does however not change. Therefore left turn motor vehicle share does not impact bicycle travel time.

An increase in bicycle intensity leads to a minor increase in average bicycle travel time. A higher bicycle intensity leads to longer queues at the traffic light and thus increases travel time. These results are consistent with those found in Figure 14.

Roundabout lines
At the roundabout the left turn motor vehicle share does not impact bicycle travel time because motor vehicles have to give way to bicycles regardless of their turn movement.

An increase in bicycle intensity hardly impacts the bicycle travel time, likely because the tested bicycle intensities are far below the roundabout bicycle capacity. This is consistent with the results shown in Figure 14.

Conclusions
The most noteworthy conclusions from Figure 18 about bicycle travel time over different bicycle intensities and left turn motor vehicle shares are:

- The roundabout offers better average bicycle travel times than the SGFC-junction
- Motor vehicle left turn share does not impact bicycle travel time at either junction type
- At the SGFC-junction increasing bicycle intensity leads to slightly increasing average bicycle travel times because an increase in bicycle queue lengths
- At the roundabout an increase in bicycle intensity hardly impacts bicycle travel time because the tested intensity values are likely far below the roundabout capacity

4.2. Results concerning safety

The average number of TTC conflicts per vehicle is used to compare the two junction designs regarding traffic safety. The following five sub-paragraphs each discuss the impact of two of the variables used to describe the traffic demand scenarios. Each sub-paragraph shows a graph with the results regarding the number of bicycle only conflicts. The results about motor vehicle only conflicts are not presented here because it turns out that at the SGFC-junction these conflicts hardly take place compared to the roundabout conflict levels. Furthermore the number of conflicts at the SGFC-junction appears to be almost independent of the traffic demand scenario. This means that under any of the tested traffic demand scenarios the number of motor vehicle conflicts is much higher at the roundabout than at the SGFC-junction which makes it uninteresting to show these results.

The results regarding motor vehicle bicycle conflicts are not presented either. At the SGFC-junction the signal control separates bicycles from motor vehicle traffic. Because the simulated road users always adhere to the traffic signals it is impossible that motor vehicle bicycle conflicts occur in the SGFC-junction simulations. Because these conflicts can occur in the roundabout simulations the roundabout by default scores worse than the SGFC-junction making it uninteresting to present these results here.

Appendix I explains in further detail why graphs regarding motor vehicle only conflicts and motor vehicle bicycle conflicts are not shown in this report.

4.2.1. *Impact of motor vehicle and bicycle intensity*

For this comparison the motor vehicle intensity and bicycle intensity are variable. The share of traffic allocated to the main road is kept at 60% for both motor vehicles and bicycles for all these simulations. The share of left turning traffic is set at 15% for both modes. The average number of bicycle only TTC conflicts per bicycle is shown in Figure 19 below.

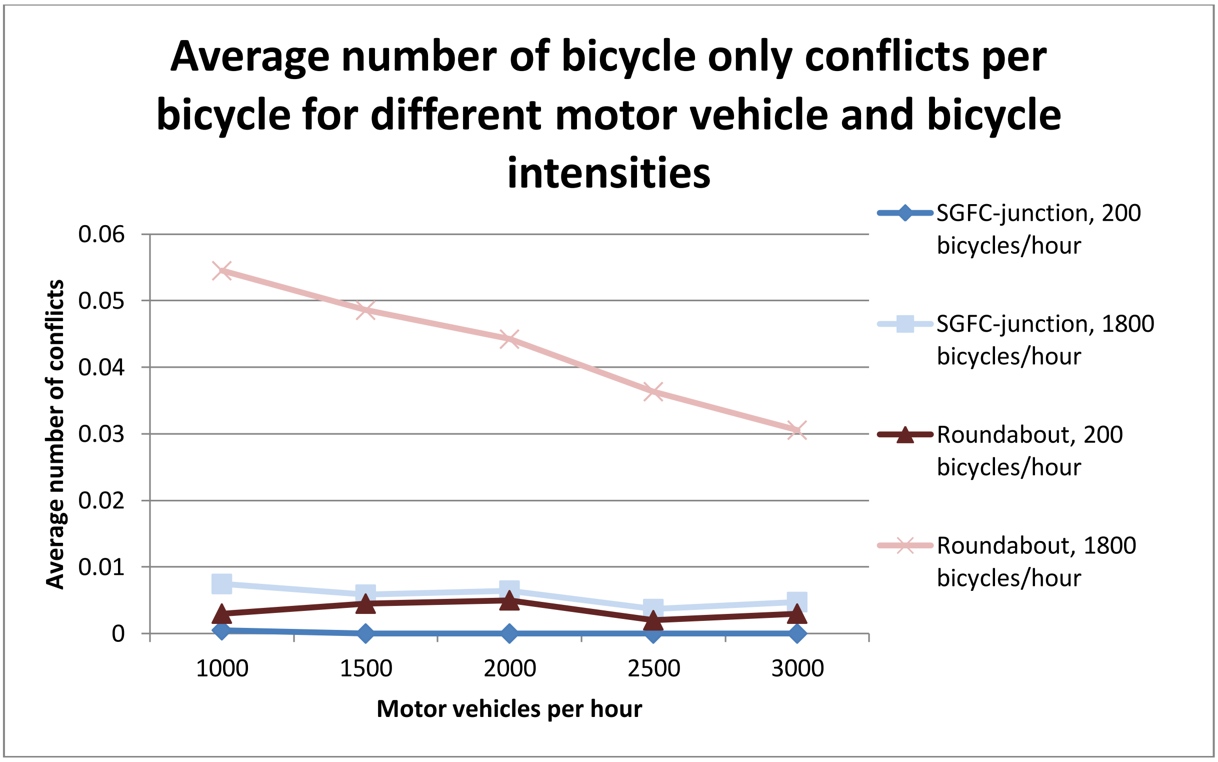

Figure 19: Average number of bicycle only conflicts per bicycle for different motor vehicle and bicycle intensities

SGFC-junction and roundabout compared

Figure 19 shows that the SGFC-junction has a lower amount of bicycle only conflicts than the roundabout in both the traffic demand scenarios with 200 bicycles per hour and with 1.800 bicycles per hour over all motor vehicle intensity levels. At some traffic demand scenarios the results of the ten simulation runs are not normally distributed and therefore the Mann Whitney u-test is used to check for statistically significant differences. Although the SGFC-junction has a lower average bicycle only conflicts rate than the roundabout for all traffic demand scenarios with 200 bicycles per hour the Mann Whitney u-test cannot support the differences at motor vehicle intensity levels of 1.000 mv/h, 2.500 mv/h and 3.000 mv/h. The differences between the junction types for the traffic demand scenarios with 1.800 bicycles per hour are statistically significant for all motor vehicle intensity levels.

An increase in bicycle intensity leads to an increase in the average amount of bicycle only conflicts per bicycle at the SGFC-junction. With an increased bicycle intensity the probability that bicycles come into conflict with each other when crossing the junction during the bicycle green phase increases. Compared to the roundabout this increase is very small. This is explained by the different distance bicycles from different junction approaches have to ride before reaching the conflict point. Bicycles from all directions start riding at the same time, because the get green light at the same time, but because of the difference in distance to the conflict point they arrive there at different moments in time meaning that most bicycles do not come into conflict with biycles from conflicting traffic flows. For a further illustration see the explanation in appendix A.

SGFC-junction lines
Both SGFC-junction lines follow a horizontal path, although the line for 1.800 bicycles per hour shows more variation. Because the traffic light control settings completely separate bicycles from motor vehicles it is logical that motor vehicle intensity does not impact the number of bicycle only conflicts.

Roundabout lines
There is a big difference between the two roundabout lines. With a higher bicycle intensity there is a higher probability that an approaching bicycle has to give way to another bicycle on the circulatory bicycle lane and therefore the probability for bicycle only conflicts increases as well.

The roundabout line for 200 bicycles per hour also follows a more or less horizontal path. Because bicycles have priority over motor vehicles in principle they do not have to brake for them and thus not cause rear end conflicts with other bicycles. It is therefore unexpected that the roundabout line for 1.800 bicycles per hour shows a decreasing trend, a higher intensity leads to a lower amount of bicycle only conflicts. An effect of a higher motor vehicle intensity is however the forming of congestion on the roundabout approaches and circulatory lane. Figure 9 shows that the average travel time for motor vehicles at the roundabout increases with motor vehicle intensity and thus indicates the increase in congestion. Under more congested conditions motor vehicles approach the roundabout at lower speeds which reduces the amount of conflicts between motor vehicles and bicycles which in turn reduces the probability for bicycle only conflicts.

Conclusions
The most noteworthy conclusions from Figure 19 about bicycle only conflicts over different motor vehicle intensities and bicycle intensities are:

- The SGFC-junction offers a lower average number of bicycle only conflicts than the roundabout
- At the SGFC-junction an increase in bicycle intensity leads to a small increase in the number of bicycle only conflicts
- At the SGFC-junction the motor vehicle intensity has no effect on the number of bicycle only conflicts
- At the roundabout an increase in bicycle intensity leads to a large increase in the number of bicycle only conflicts
- At the roundabout motor vehicle intensity has no effect on bicycle only conflicts at a low bicycle intensity
- At a high bicycle intensity an increase in motor vehicle intensity leads to a decrease in the number of bicycle only conflicts

4.2.2. Impact of motor vehicle intensity and motor vehicle main- / side road distribution

For this comparison the motor vehicle intensity and main- / side road distribution are variable. The bicycle intensity is set at 1.000 bicycles per hour. The share of bicycles allocated to the main road is kept at 60%. The share of left turning traffic is set at 15% for both modes. The average number of bicycle only TTC conflicts per bicycle is shown in Figure 20 below.

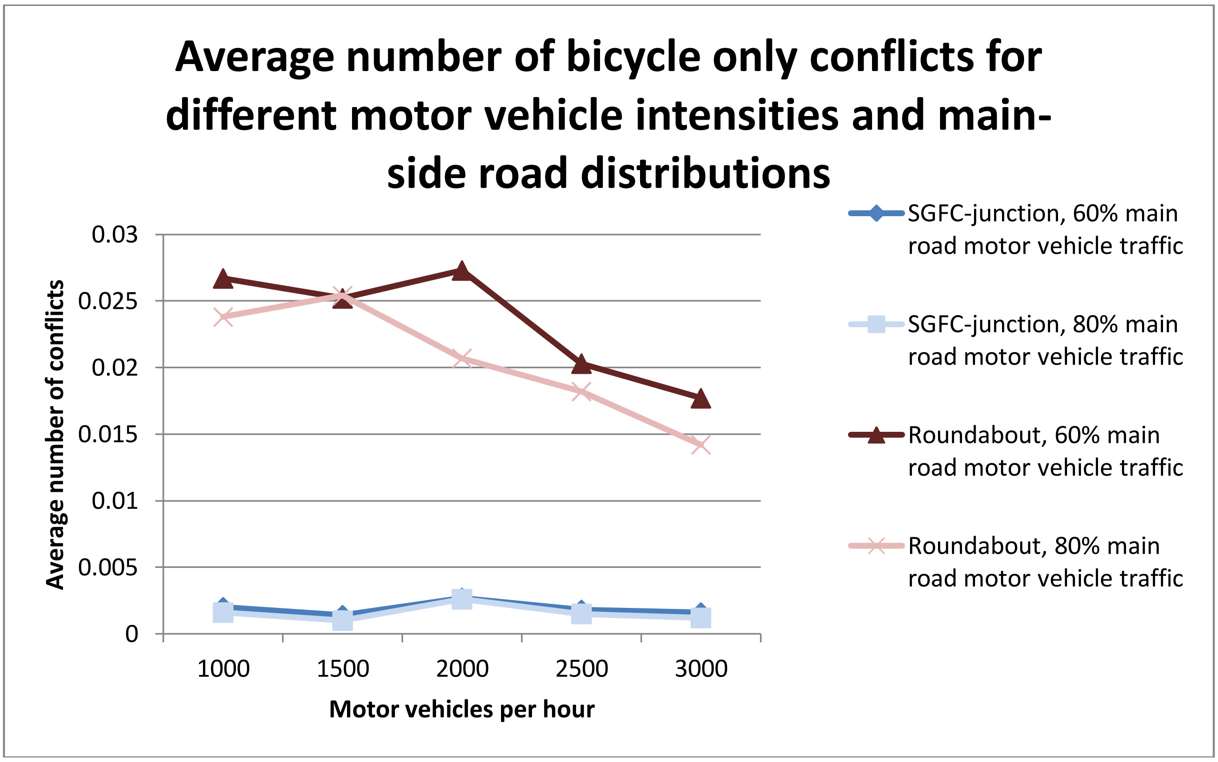

Figure 20: Average number of bicycle only conflicts for different motor vehicle intensities and main- / side road distributions

SGFC-junction and roundabout compared

Figure 20 shows that again the SGFC-junction scores a lower bicycle only conflict rate than the roundabout for all simulated traffic demand scenarios. Because not all the traffic demand scenario results are normally distributed the Mann Whitney u-test is used to check for statistical differences. The Mann Whitney u-test shows that the differences between the two junction types are statistically significant.

SGFC-junction lines

There is almost no difference between both SGFC-junction lines which means that the main- / side road distribution of motor vehicles does not influence the number of bicycle only conflicts. This is explained by the fact that different motor vehicle distributions do not impact the frequency of bicycle green phases much. Therefore the amount of bicycles that crosses the junction per bicycle green phase remains relatively constant over different motor vehicle intensity levels.

Just as in Figure 19 both the SGFC-junction lines follow a more or less horizontal path meaning that motor vehicle intensity does not influence the average number of bicycle only conflicts per bicycle.

Roundabout lines

Generally the roundabout line for the traffic demand scenarios with 80% main road motor vehicle traffic lies below the roundabout line with 60% main road motor vehicle traffic. Side road motor vehicle traffic conflicts with more bicycle traffic because there are more bicycles on the main road. Therefore shifting motor vehicle traffic from the side road to the main road reduces the probability that a motor vehicle comes into conflict with a bicycle. In turn this also decreases the probability that two bicycles come into a rear end conflict because the leading bicycle is forced to brake for a motor vehicle taking priority.

Both the roundabout lines show a decreasing trend. The same effects that caused the decreasing trend in the 1.800 bicycles per hour roundabout line in Figure 19 are likely causes of this downward slope. Because both lines now represent a medium intensity of 1.000 bicycles per hour the downward slope of the lines would logically be expected to be between the slope of the 1.800 bicycles roundabout line and the 200 bicycles roundabout line of Figure 19, which is also the case.

Conclusions

The most noteworthy conclusions from Figure 20 about bicycle only conflicts over different motor vehicle intensities and main-/ side road distributions are:

- The SGFC-junction offers a lower average number of bicycle only conflicts than the roundabout
- At the SGFC-junction motor vehicle main-/ side road distribution does not impact the number of bicycle only conflicts
- At the roundabout an increase in the share of main road motor vehicle traffic leads to a decrease in the number of bicycle only conflicts

4.2.3. Impact of bicycle intensity and motor vehicle main and side road distribution

For this comparison the bicycle intensity and motor vehicle main- / side road distribution are variable. The motor vehicle intensity is set at 2.000 motor vehicles per hour. The share of bicycles allocated to the main road is kept at 60%. The share of left turning traffic is set at 15% for both modes. The average number of bicycle only TTC conflicts per bicycle is shown in Figure 21 below .

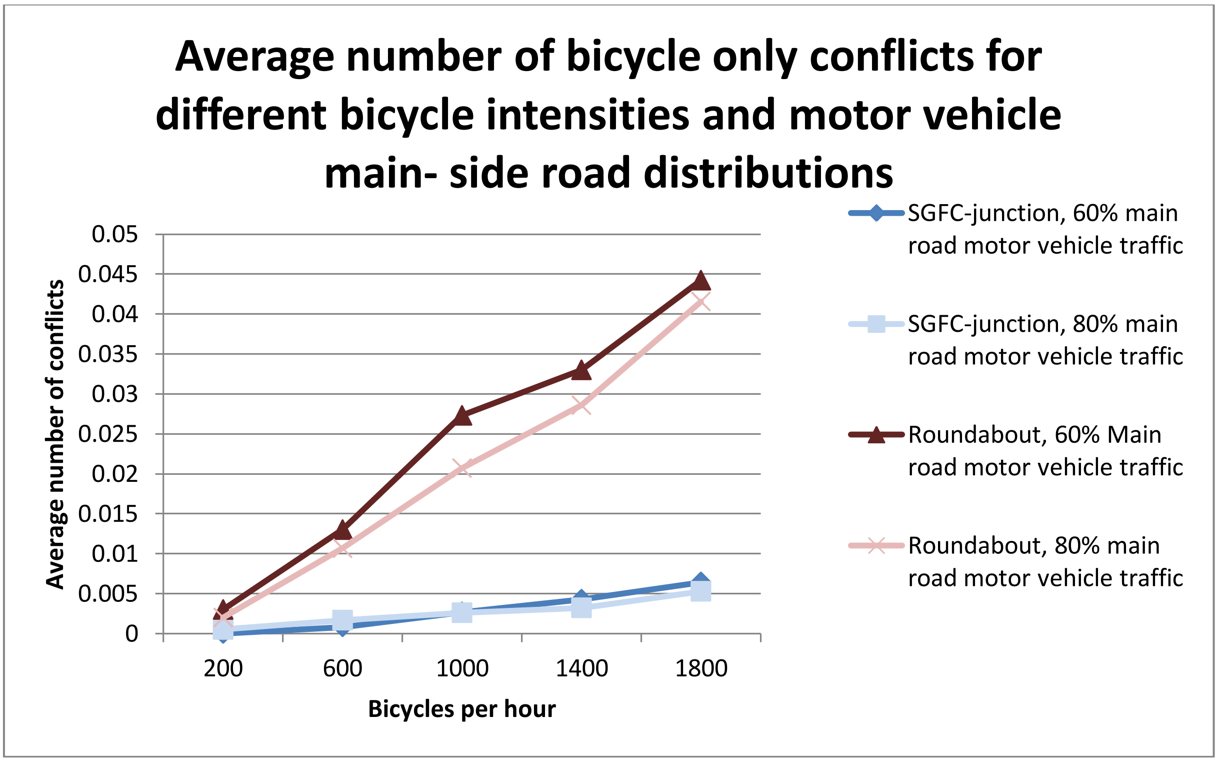

Figure 21: Average number of bicycle only conflicts for different bicycle intensities and motor vehicle main- / side road distributions

SGFC-junction and roundabout compared

Figure 21 shows that again the SGFC-junction scores the lowest number of average bicycle only conflicts per bicycle for every traffic demand scenario compared to the roundabout. The Mann Whitney u-test shows that these differences between the junction types are statistically significant for all bicycle intensity levels except the level of 200 bicycles per hour.

SGFC-junction lines

Both the SGFC-junction lines follow the same path. This means that the motor vehicle main- / side road distribution does not influence the average number of bicycle only conflicts per bicycle, see the explanation below Figure 20.

With an increase in bicycle intensity the average number of bicycle conflicts per bicycle increases only slightly, represented by the small slope of the SGFC-junction graphs. This is explained by the different distance bicycles from different junction approaches have to ride before reaching the conflict point, which means that an increase in the number of bicycles that crosses the junction per green phase only has a small effect on the number of bicycle only conflicts that occurs. This is further explained in appendix A.

Roundabout lines
The roundabout line for 80% main road motor vehicle traffic lies slightly lower than the roundabout line for 60% main road motor vehicle traffic. As explained in detail under Figure 20 transferring motor vehicle traffic from the side road to the main road decreases the probability that a motor vehicle comes into conflict with a bicycle which in turn decreases the probability that two bicycles come into conflict with each other.

Both roundabout lines have an upward slope as expected. A higher number of bicycles increases the probability that an approaching bicycle has to give way to a bicycle already on the roundabout and thus increases the probability that bicycles come into conflict with each other.

Conclusions
The most noteworthy conclusions from Figure 21 about bicycle only conflicts over different bicycle intensities and motor vehicle main-/ side road distributions are:

- The SGFC-junction offers a lower average number of bicycle only conflicts than the roundabout
- At the SGFC-junction an increase in bicycle intensity leads to a relatively small increase in the average number of bicycle only conflicts
- At the roundabout an increase in bicycle intensity leads to an increase in the average number of bicycle only conflicts

4.2.4. Impact of the share of left turning motor vehicles

For this comparison the motor vehicle intensity and left turn share are variable. The bicycle intensity is set at 1.000 bicycles per hour. The share of traffic allocated to the main road is kept at 60% for both modes. The share of left turning bicycles is set at 15%. The average number of bicycle only TTC conflicts per bicycle is shown in Figure 22 below .

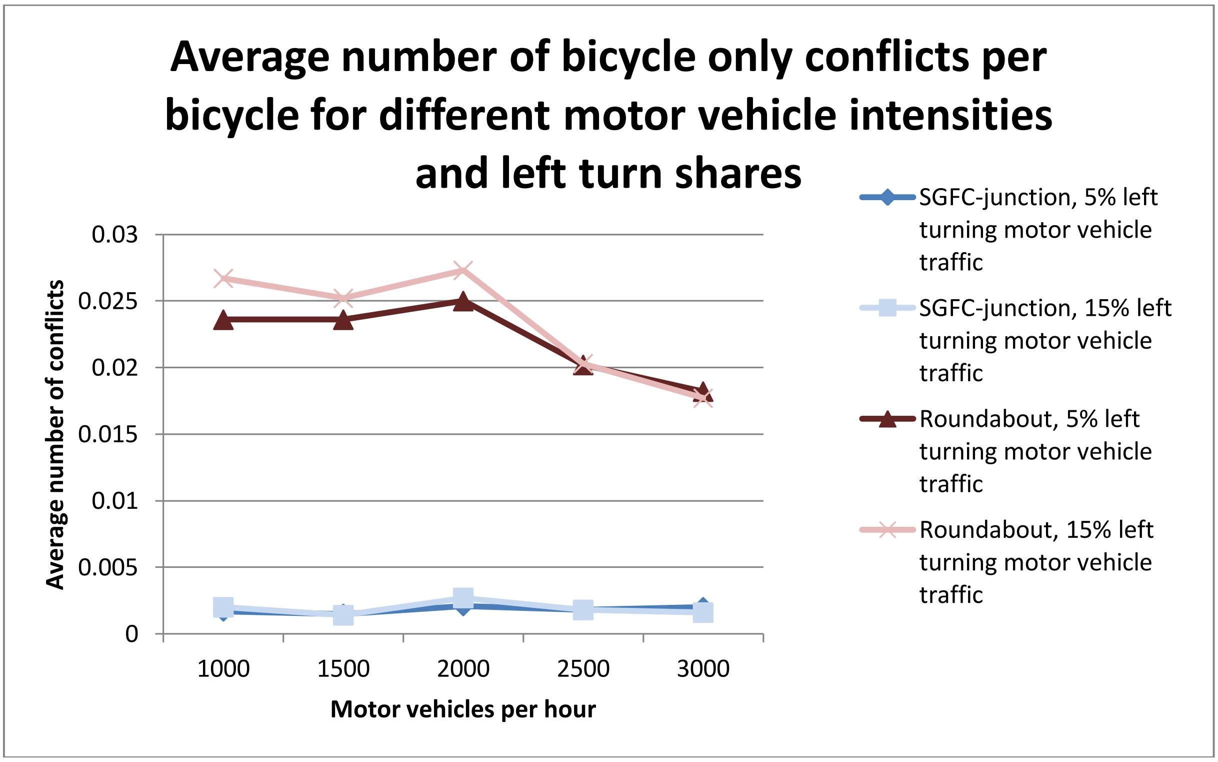

Figure 22: Average number of bicycle only conflicts per bicycle for different motor vehicle intensities and left turn shares

SGFC-junction and roundabout compared

From Figure 22 it is clear that again the SGFC-junction provides lower average bicycle only conflicts per bicycle than the roundabout. The Mann Whitney u-test results show that these differences between the two junction types are statistically significant.

SGFC-junction lines

Both SGFC-junction lines follow the same path which means that the left turning motor vehicle share does not influence the average number of bicycle conflicts. A change in the left turning motor vehicle share results in a redistribution of green time allocated to different motor vehicle green phases. The distribution of green time over motor vehicles and bicycles however remains constant. Therefore the amount of bicycles crossing the junction during a green phase also is unaffected by a change in the motor vehicle left turn share.

Both SGFC-junction lines follow a horizontal path meaning that motor vehicle intensity does not impact the number of bicycle only conflicts. This is consistent with the results presented in Figure 19and Figure 20.

Roundabout lines

As expected the two roundabout lines generally follow the same path. All motor vehicles potentially interact with bicycles when entering and when exiting the roundabout regardless of the turning movement they make. Therefore the number of bicycle only conflicts remains the same even when the turn shares of motor vehicles change.

Again both roundabout lines seem negatively related to motor vehicle intensity. This is likely explained by increasing motor vehicle congestion at higher motor vehicle intensities, see below Figure 19 and Figure 20 for further explanation.

Conclusions

The most noteworthy conclusions from Figure 22 about bicycle only conflicts over different motor vehicle intensities and motor vehicle left turn shares are:

- The SGFC-junction offers a lower average number of bicycle only conflicts per bicycle compared to the roundabout
- The left turn share of motor vehicles does not impact the number of bicycle only conflicts at the SGFC-junction
- At the roundabout the left turn motor vehicle share seems to slightly impact the number of bicycle only conflicts

4.2.5. Impact of bicycle intensity and motor vehicle left turn shares

For this comparison the bicycle intensity and motor vehicle left turn share are variable. The motor vehicle intensity is set at 2.000 motor vehicles per hour. The share of traffic allocated to the main road is kept at 60% for both modes. The share of left turning bicycles is set at 15%. The average number of bicycle only TTC conflicts per bicycle is shown in Figure 23 below .

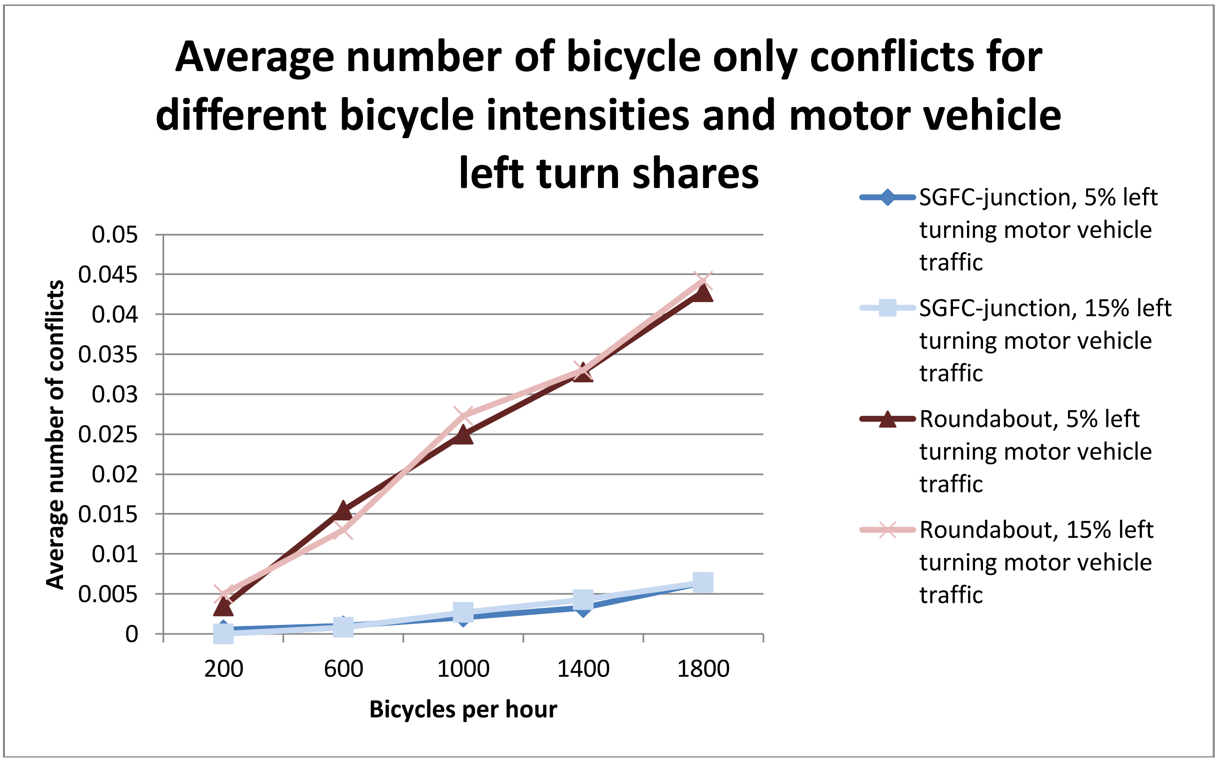

Figure 23: Average number of bicycle only conflicts per bicycle for different bicycle intensities and motor vehicle left turn shares

SGFC-junction and roundabout compared

Figure 23 shows that again the SGFC-junction offers lower average numbers of bicycle only conflicts than the roundabout for all traffic demand scenarios. Except for the intensity level of 200 bicycles per hour all differences between the junction types are statistically significant according to the Mann Whitney u-test.

SGFC-junction lines

There is almost no difference between both SGFC-junction lines meaning that the motor vehicle left turn share has no impact on the number of bicycle only conflicts. This is consistent with the results found in Figure 22.

The bicycle intensity only slightly affects the safety score of the SGFC-junction which is a result of the different distances bicycles from opposing arms need to travel before the reach the conflict point, see appendix A for a detailed explanation. This is consistent with the results presented in Figure 21.

Roundabout lines
An increase in bicycle intensity has a bigger influence on the bicycle only conflicts at the roundabout, represented by the steeper slope of the roundabout graphs compared to the graphs for the SGFC-junction. As bicycle intensity increases the probability that a bicycle approaching the roundabout has to give way to a bicycle already on the roundabout increases. With it the probability of a bicycle only conflict also goes up. The results here are consistent with the results presented in Figure 21.

Both roundabout lines have similar values meaning that the motor vehicle left turn share does not impact the number of bicycle only conflicts. This is consistent with the results presented in Figure 22.

Conclusion
The most noteworthy conclusions from Figure 22 about bicycle only conflicts over different motor vehicle intensities and motor vehicle left turn shares are:

- The SGFC-junction offers a lower average number of bicycle only conflicts per bicycle compared to the roundabout

4.3. Results concerning emissions

To compare the two junction designs regarding environmental impact the amount of emissions is used. Each sub-paragraph shows a graph with the results regarding the average total carbon emissions. Total carbon emissions are the combined CO and CO_2 emissions. The results regarding emissions of NO_x and PM_{10} are not presented here because these graphs show similar demand scenarios to the graphs of the total carbon emissions. An example is presented in appendix I.

Obtaining results from the emissions module turned out to be a difficult process. After the first try the resulting graphs were completely illogical. This was the result of miscommunications between the S-Paramics traffic model and the AIRE emissions model. In S-Paramics each vehicle type is defined within a vehicle category. In this study the vehicle type bicycle is defined under the vehicle category car. This means that the AIRE emissions module also tried to allocate emission values to all bicycles. Because the default amount of vehicle types under the category car was increased with one new type the AIRE emission model generated an error for every bicycle data line it encountered. Upon recommendation of Grontmij (Dutch distributor of S-Paramics software) and SIAS (Developer of S-Paramics and AIRE software) some new simulation runs were conducted with the vehicle type bicycles defined under the category electric tram. Given that the electric tram does not produce emissions AIRE does not have to process data of this vehicle category. However the errors remained. The next step was to filter the S-Paramics output before it was entered into AIRE. This filtering removed all data regarding bicycles from the S-Paramics before it was implemented into AIRE. This solved the previously error messages and led to the results presented in the remainder of this chapter.

However these results still are illogical, for two reasons. First the emissions per vehicle decrease when motor vehicle intensity increases. An increase in motor vehicle demand increases the amount of congestion resulting in lower average speeds, which is demonstrated by the higher travel times displayed in section 4.1 above. A motor vehicle driving at a slower speed has to overcome less air resistance and would therefore produce less emissions than a fast driving vehicle. However congestion normally leads to a higher variance in speed which results in an overall increase in emissions per vehicle. This argument is explained further in the sub-paragraphs below. The second type of illogical result is that sometimes the total emissions decrease when the total number of motor vehicles increases.

The hypothesis that the first type of illogical emission results are caused by the fact that motor vehicles enter the simulation network at a speed of 15 km/h was tested. For a high average speed, e.g. 40 km/h, and thus low emissions (see explanation in the sub-paragraphs below) the modelled vehicle initially needs to greatly accelerate resulting in high emissions. The test was done by leaving emissions produced at the outer most links of the network, where this initial acceleration takes place, out of the calculations. Of course this led to lower overall emission values, but the course of the graphs remained the same which means that this does not explain the illogical results.

Another potential cause for the illogical reduction in emission following increased congestion would be that the congestion does not lead to an increase in the variance in speed. In that case the lower average speed would lead to a lower level of emissions per vehicle. The average motor vehicle speed and speed variance are calculated based on the 'CarpositionsXXXXXX.csv' output from S-Paramics. For every time step this file contains, amongst other things, the instantaneous speed of every vehicle. These values are imported into Matlab to calculate the average motor vehicle speed and the speed variance. The results for a few key traffic demand scenarios are shown in Table 14 below.

Overview of average speed, speed variance and emissions over different demand scenarios				
Bicycle intensity = 200 bicycles/hour, main road traffic share = 60% and left turn share = 5% in all cases				
Junction type	**Motor vehicle intensity [mv/h]**	**Mean motor vehicle speed [km/h]**	**Motor vehicle speed variance []**	**Average CO2 emissions per motor vehicle [mg]**
Roundabout	1000	37.1	242.3	57655
	1500	31	302.7	59685
	2000	17.8	292.7	51293
	2500	13.7	222.7	40960
	3000	12.4	195.3	32643
SGFC-junction	1000	17.8	400.2	66099
	1500	7.6	177.9	48904
	2000	8.9	199.9	37960
	2500	9.3	198.1	31314
	3000	10.8	217	26724

Table 14: Overview of average motor vehicle speed, speed variance and emissions over different demand scenarios

For the roundabout there seems to be a correlation between the speed variance and the average CO_2 emissions per vehicle: when the variance decreases the emissions decrease as well. However for the SGFC-junction there is no clear relation between speed variance and emissions. Therefore looking into the variance of motor vehicle speeds does not offer an explanation for the illogical emission results.

Within the time constraints of this study it is not possible to further investigate this matter.

4.3.1. Impact of motor vehicle and bicycle intensity

For this comparison the motor vehicle intensity and bicycle intensity are variable. The share of traffic allocated to the main road is kept at 60% for both motor vehicles and bicycles for all these simulations. The share of left turning traffic is set at 15% for both modes. The average total carbon emissions per vehicle are shown in Figure 24 below.

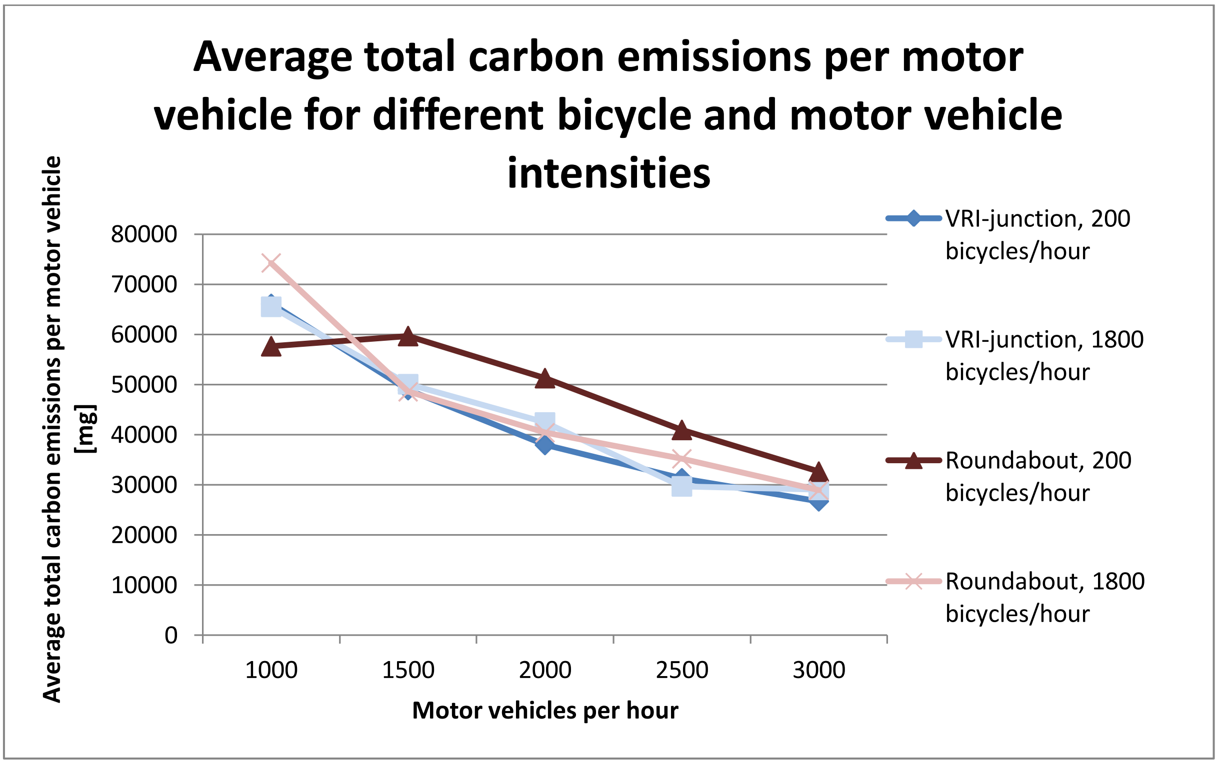

Figure 24: Average total carbon emissions per motor vehicle for different motor vehicle and bicycle intensities

Figure 24 shows that all lines follow a downward trend. This means that the emissions per vehicle decrease as the motor vehicle intensity increases.

The roundabout line for a bicycle intensity of 200 bicycles per hour is analysed in greater detail to assess the validity of the results. Figure 9 which shows the motor vehicle travel time for the same traffic demand scenarios as Figure 24 is used to determine the average motor vehicle travel times. At a motor vehicle intensity of 1.000 mv/h the average motor vehicle travel time is 122 seconds. At the motor vehicle intensity level of 3.000 mv/h the average motor vehicle travel time is 185 seconds. Given that the travel distance through the network is 625 meter the average speeds for these situations are: 18,4 km/h and 12,2 km/h.

Several studies show that the relation between motor vehicle emissions and vehicle speed is bowl shaped for example: (Boer & Vermeulen, 2004), (Anable, Mitchell, & Layberry, 2006), (Federal Highway Administration, 2006). Starting at a speed of 0 km/h an increase in speed leads to lower vehicle emissions up to around a speed of 70 km/h where the emissions are lowest. From that speed an increase in speed results in an increase of emissions. Figure 25 below, adapted from the study by Boer and Vermeulen, illustrates this (Boer & Vermeulen, 2004).

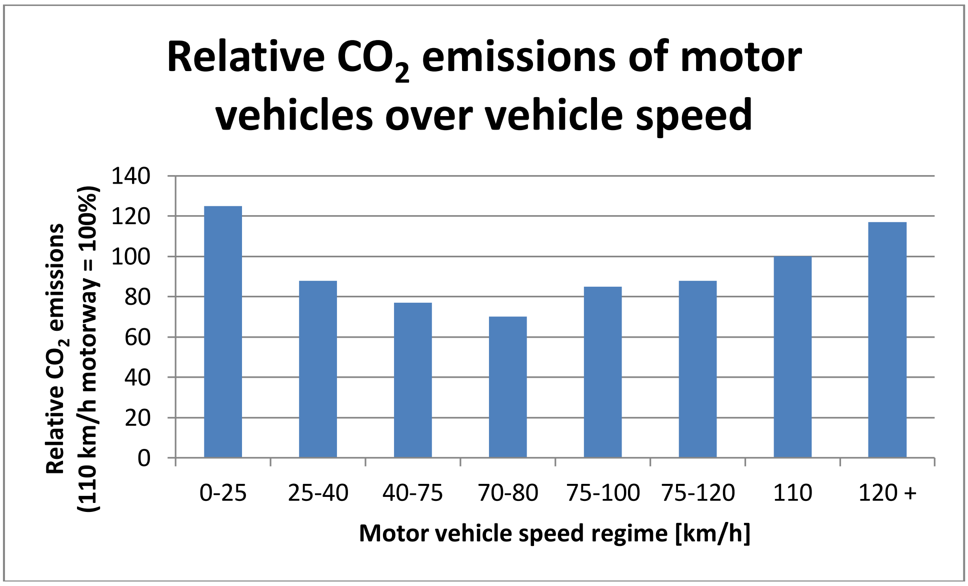

Figure 25: Relative CO_2 emissions of motor vehicles over vehicle speed

Although the average speeds that came out of the simulation, 18,4 km/h and 12,2 km/h, both fall in the first bar of Figure 25 it is likely that the decrease in speed should have led to an increase of vehicle emissions given the trend represented in Figure 25.

To further investigate the validity of the emission outputs Figure 26 below shows the average Total total emissions per model run, so not averaged out per vehicle, for the same traffic demand scenarios.

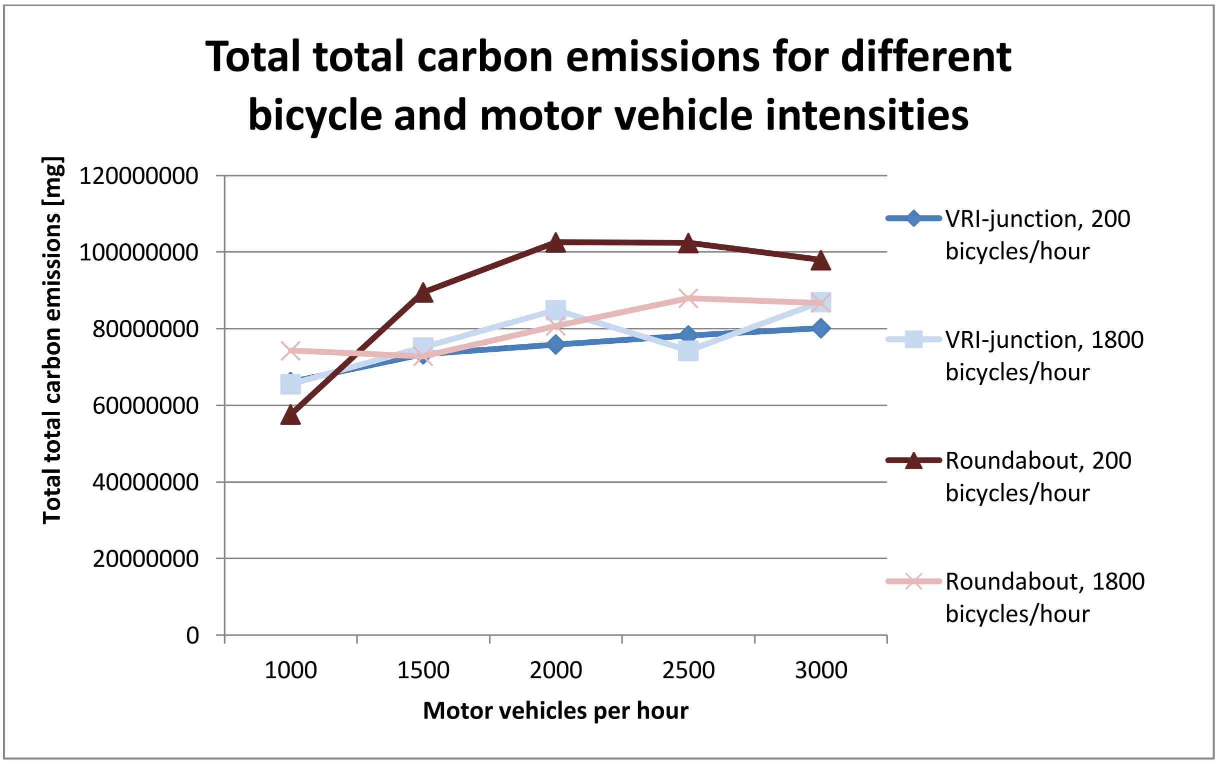

Figure 26: Total total carbon emissions per run for different bicycle and motor vehicle intensities

In both graphs the roundabout line for the traffic demands with 200 bicycles per hour has the highest emissions except for the lowest motor vehicle intensity where the roundabout line for traffic demands with 1.800 bicycles per hour is highest. The roundabout line for 200 bicycles per hour shows a decrease from the motor vehicle intensity level of 2.000 mv/h to the level of 3.000 mv/ h. The fact that the total amount of emissions decreases even though the amount of motor vehicles increases with 50% goes against common sense and further increases the doubts about the correctness of the emission model outputs.

4.3.2. Impact of motor vehicle main and side road distribution

For this comparison the motor vehicle intensity and main- / side road distribution are variable. The bicycle intensity is set at 1.000 bicycles per hour. The share of bicycles allocated to the main road is kept at 60%. The share of left turning traffic is set at 15% for both modes. In order to be able to check if the results are logical the average total carbon emissions per run, so not the emissions averaged per motor vehicle, are shown in Figure 27 below.

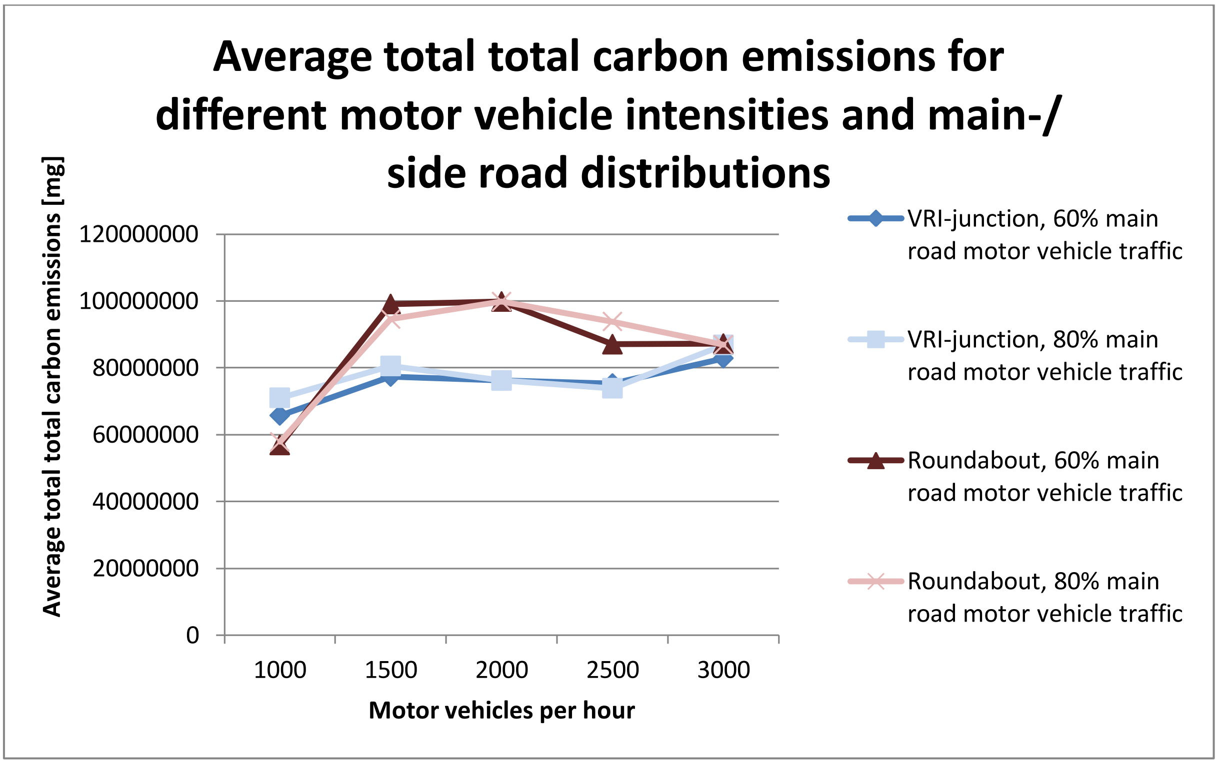

Figure 27: Average total carbon emissions per run for different motor vehicle intensities and main-/ side road distributions

Again the emission lines in Figure 27 show a downward trend over an increase in motor vehicle intensity. Both roundabout lines decrease between the motor vehicle intensity levels of 2.000 mv/ h and 3.000 mv/h. Both SGFC-junction lines show a decrease between the motor vehicle levels of 1.500 mv/h and 2.500 mv/h. So for these traffic demand scenarios as well the results go against common sense and cannot be used to judge junction performance.

4.3.3. Impact of motor vehicle main and side road distribution and bicycle intensity

For this comparison the bicycle intensity and motor vehicle main- / side road distribution are variable. The motor vehicle intensity is set at 2.000 motor vehicles per hour. The share of bicycles allocated to the main road is kept at 60%. The share of left turning traffic is set at 15% for both modes. The average carbon emissions per motor vehicle are shown in Figure 28 below.

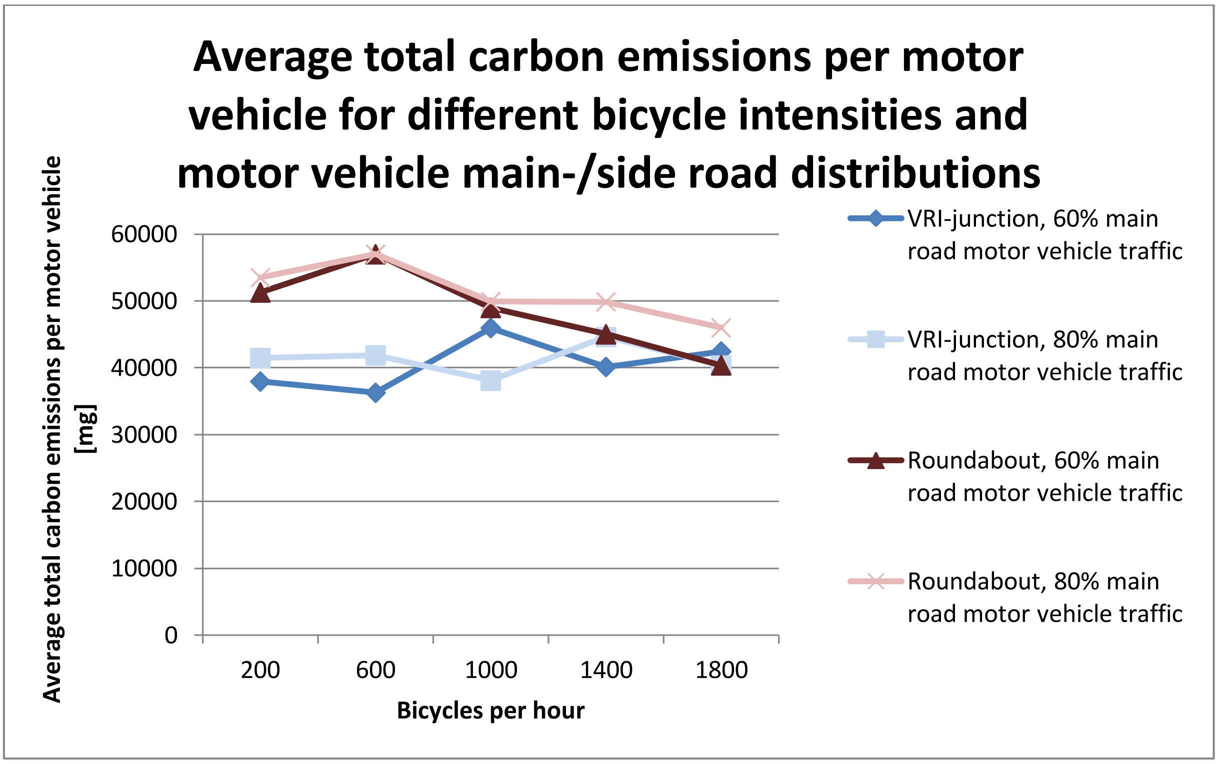

Figure 28: Average total carbon emissions per run for different bicycle intensities and motor vehicle main-/ side road distributions

Figure 28 shows that both roundabout lines have a decreasing trend with increasing bicycle intensity. The motor vehicle travel times for these traffic demand scenarios are displayed in Figure 13. The travel times displayed in this figure are shown in Table 15 below.

Average motor vehicle speeds for specified points			
Graph line	Bicycle intensity [b/h]	motor vehicle travel time [s]	average motor vehicle speed [km/h]
Roundabout 60% main road motor vehicle traffic	200	116	19.4
	1800	235	9.6
Roundabout 80% main road motor vehicle traffic	200	118	19.1
	1800	159	14.2
SGFC-junction 60% main road motor vehicle traffic	200	226	10.0
	1800	238	9.5
SGFC-junction 80% main road motor vehicle traffic	200	194	11.6
	1800	210	10.7

Table 15: Average motor vehicle speeds for specific points corresponding with the above emissions graph

For both the roundabout lines the average motor vehicle speeds decrease with an increase in bicycle intensity. This should lead to an increase in average motor vehicle emissions given the previously cited studies (Boer & Vermeulen, 2004), (Anable, Mitchell, & Layberry, 2006) and (Federal Highway Administration, 2006).

For the SGFC-junction lines the emission values seem to be correct. In both cases the average motor vehicle speed decreased slightly with an increase in bicycle intensity and the average motor vehicle emissions show a slight increase.

Because the motor vehicle intensity remains constant over the tested traffic demand scenarios it is not possible to test the validity of the emission results by looking at the total emissions.

4.3.4. Impact of motor vehicle intensity and motor vehicle left turning share

For this comparison the motor vehicle intensity and left turn share are variable. The bicycle intensity is set at 1.000 bicycles per hour. The share of traffic allocated to the main road is kept at 60% for both modes. The share of left turning bicycles is set at 15%. The total total carbon emissions per run are shown in Figure 29 below.

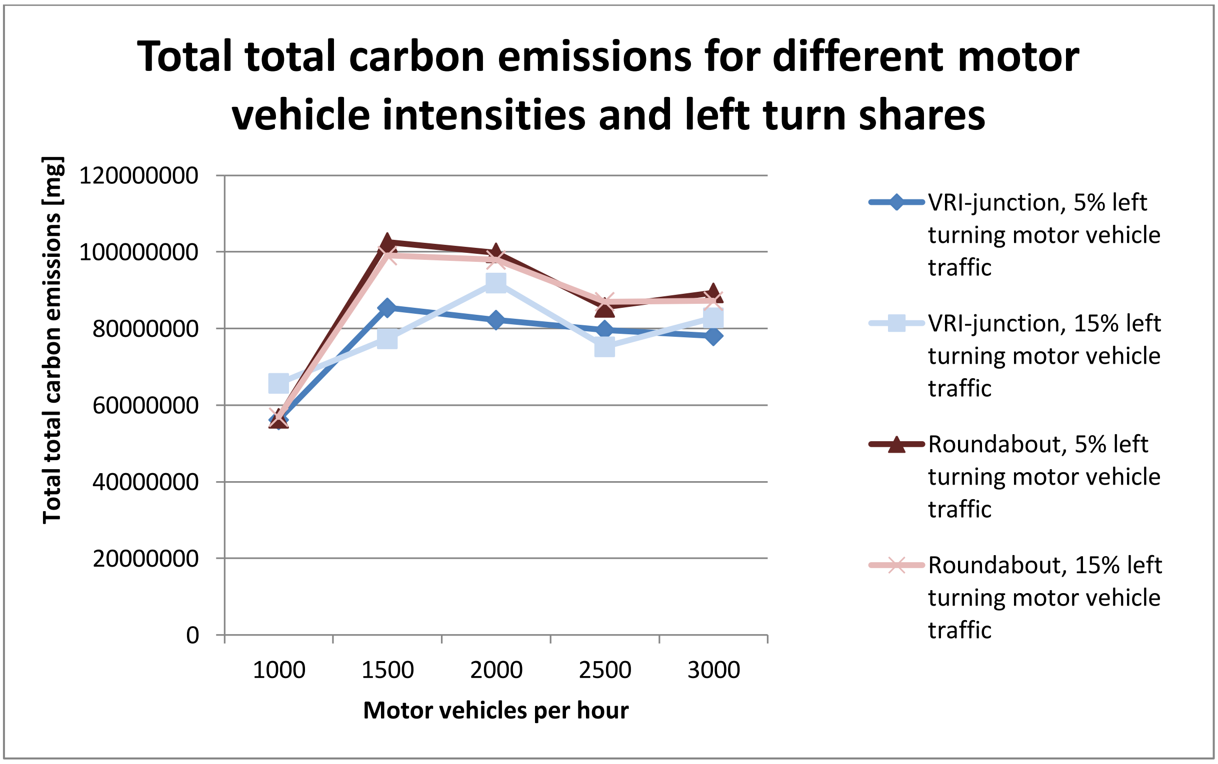

Figure 29: Total total carbon emissions per run for different motor vehicle intensities and left turn shares

Figure 29 shows that all lines at some point show a decreasing trend with a motor vehicle increase. This means that the found results do not give a reliable representation of real world conditions. An increase in the amount of motor vehicles should lead to an increase of the total emissions. Furthermore at least for the roundabout an increase in motor vehicles decreases the average motor vehicle speed, as shown in Figure 15, which indicates an increase in congestion. This factor should increase the amount of emissions per vehicle and should therefore mean that the total amount of emissions should increase even more. At the SGFC-junction this argumentation is not always applicable because the adding of a dedicated turn lane may actually improve traffic flow despite the increase in intensity. So in short the emission results again prove to be unsuitable to use.

4.3.5. Impact of bicycle intensity and motor vehicle left turn shares

For this comparison the bicycle intensity and motor vehicle left turn share are variable. The motor vehicle intensity is set at 2.000 motor vehicles per hour. The share of traffic allocated to the main road is kept at 60% for both modes. The share of left turning bicycles is set at 15%. The total total carbon emissions per run are shown in Figure 30 below.

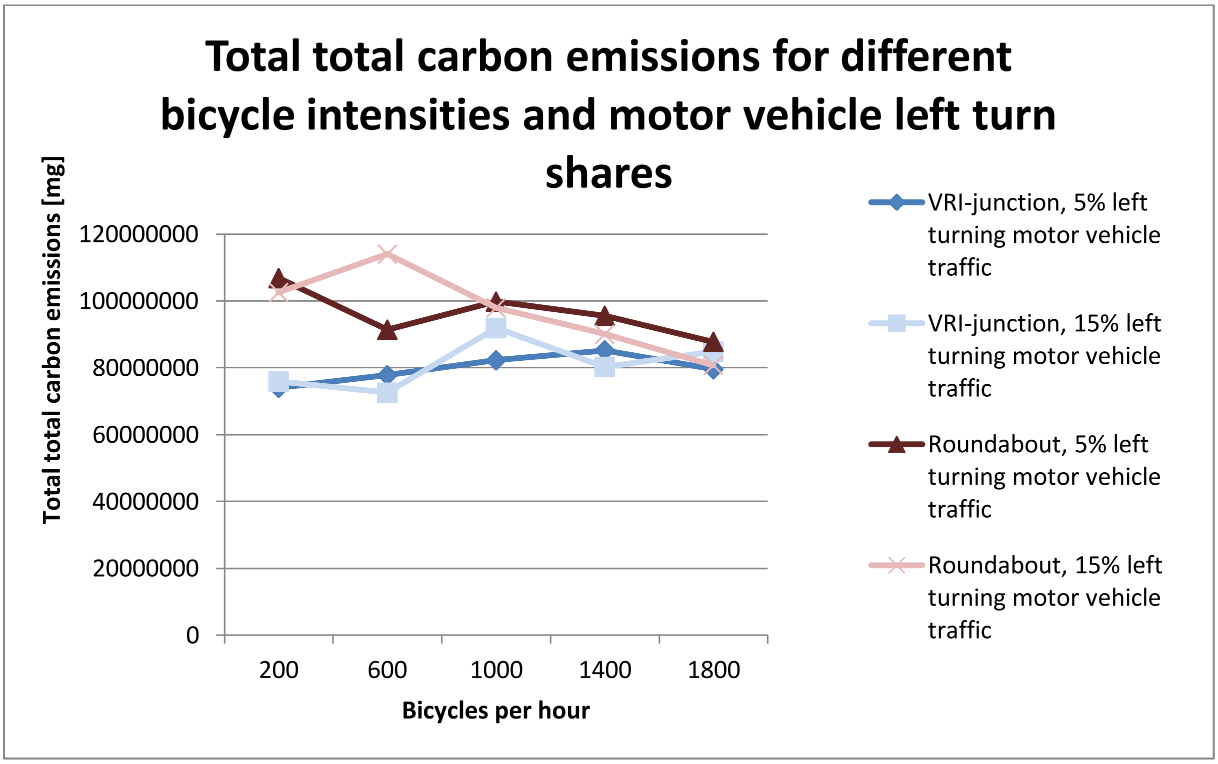

Figure 30: Total total carbon emissions per run for different bicycle intensities and motor vehicle main-/ side road distributions

Figure 30 shows a similar picture as Figure 28 with decreasing roundabout lines and slightly increasing SGFC-junction lines. Just as before an increase in bicycle intensity leads to an increase in motor vehicle travel time and thus a decrease in motor vehicle travel time. This should lead to an increasing emissions graph which is the opposite of the presented result. Therefore the emission results are not suitable to use in the junction comparison.

4.4. Overview of results

This paragraph contains an overview of the results and discusses the general findings of this study. The general relative rankings of the junction types per performance indicators are shown in Table 16 below.

General relative position of junction type		
Junction performance type	Junction design type	
	SGFC-junction	Roundabout
Motor vehicle travel time	-	+
Bicycle travel time	-	+
Motor vehicle only conflicts	+	-
Bicycle only conflicts	+	-
Motor vehicle bicycle conflicts	+	-
Total Carbon emissions	No Result	No Result
NO_x emissions	No Result	No Result
PM_{10} emissions	No Result	No Result

Table 16: Overview of the general relative ranking of both junction types

4.4.1. Impact of traffic demand scenario variables

Table 18 shows how the relative attractiveness of the roundabout compared to the SGFC-junction changes as a reaction to an increase in the mentioned traffic demand scenario variable. Table 17 contains an example of how Table 18 is determined. Table 17 shows that at the lowest motor vehicle intensity the motor vehicle travel time at the roundabout is only 47% of the motor vehicle travel time at the SGFC-junction. Only looking at the motor vehicle travel time the roundabout is by far the more attractive junction type. At the highest motor vehicle intensity the motor vehicle travel time at the roundabout is 94% of that of the SGFC-junction. Over the increasing motor vehicle intensity the attractiveness of the roundabout compared to the SGFC-junction has greatly decreased. Therefore the top left cell in Table 18 contains a '--'.

Change in relative attractiveness of roundabout compared to SGFC-junction					
	Motor vehicle intensity				
	1.000	1.500	2.000	2.500	3.000
Roundabout motor vehicle travel time	57.6	69.4	115.8	*144.9 **	*174.0 **
SGFC-junction motor vehicle travel time	121.7	268	225.9	215.1	184.6
Relative attractiveness roundabout	47%	26%	51%	67%	94%

Table 17: Change in attractiveness of the roundabout caused by an increase in the traffic demand variable.

The changes in the attractiveness of the roundabout compared to the SGFC-junction in Table 18 are shown because it helps to indicate which of the used traffic demand scenario variables have a big impact on junction performance and which variables can be left out of future studies because their impact is very small. A '--' means that the attractiveness of the roundabout compared to the SGFC-junction decreases greatly (>10% over variable range) as the respective demand scenario variable is increased. A '-' means a small decrease (between 5% and 10% over variable range) in roundabout attractiveness and a '0' means that the difference between the junctions remains almost constant (<5% over variable range).

Effect of an increase in the demand variable on attractiveness of the roundabout compared to the SGFC-junction								
Traffic demand variable	Performance indicator							
	MV TT	Bicycle TT	#MV only conflicts	#Bicycle only conflicts	#MV bicycle conflicts	TC emissions	NO_x emissions	PM_{10} emissions
mv intensity	--	0	++	--	--	No result	No result	No result
bicycle intensity	--	+	0	++	++	No result	No result	No result
mv main road traffic share	--	+	--	++	++	No result	No result	No result
b main road traffic share	0	+	0	++	+	No result	No result	No result
mv left turn share	++	0	++	+	0	No result	No result	No result
b left turn share	0	0	0	--	--	No result	No result	No result

Table 18: Change in difference between junction types' performance caused by an increase in the traffic demand variable

In general the roundabout offers better travel times for motor vehicles than the SGFC-junction, see Table 16. Table 18 shows that there are three demand scenario variables that greatly reduce the relative attractiveness of the roundabout compared to the SGFC-junction as they are increased. This means that the higher the motor vehicle intensity, bicycle intensity and / or motor vehicle main road share the higher the probability that the SGFC-junction offers a lower motor vehicle travel time. The opposite is true for the motor vehicle left turn share, an increase only increases the attractiveness of the roundabout compared to the SGFC-junction. Given that the roundabout already yields lower motor vehicle travel times at low motor vehicle left turn shares this means that the roundabout will never be outperformed by the SGFC-junction solely by increasing the motor vehicle left turn share.

The result that the roundabout offers better travel times for bicycles than the SGFC-junction is a very robust outcome. As Table 18 shows increasing any of the demand scenario variables beyond the tested value will not impact or even increase the advantage that the roundabout has over the SGFC-junction.

Regarding the amount of motor vehicle only conflicts the SGFC-junction always scored much better for the tested traffic demand scenarios, which was the reason to not present these graphs as explained in appendix I. Table 18 shows that increasing the share of main road motor vehicle traffic decreases the advantage that the roundabout has over the SGFC-junction. It is possible that increasing the main road motor vehicle share would result in the roundabout having a lower motor vehicle only conflict score than the SGFC-junction. Increasing the motor vehicle intensity or motor vehicle left turn share beyond the tested values would only increase the advantage that the SGFC-junction has over the roundabout.

For the tested traffic demand scenarios the SGFC-junction offers the lowest amount of bicycle only conflicts. Only an increase in motor vehicle intensity decreases the advantage that the SGFC-junction has over the roundabout and could eventually result in the roundabout offering a better bicycle safety than the SGFC-junction. An increase in any of the five other demand scenario variables only increases the difference between the junction types.

Similar principles apply to the motor vehicle bicycle conflicts score. The SGFC-junctions scored better for all tested demand scenarios and only an increase in motor vehicle intensity beyond the tested values could eventually lead to a different ranking of the junction types. Apart from the motor vehicle left turn share an increase in any of the other four variables leads to an increase in the difference between both junction designs.

5. Conclusions and discussion

In this chapter conclusions from the results as presented in chapter 4 are presented. Paragraphs **Fout! Verwijzingsbron niet gevonden.**, 0 and 5.3 each discuss conclusions regarding one of the three performance criteria: throughput, traffic safety and environmental impact. Each of those paragraphs compares the outcomes of this study with earlier findings from literature. Paragraph 5.4 presents the overall conclusions on the impact each of the six traffic demand scenario variables has on junction performance. The final part of this chapter, i.e. paragraph 5.5, presents the most important limitations of this research.

5.1. Throughput

As already explained in paragraph 4.1.1 the simulation results of the roundabout are influenced by the fact that at high motor vehicle intensities the queues completely filled the network. In order to provide a more accurate answer to the research question new travel time values are estimated for the roundabout. These estimations are linear extrapolated values based on the travel times at the motor vehicle levels of 1.000 mv/h and 2.000 mv/h. At these intensity levels the limited network size has no large impact. Table 19 below shows the travel time values for different motor vehicle and bicycle intensities. For these results the bicycle main road share is fixed at 60% and both left turn shares are 15%. All extrapolated values are denoted with a *. In Table 19 are six demand scenarios where the SGFC-junction provides a lower travel time than the roundabout, visualised with bold numbers.

Motor vehicle travel time in seconds						
Main road motor vehicle traffic share 60%						
Bicycle intensity	Junction type	Motor vehicle intensity				
		1.000	1.500	2.000	2.500	3.000
200	Roundabout	57.6	69.4	115.8	*144.9 **	*174.0 **
	SGFC-junction	121.7	268	225.9	215.1	184.6
600	Roundabout	59.1	82.7	133.1	*170.1 **	***207.1 ***
	SGFC-junction	122.8	278	234.1	235.7	190.8
1.000	Roundabout	62.1	112.3	159.1	*207.6 **	***256.1 ***
	SGFC-junction	123.7	276.9	236.1	242.9	192.9
1.400	Roundabout	82.5	162.3	190.8	*245 **	***299.1 ***
	SGFC-junction	122.4	278.6	237.9	247.1	194.3
1.800	Roundabout	**144.3**	217.9	234.6	***279.8 ***	***324.9 ***
	SGFC-junction	120.7	274.1	238	250.3	192.3

Table 19: Travel times in seconds for different motor vehicle and bicycle intensities. For the highest two motor vehicle intensity levels roundabout travel times are estimated. An * marks an extrapolated value.

Table 19 above shows six situations where the SGFC-junction scores lower travel times than the roundabout. The demand scenarios where the SGFC-junction scores better are characterised by high motor vehicle intensities and / or high bicycle intensities, except for the demand scenario of 1.000 mv/h and 1.800.

Table 20 below presents the extrapolated values for different traffic demand scenarios with 80% main road motor vehicle traffic. Again the bicycle main road share is fixed at 60% and both left turn shares are fixed at 15%.

Motor vehicle travel time in seconds						
Main road motor vehicle traffic share 80%						
Bicycle intensity	**Junction type**	**Motor vehicle intensity**				
		1.000	**1.500**	**2.000**	**2.500**	**3.000**
200	**Roundabout**	59.3	92.8	117.6	*146.8 **	***175.9 ****
	SGFC-junction	127.6	195.7	193.7	182.6	148.8
600	**Roundabout**	60.8	100.3	122.0	*152.6 **	***183.2 ****
	SGFC-junction	127.6	207.8	206.2	203.9	154.3
1.000	**Roundabout**	64.8	111.2	129.9	*162.5 **	***195 ****
	SGFC-junction	132.6	201.7	209.2	214.2	157.3
1.400	**Roundabout**	77.2	126.4	140.7	*172.5 **	***204.2 ****
	SGFC-junction	126.4	206.0	210.0	219.1	162.2
1.800	**Roundabout**	111.3	149.7	159.1	*183.0 **	***206.9 ****
	SGFC-junction	125.9	205.2	209.7	222.2	159.6

Table 20: Travel times in seconds for different motor vehicle and bicycle intensities. For the highest two motor vehicle intensity levels roundabout travel times are estimated. An * marks an extrapolated value.

Table 20 above shows five demand scenarios where the SGFC-junction scores lower travel times than the roundabout. Again these demand scenarios are characterised by a high motor vehicle intensity.

Figure 43 in appendix I.1 shows that the bicycle main-/side road distribution hardly impacts motor vehicle travel times at the roundabout. Traffic demand scenarios with 80% bicycle main road traffic thus yield motor vehicle travel times similar to the already presented demand scenarios with 60% main road bicycle traffic. New travel time estimates for demand scenarios with 80% main road bicycle traffic are therefore not made. Motor vehicle and bicycle left turn shares hardly impact motor vehicle travel times at the roundabout as well, see Figure 15 and Figure 17 in section 4.1 and Figure 45 in appendix I.1, therefore no new estimates are made for the 5% left turn shares.

The three variables that are assumed to not affect motor vehicle travel time each have two possible levels. This means that Table 19 is only one out of $2 \times 2 \times 2 = 8$ travel time tables for traffic patterns with 60% main road motor vehicle traffic. The ranking of the junctions in these other tables are however expected to be the same as those presented here. With 19 out of every 25 demand scenarios the roundabout scores the lowest motor vehicle travel time on 76% of the demand scenarios with 60% main road motor vehicle traffic.

Table 20 is also only one out of eight possible tables representing all traffic demand scenarios with 80% main road motor vehicle traffic. With 20 out of every 25 demand scenarios the roundabout scores the lowest motor vehicle travel time for 80% of the demand scenarios with 80% main road motor vehicle traffic.

The conclusion regarding motor vehicle travel time is that in general the roundabout yields the lowest motor vehicle travel times only at high traffic intensities does the SGFC-junction perform better. This conclusion is in line with the expectation that at high demand a signalised junction yields shorter travel times than a roundabout. For instance the Dutch "Recommendations for urban traffic infrastructure" (CROW, 2012) recommends to construct a junction between two distributor roads as a roundabout unless demand is so high that problems regarding capacity, delay, use of space or costs occur. This means that it is assumed that for high demands roundabouts yield less throughput and higher travel times than other junction types. This study shows that this tipping point lies around motor vehicle intensity levels of 2.500 mv/h or 3.000 mv/h depending on the motor vehicle main road traffic share and bicycle intensity.

Other studies have concluded that roundabouts can offer lower average travel times for motor vehicles even at high intensities as long as the traffic is relatively evenly distributed over the roundabout arms, for instance (SWOV, 2012), (Turner, 2011) and (Hydén & Várhelyi, 2000). This study supports these conclusions. The found tipping point from which the SGFC-junction scores lower travel times around motor vehicle intensities of 2.500 or 3.000 mv/hour lies very close to the theoretical roundabout capacity of 2.700 PCE/hour (CROW, 2008). Despite a big variation in the simulated traffic demand scenarios in this study, they all have a relatively even distribution of traffic over the arms. The traffic intensity at the two major arms is always equal and the intensity at one minor arm always equals the intensity at the other minor arm. For further studies it is recommended to test traffic demand scenarios that do not have this symmetry between junction arms.

The limited size of the simulation network that was used, has affected the performance outcomes of the roundabout, therefore the final conclusions are partly drawn on extrapolated data. For instance in Figure 9 the average motor vehicle travel time at the roundabout initially increases with an increase in motor vehicle intensity, but at the high intensity values the travel time value levels out. This is caused by the fact that at high motor vehicle demands the queues cover the entire junction approach. As a result it is then no longer possible for new vehicles to enter the network. If the network would have been larger, i.e. the approaches would have been longer, this may not have occurred. The queues would have grown longer resulting in longer average motor vehicle travel times. An important recommendation for future studies is therefore to not only check if all vehicles have been processed at the end of the simulation period but also ensure that all vehicles can be added to the network at the intended time during the complete simulation period.

5.2. Traffic safety

The SGFC-junction has lower conflict scores than the roundabout for each of the three types of conflicts for all traffic demand scenarios. However, several studies using accident data at existing junctions show that roundabouts yield a lower injury accident risk than a signalised junction (Minnen J. v., 1995), (Turner, 2011) and (Dijkstra, 2014). There are several factors that could explain this difference between the outcomes of the simulations and these earlier findings i.e. the set up of accident data studies, general limitations of micro simulations in traffic safety assessment and errors in model setup.

The first factor is that most of these accident data studies use only the broad definition 'signalised junctions'. Since at SGFC-junctions motor vehicle traffic and bicycle traffic are separated, it is likely that these are safer than an average signalised junction. This means that the safety benefit of a roundabout over an SGFC-junction may actually be smaller than the benefit of a roundabout over the complete group of signalised junctions.

The other two factors are a consequence of using a micro simulation model. The simulated road users in a micro simulation model strictly adhere to the traffic rules and are always completely focussed on their driving task. As a result traffic accidents never occur in the simulations making it necessary to use surrogate safety measures. In this study the amount of TTC conflicts is used as surrogate safety measure. The indicator amount of TTC conflicts itself has two main limitations in assessing traffic safety.

The strict rule compliance and focus may impact the relative score of the two junction types if rule braking occurs more frequently and / or when a traffic rule infraction has a greater safety risk at one junction type compared to the other. It is possible that driving through a red light has a higher accident risk than taking priority at a roundabout because road users usually approach a roundabout at a lower speed than a signalised junction. Furthermore road users at a signalised junction may focus on the traffic signal and therefore pay less attention to other road users compared to a roundabout where, because of the priority control, road users are forced to interact and thus pay attention to other road users. Therefore road users may detect rule violations of others sooner and thus lower the probability that a rule violation leads to an injury accident.

Any surrogate safety measure has some limitations. The most important limitations of the use of the number of TTC conflicts as surrogate measure are the uncertainty about the relation between the number of TTC conflicts and the amount of accidents, the fact that a conflict requires two vehicles involved and a lower accuracy at signalised junctions.

Because the exact relation between the number of TTC conflicts and the amount of accidents is unclear, it is possible that the ratio between the number of TTC conflicts and the number of accidents at a SGFC-junction differs from that ratio at a roundabout. In general drivers cross a signalised junction at a higher speed than a roundabout. Therefore it is possible that the TTC conflicts at a roundabout happen at lower speeds than the TTC conflicts at a SGFC-junction. Which would mean that the probability that a TTC conflict results in an injury accident is lower at a roundabout than at a SGFC-junction.

By definition a TTC conflict can only occur between two vehicles. Using the number of TTC conflicts as a safety indicator therefore means that single vehicle accidents are excluded from the comparison between the junction types. If the number of single vehicle accidents is higher at SGFC-junctions than at roundabouts, then this could partly explain why the simulation results indicate that the SGFC-junction is safer than the roundabout in contrast to previous studies. This argument is however not valid. An earlier study where accident statistics of junctions that were converted to roundabouts were analysed showed that the average number of single vehicle accidents actually doubled after the junction conversion (Schoon & Minnen, 1993).

A study comparing the number of crashes on real world junctions to the number of simulated conflicts for these junctions found that, regarding real world crashes the accident risk at signalised junctions is much higher than at priority controlled junctions whilst regarding the amount of simulated conflicts there is almost no difference between these junction types (Dijkstra, Marchesini, Bijleveld, Kars, Drolenga, & Maarseveen, 2010). Based on the number of simulated conflicts the amount of real world conflicts is underestimated. The same study also specified both the crashes and conflicts into three types: Frontal, Rear-end and Lateral. For priority controlled junctions the distribution of simulated conflicts over these three types followed the distribution of crashes over these types much closer than for the signalised junctions. The method of assessing traffic safety through the amount of simulated conflicts is more accurate for priority controlled junctions compared to signalised junctions. In this study the roundabout, which is a type of priority controlled junction, and the SGFC-junction, which is signalised, are compared. It is therefore likely that in this research the safety comparison between the roundabout and the SGFC-junction is skewed because of an under estimation of the number of crashes that would occur on the SGFC-junction.

As already mentioned in paragraph 4.2 the average number of motor vehicle only conflicts per vehicle turned out to be almost zero for the SGFC-junction for all traffic demand scenarios. Especially the unresponsiveness to motor vehicle intensity may point at an error in the SWOV TTC module or the processing of the module outcomes. Because the amount of conflicts does increase with an increase in motor vehicle intensity at the roundabout and because the amount of motor vehicle only conflicts at the SGFC-junction is not exactly zero at least some parts of the safety module seem to be working.

Although there are several factors that could help explain the differences between the simulation results in this study and accident data observations these factors are unlikely to explain the complete difference. Therefore the study outcome that the SGFC-junction is safer than the roundabout for all simulated traffic demand scenarios should be re-evaluated . It is recommended to conduct further research on how the SGFC-junction scores on accident risk based on real world observations. In addition more research is needed to clarify the relation between the number of TTC conflicts and actual accidents occurring at different junction design types.

The analysis of the change in differences between junction designs regarding safety performance shows that over the three variables there are three traffic demand scenario variables that reduce the attractiveness of the roundabout compared to the SGFC junction. These are:

- Motor vehicle main road traffic share for motor vehicle only conflicts
- Motor vehicle intensity for bicycle only conflicts and motor vehicle bicycle conflicts
- Bicycle left turn share for bicycle only conflicts and motor vehicle bicycle conflicts

However because the results regarding safety are unexpected these findings may not be very reliable. The outcome of the first two points likely depends on the forming of motor vehicle congestion. In itself an increase in motor vehicle intensity should increase the amount of conflicts because a greater number of motor vehicles means that the probability that a bicycle and a motor vehicle meet increases. As motor vehicle intensity increased the congestion did as well. At the roundabout this means that motor vehicle approach and cross the bicycle path at lower speeds reducing the probability that they come into conflict with a bicycle. This in turn also reduces the probability that two bicycles come into conflict because the probability that a bicycle has to react to a motor vehicle also is reduced. The presence of congestion may therefore be the 'hidden' third factor that can explain this illogical negative correlation between motor vehicle intensity and safety. Further research that explicitly records the presence of congestion is recommended.

5.3. Emissions

In chapter 4 it was shown that in many cases total emissions decrease when motor vehicle demand increases. This contradicts common sense. Another unrealistic outcome is that a decrease in the average motor vehicle speed leads to lower emissions per motor vehicle. Several studies have found that motor vehicles drive most efficient at speeds of around 70 km/h, see (Boer & Vermeulen, 2004), (Anable, Mitchell, & Layberry, 2006) and (Federal Highway Administration, 2006). Below this value a decrease in average speed leads to an increase in emissions per vehicle.

A lower average speed indicates that there is more congestion at the simulated junction which likely results in more acceleration and deceleration per vehicle. The increased amount of accelerations should lead to higher emissions per vehicle because it costs more energy than driving at a constant speed. Under these assumptions the emission performance of the junction types would be related to the travel time performances. The junction that offers the lowest average motor vehicle travel time is the junction with the lowest amount of congestion and therefore also the junction with the lowest emissions. A literature study into roundabouts applied in the U.S. supports this theory concluding that the improved traffic flow at roundabouts is also beneficial for air quality (Turner, 2011). Moreover a study about the effects of transforming priority controlled junctions and signalised junctions into roundabouts provides examples of this theory in both ways (Hydén & Várhelyi, 2000). The transformation from priority controlled junction to roundabout would slightly increase motor vehicle travel time and also resulted in an increase in emissions. Transformation from signalised junction to roundabout decreased travel time and emissions along with it.

The rate of travel time decrease is not equal to the decrease in emissions. A study using micro-simulation to study the consequences of replacing a specific signalised junction with a roundabout found that although the motor vehicle travel time would reduce with 20%, the emissions only decreased with 2% - 5% depending on the pollutant (Gastaldi, Meneguzzer, Rossi, Della Lucia, & Gecchele, 2014). Further research into the environmental performance of the two junction types is therefore recommended.

5.4. Impact of traffic demand scenario variables

This study has used six different variables to describe the used traffic demand scenarios. These variables were chosen because it was thought that they would have the biggest impact on the junction performances and therefore also influence the ranking of the junction types. Table 21 again shows the impact that the traffic demand scenario variables had on the relative performance of the roundabout compared to the SGFC-junction. For example the '--' in the top left cell of the table means that an increase in motor vehicle intensity leads to a stronger increase in motor vehicle travel time at the roundabout compared to the SGFC-junction. This means that compared to the SGFC-junction the roundabout loses some of its attractiveness.

Effect of an increase in the demand variable on attractiveness of the roundabout compared to the SGFC-junction								
Traffic pattern variable	**Performance indicator**							
	MV TT	**Bicycle TT**	**#MV only conflicts**	**#Bicycle only conflicts**	**#MV bicycle conflicts**	**TC emission**	**Nox emission**	**PM10 emission**
mv intensity	--	0	++	--	--	*-- **	*-- **	*-- **
bicycle intensity	--	+	0	++	++	*-- **	*-- **	*-- **
mv main road traffic share	--	+	--	++	++	*-- **	*-- **	*-- **
b main road traffic share	0	+	0	++	+	*0 **	*0 **	*0 **
mv left turn share	++	0	++	+	0	*++ **	*++ **	*++ **
b left turn share	0	0	0	--	--	*0 **	*0 **	*0 **

Table 21: Change in attractiveness of the roundabout caused by an increase in the traffic demand variable. The emission results, marked with an *, are estimated on the motor vehicle travel time performances not on the emission results.

By checking the rows of Table 21 it is possible to tell if a traffic demand scenario variable has a big impact, be it positive or negative, on the attractiveness of the roundabout compared to the SGFC-junction. The most influential variable is the motor vehicle main road traffic share which has a small influence on the junction attractiveness regarding bicycle travel time and a big influence for all other performance indicators. Motor vehicle intensity is the second biggest influencer. Apart from the bicycle travel time this variable has a strong impact on junction type ranking for all junction performance types. Bicycle intensity is the third most influencing variable followed by motor vehicle left turn share.

The bicycle left turn share only has a big impact on the relative attractiveness of the roundabout regarding two junction performance variables, both of these are traffic safety related. Because the bicycle left turn share hardly impacts any of the other junction performance criteria, future studies that are not looking into traffic safety can exclude this demand variable. The bicycle main road traffic share has a large impact on the number of bicycle only conflicts and a small impact on bicycle travel time and the number of motor vehicle bicycle conflicts. In further studies not regarding traffic safety this traffic demand variable could be excluded.

5.5. Research limitations

This chapter discusses the main limitations and uncertainties of the results and conclusions of this research. The junction performance depends on a large number of factors. The main limitations of this study fall into two categories. The first category of limitations is related to the great number of factors that can influence junction performance. Not every factor that (potentially) influences junction performance could be included in the comparison between the two junction types. The second category of limitations relates to the use of a simulation model. A simulation model is a simplified representation of reality and thus inherently has limitations to the applicability of its results. Paragraph 5.5.1 presents the main limitations of the first category. Paragraph 5.5.2 presents the main limitations of the second category.

5.5.1. *Limitations due to research methodology*

Not all factors that impact junction performance could be taken into account, because there are simply too many influencing factors. For instance pedestrians have not been included in this research meaning that the conclusions of this research are not applicable for locations where pedestrian traffic is not negligible.

Even though the junction designs have been tested for a large variety of traffic demand scenarios there are still a lot of demand scenarios for which they have not been tested and for which the real performances may differ from the research conclusions. The most obvious limitation of the tested traffic demand scenarios are that they are all symmetrical. Although the amount of traffic varies over the different traffic demand scenarios the one main road approach always handles the same amount of traffic as the other main road approach. The same is true for the minor road approaches. Furthermore within each traffic demand scenario all four arms have the same distribution of right turn, straight through and left turn traffic. In reality traffic demand scenarios will rarely be exactly symmetrical.

As already discussed in section 5.2 the choice for using a simulation model and the number of TTC-conflicts as an indicator to assess traffic safety has limitations. Traffic accidents cannot occur in simulations which means it is necessary to use a surrogate safety measure. Because the exact relation between surrogate safety measures and the number of accidents is unknown the safety results can only be used to compare the two designs to each other. In order to get more reliable safety results future studies could gather real world accident data in order to make a safety assessment of the SGFC-junction. In case it is necessary to use simulation models it would be wise to first investigate if other surrogate safety measures are more accurate at predicting traffic safety at signalised junctions compared to the number of TTC conflicts.

The fact that this research could not produce reliable results regarding the junctions environmental performance obviously is a big limitation. Other studies have been able to produce reliable emission results using an instantaneous emission models, such as (Hydén & Várhelyi, 2000), and even using the AIRE emission model, for example (Gastaldi, Meneguzzer, Rossi, Della Lucia, & Gecchele, 2014) and (O'Brien, Bell, Namdeo, Sinclair, & Scaife, 2012). These studies demonstrate that the research methodology used here can be a valid methodology.

5.5.2. *Accuracy of the simulations*

The conclusions of this research are based on simulation results. Although the simulations aim to exactly reproduce the real traffic situation certain inaccuracies exist. One important factor field where the simulations differ from reality is traffic safety. Because this research uses TTC as a measure of safety by definition single vehicle accidents are not included.

Small design variations can have a big influence on junction safety. For instance the angle at which two roads cross impacts how far drivers have to turn their heads in order to check if they have to give way. Altering this angle and thus the sight lines that road users have has a significant impact on traffic safety (European Road Safety Observatory, 2006). It is however difficult to incorporate these effects into model simulations.

The S-Paramics software does not by default include the simulation of bicycles and therefore has some limitations regarding representing the behaviour of bicyclists. One of these limitations regards the minimum gap in queue. The minimum gap drivers maintain between their vehicles when they are stationary is currently a variable that affects all vehicle types. In reality bicyclists will queue at a much smaller distance to each other than motorists. In this research the value for motorists has been used which means that cyclists queue with a greater distance between them than in reality. This results in a lower capacity for bicyclists because, except for the first bicycle in queue, bicyclists stop at distance further away from the stop line than reality. So when they get a green light it will take them more time to cross the stop line than in reality, resulting in lower capacity. To solve this problem it is recommended that another variable minimum gap in queue specifically for bicyclists is introduced on the model software.

6. References

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Appendices

A. Junction types under consideration

This section first describes some characteristics both junction types will have in common. These are mainly about the characteristics of the roads connecting to the junction itself. Below that, specific characteristics of the two junction types are described in their own paragraphs.

A.1. Common design characteristics

Within urban areas in the Netherlands four standardised speed limits are applied: 15 km/h, 30km/h, 50km/h and 70km/h. The speed limit of 15 km/h is applied at 'woonerven'. The function of 'woonerven' is more social than transport related, for instance children should be able to play on the street. For this reason they are only applied at roads with very low volumes of traffic. Because of the low traffic volumes and the low speed differences between road users traffic problems at junctions are minimal. Therefore this road type and speed limit will not be considered in this research.

The sustainable safe traffic system('duurzaam veilig') used for road design in the Netherlands defines two different functions of roads: enabling traffic flows and enabling access to and exchange with the adjoining properties. Traffic flows are defined as moving of vehicles in a more or less constant direction at a more or less constant and relatively high speed. Exchange is defined as: moving vehicles with varying directions at varying speeds. This includes departing and stopping/parking. The sustainable safe traffic system defines two road types within the urban area: access roads and distributor roads (SWOV, 2005).

The access roads are aimed at providing access to private (both residential and commercial) land plots and public land plots. These roads are intended to focus more on enabling an exchange of people and goods between the road network and the adjoining land uses than on enabling flowing of high traffic volumes. Because of this function the access road is only applied at locations with relatively low volumes of traffic and the speed limit is set at a low level; 30 km/h. The standard recommended junction form for a junction of two access roads is an uncontrolled junction. Because of the relatively low traffic volumes and the relatively low speed differences between road users problems at junctions are usually small. Therefore this road type and speed limit will not be looked at in the simulations.

The distributor road is aimed at enabling traffic flow at the road sections and at allowing access and exchange at its junctions. Because of the traffic flow function the speed limit is higher than on access roads, the standard value within the urban area is 50 km/h. The traffic volume is moderate to high. Because of the higher flows of motor vehicles and their higher speed bicycles should not mix with the motor traffic. At least bicycle lanes should be provided, but cycle tracks or rerouting bicycles over different roads is recommended. Junctions between two distributor roads should not be handled by uncontrolled junctions, the recommended design type is the roundabout. However the signalised junction is also frequently applied, for instance when there isn't enough space for a roundabout or to give road authorities more control over traffic flows.

Some distributor roads within the urban area have a 70 km/h speed limit instead of 50 km/h. The amount of 70km/h roads within the urban areas is low, it is not a recognised road type within the sustainable safe traffic system (SWOV, 2005). The roads with a speed limit of 70 km/h are placed in, and designed for, locations with high volumes of motorised traffic for instance as ring roads around city centres. Because of these high traffic flows at relatively high speeds it is recommended to keep bicycle traffic off these roads and use grade separated junctions. Therefore this road type will not be looked at in this research. To summarise: the junction types under consideration are situated at crossroads of distributor roads with a speed limit of 50 km/hour.

The recommended lane width for motorised traffic on a distributor road with a speed limit of 50 km/h is 3,25 meters (CROW, 2012). For a separate bicycle lane the recommended width is: 2,00 meters (CROW, 2012). The separation between the motor traffic lane and the bicycle lane should be at least 0,60 meters (CROW, 2012). These lane widths are used in the simulations.

Within the urban area 53% of junctions between two distributor roads have three arms and 45% have four arms (CROW, 2012). The research is limited to four arm junctions because these junctions have more conflict points than three armed junctions and are therefore more likely to have traffic problems. Junctions with more than four arms are not considered because they are rare, the make up only 2% of the junctions between two distributor roads within the urban area (CROW, 2012). Every road will have a single carriageway with one lane per direction. Whether the approaches to the junction have more lanes, for instance a dedicated left turn lane, is determined by the traffic demand scenario. The roads are connected perpendicularly in accordance with the design guidelines (CROW, 2012).

A.2. Signalised junctions

Signalised junctions can increase traffic safety because they separate conflicting traffic flows in time. Compared to priority junctions, signalised junctions reduce the amount of right angle collisions. However in some cases the installation of traffic signals will increase the number in rear-end collisions (Kennedy & Sexton, 2009). Each group of traffic lights that has green at the same time is called a signal group.

Separating conflicting traffic flows means that flows will have to be stopped during certain parts of the time but this does not have to have a negative impact on the junction capacity. In free flowing traffic the gaps between vehicles will on average be large, large enough to allow safe crossing. However at higher traffic intensities the average gap becomes too small to allow safe crossing. The traffic intensity at that moment is probably still far below capacity; the average gap between vehicles is still bigger than the minimum safe following gap, which is usually set at 2 seconds. By stopping a traffic flow temporarily a traffic light sums the excess gap times between vehicles to create one bigger gap in the traffic flow to allow conflicting flows to cross.

Another characteristic of signalised junctions is that they offer high controllability. By adjusting signal timings a road authority can control the traffic flow on the individual arms. By providing different signals and different signal phases for different modes of transport road authorities can actively distribute road capacity over the different modes. By providing more green time to bicycles road authorities can make junctions more bicycle friendly.

As mentioned previously this research is about junctions of roads with a speed limit of 50 km/h. At this kind of roads the Dutch sustainable safe traffic system recommends that bicycles have a bicycle path separated from motor traffic which can for instance be achieved with a kerb with buffer zone or by parked cars (SWOV, 2005). Because bicycles have their own path separated from motor traffic they will also have their own traffic lights. Separated traffic lights give road authorities the ability to control bicycle traffic independent from motor traffic.

The high controllability of traffic lights also enables road authorities to optimise traffic flows for different types of goals. A traffic light control program can be aimed at the highest throughput, possible for a specified direction, or at lowering emissions for instance by extending the green time when a lorry is approaching. This characteristic means that 'a signalised junction' is not a narrow and exact definition but can describe a broader range of designs and control logics and thus a range of junction performances. It is therefore necessary to specify a more exact definition of the type of traffic light control junction that is used in the comparison in this research. This is done in section A.2.2.

A.2.1. Junction with traffic lights with SGFC

Signalised junctions with SGFC were first implemented in the Netherlands in the 1980's. The main goal behind implementing this junction type is making signalised junctions attractive for cyclists by reducing waiting time and increasing traffic safety (Lieshout, 1996). The distinctive feature of an SGFC-junction is obviously the dedicated green phase for cyclists. During this phase all cyclists, so at all the junction arms, get green and can cross the junction in all three directions. All motor vehicle movements are stopped during this bicycle phase. At most of the junctions where SGFC is implemented cyclists are allowed to turn right on red. To even further decrease waiting time for cyclists the green phase for cyclists is realised two times per traffic light cycle at some locations.

The three most important advantages of the SGFC-junction are: short average waiting times for cyclists, cyclists can turn left diagonally across the junction in one go and there are no conflicts between cyclists and motorised traffic (Eggen, Salomons, & Zeegers, 2003). The SGFC-junction is the only signalised junction type that has demonstrated in practice that it can approach or meet the guidelines regarding average waiting time and the probability of passing without having to stop formulated for bicycle friendly signalised junctions in CROW publication 230 "Ontwerpwijzer fietsverkeer" (CROW, 2006), see chapter Bicycle friendly junctions.

Being able to turn left diagonally over the junction in one go increases comfort and reduces delay for cyclists compared with the two stage left turn which is common at other signalised junctions, see Figure 31. For a two stage left turn cyclists first have to cross the side road, the arrow denoted with 1. in the figure. After that they can cross the main road to make the left turn denoted by the arrow with number 2.

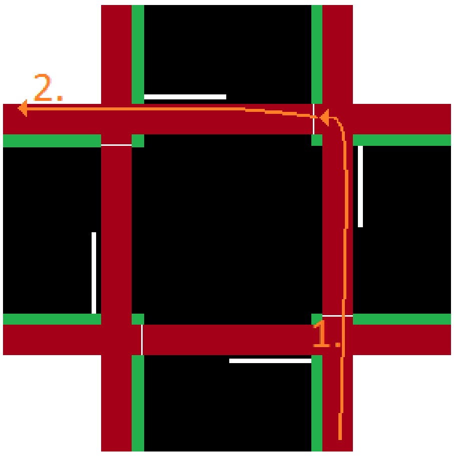

Figure 31: Two stage left turn at a signalised junction

Because the bicycle traffic is completely separated in time from motor traffic, safety for cyclists is very high. Because all bicycle traffic travels at the same time there still are conflicts between bicycles, but because the low speed and high manoeuvrability these conflicts seldom lead to accidents (Fietsberaad, 2003).

The two most important weaknesses of the SGFC-junctions are: the increased waiting time for motorised traffic and difficulties with integrating this junction design with pedestrian traffic (Eggen, Salomons, & Zeegers, 2003). Naturally when implementing a double green phase for cyclists the waiting time for motorised traffic will increase. This negative effect can be minimised by using a short green phase for cyclists. Short green phases are possible when the junction is compact because this leads to short clearance times for bicycles. Applying a double realisation of the bicycle green phase reduces the amount of cyclists that needs to cross the junction in that phase which helps in enabling shorter green times per bicycle phase.

Integrating pedestrians into the SGFC-junction is difficult. Giving pedestrians green at the same time as the cyclists leads to conflicts between cyclists and pedestrians. Giving pedestrians their own green phase, conflict free with other modes, leads to long cycle times and thus long waiting times for all traffic at the junction. The last option, handling pedestrians in potential conflict with turning vehicles has obvious traffic safety disadvantages, for example the possibility of blind spot accidents with lorries. However, as explained in paragraph 2.3 pedestrians will not be included in this research.

An often perceived disadvantage of this junction type is a high number of bicycle only conflicts. However based on the number of registered accidents there are no known problems with high bicycle accidents (Fietsberaad, 2003). One reason that the number of conflicts and accidents between bicycles is low is that the distance between the stop line and the conflict point is different for different for bicycles from different arms, see Figure 32 below. Distance D1 is a bigger than distance D2. Because all bicycle traffic lights turn green at the same time and bicycles will roughly accelerate at the same rate a bicycle leaving from point 2 will have already cleared the conflict point before a bicycle from point 1 reaches it. Therefore the first group of bicycles that was queued at the stop line will usually be able to cross the junction conflict free. Furthermore if there is a potential conflict between bicycles they usually have enough time to adjust because of their low speed.

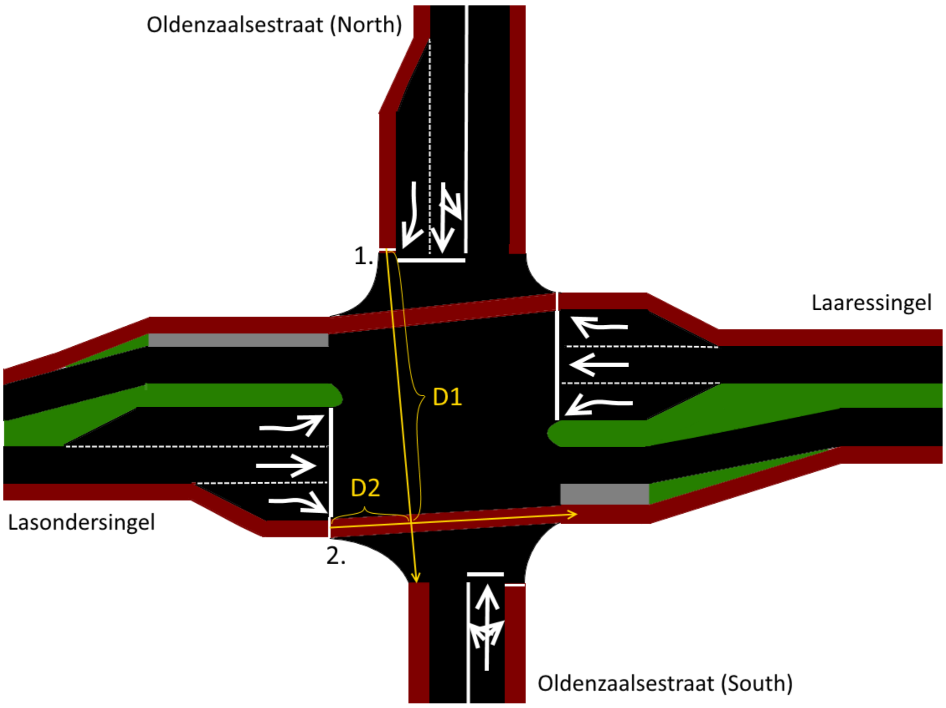

Figure 32: Different distance between the stop line and conflict point for bicyclists at different SGFC-junction approaches

The above listed advantages and disadvantages of the SGFC-junction lead to the following recommendations about where the SGFC-junction should be applied (Eggen, Salomons, & Zeegers, 2003):

-The junction needs to be relatively compact

If the junction is too big, for instance because of a large number of (turn)lanes for cars, the clearance time for the bicycles will become too long making the SGFC design inefficient.
-The junction should have a clear design
This recommendation applies especially to left turning cyclists. For them it should be clear where they need to go. Therefore it is recommended that the cycle path on the out flowing roads are close to the roadway or are constructed as on road cycle lanes.
-Preferably the motor traffic is handled with two or three different phases
If there are already a lot of phases for motorised traffic adding two phases for cyclists may increase the cycle time to unacceptable levels.
-The number of motor vehicles crossing the junction should not exceed 25.000 / day
This recommendation is derived from the first (compact junction) and third recommendation (two or three phases for motor traffic).
-The percentage of left turning cyclists should be substantial (>10%)
To truly utilise the advantage that cyclists can turn left in one go there should be a substantial amount of cyclists that want to make this turn.

A further recommendation from (Lieshout, 1996):
-On junction approaches there should be a physical separation between the bicycle lane(path) and car lane
By physically separating the motor traffic flow from the bicycle flow the risk that cyclists ride through red when the parallel motor traffic has green light is reduced.

A.2.2. Detailed junction design and traffic control logic

As mentioned earlier the term signalised junction can represent multiple design and light control logic variations. To determine in what situations the conclusions of this research are applicable and to enhance the reproducibility of this research a detailed definition of the SGFC-junction under consideration is given here. First the junction design is discussed than the traffic control logic.

The amount of lanes on the junction approach is not limited to one, it varies depending on the traffic demand scenario. The number of lanes present not only directly influences junction capacity it also specifies what types of control logic can be applied. For example if there is no dedicated left turn lane available it is not possible to include that left turning traffic as a separate stage in the light control logic.

Whether or not a dedicated left turn lane is necessary is based on CROW design recommendations. This recommendation is represented in Figure 33 below (CROW, 2012). The letters $I_{a,}$ on the x-axis, and $I_{o,}$ on the y-axis, represent the intensity of the trough traffic. Each of the lines represent the percentage of left turning traffic from the total flow of I_a, denoted by L. If the point in the graph described by the two through traffic intensities lies to the right/above the line with the appropriate percentage of left turning traffic than a dedicated left turn lane is recommended. When this guideline recommends a left turn lane for a certain traffic demand scenario then it is applied in the simulation. The same guideline is applied to determine if a dedicated right turn lane is implemented.

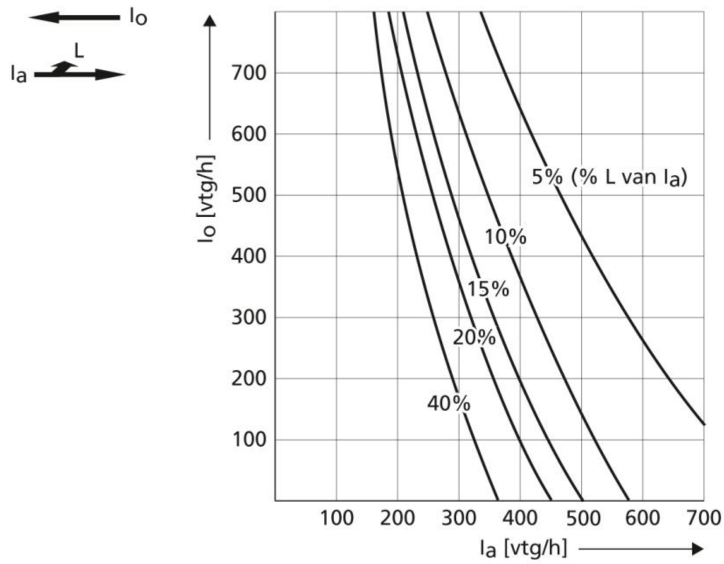

Figure 33: Design recommendation about left turn lanes. If the intersection point of the two through traffic flows is above/right of the line with the respective left turn percentage a dedicated line is needed.

Because the traffic demand scenarios in this study are symmetrical, as indicated in appendix H, I_o and I_a are always equal. The threshold value from which a dedicated turn lane is necessary can therefore be found at the intersection of the respective turn share line and an imaginary line at an 45 degree angle. In all traffic demand scenarios the right turn share is 10% which means that the threshold intensity for a dedicated right turn lane is 390 through going motor vehicles per hour. In case the left turn share is 5% the threshold intensity is 480 through going motor vehicles per hour. In case the left turning share is 15% the threshold intensity is through going 340 motor vehicles per hour. The turn lane configuration is calculated with these threshold values. The example below demonstrates these calculations for a traffic demand scenario with: a total motor vehicle intensity of 2.500 mv/h, 60% of motor vehicle traffic on the main road and 15% left turning traffic.

Together the two approaches that form the main road get to process $2.500 \times 60\% = 1.500$ motor vehicles. Each main road approach thus processes $1.500 \div 2 = 750$ motor vehicles. The amount of through going vehicles is $100\% - 10\% - 15\% = 75\%$ which results in a through going motor vehicle intensity of $750 \times 75\% = 562{,}5$ mv/h. This is higher than the threshold value of 390 mv/h required for a dedicated right turn lane so a right turn lane is provided. The value of 562,5 mv/h is higher than the threshold value of 340 mv/h so a left turn lane is provided as well.

Together the two approaches that form the side road get to process $2.500 \times 40\% = 1.000$ motor vehicles. Each main road approach thus processes $1.000 \div 2 = 500$ motor vehicles. The amount of through going vehicles is $100\% - 10\% - 15\% = 75\%$ which results in a through going motor vehicle intensity of $500 \times 75\% = 375$ mv/h. This is lower than the threshold value of 390 mv/h required for a dedicated right turn lane so a right turn lane is not provided. The value of 375 mv/h is higher than the threshold value of 340 mv/h so a left turn lane is provided.

Table 22 below gives an overview of the turn lane configurations used in each traffic demand scenario.

Overview of the used turn lane configuration over the different traffic demand scenarios					
Main road and left turn share combination	**Motor vehicle intensity**				
	1000	**1500**	**2000**	**2500**	**3000**
60% main road, 5% left turn	- - - -	- - - -	Left turn major Right turn major - -	Left turn major Right turn major - Right turn minor	Left turn major Right turn major Left turn minor Right turn minor
60% main road, 15% left turn	- - - -	- - - -	Left turn major Right turn major - -	Left turn major Right turn major Left turn minor -	Left turn major Right turn major Left turn minor Right turn minor
80% main road, 5% left turn	- - - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -
80% main road, 15 % left turn	- - - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -	Left turn major Right turn major - -

Table 22: Overview of the used turn lane configuration over the different traffic demand scenarios

The traffic light cycle will use the same pattern as junctions in city of Enschede. Information about this is taken from the camera observations. Using the programming interface in the S-Paramics software a control logic for the traffic signals is written. The exact signal timings produced by this logic will depend on the amount of traffic. For every different lane configuration a separate control logic is made. The control logic follows the principles listed below:

1) The bicycle green times is set between five and eight seconds, depending on the junction size
2) The length of the bicycle green phase is independent of the amount of bicycles present
3) The bicycle phase can be realised two times per traffic light cycle
4) the maximum total cycle time is set at 120 seconds just like in Enschede (Eggen, Salomons, & Zeegers, 2003).
5) Within the dedicated green time for cyclists and the maximum cycle length the throughput of motor vehicles is optimised.
6) The division of green time for motor vehicles over the main- and the side road will mirror the division used in the generation of the traffic demand scenario
7) If dedicated left turn lanes are present on the main road then the left turn will in principle get a separate stage.
8) If there are no left turning vehicles present on only one approach the green time of the opposing through traffic can be extended into the left turn stage.

A.3. Roundabouts

In the Netherlands a roundabout is officially defined as a junction that: handles traffic in circular motion, where the junction arms connect radially and where traffic on the junction has priority over traffic approaching the junction (CROW, 2008). The main reason for implementing roundabouts, regardless of the priority situation for cyclists, is an increased traffic safety compared to signalised junctions. In a meta study combining 28 international studies about the safety of roundabouts compared to signalised junctions (Elvik, 2003) found a reduction in injury accidents between 30% and 50%. This higher traffic safety is partly achieved because the number of conflict points on a roundabout is much lower than on a signalised junction. The conflicting traffic flow comes only from one side, although this does not always apply to cyclists and pedestrians, making the traffic situation clearer. Another safety benefit is caused by the lower speed of traffic. All traffic, even through traffic, has to make turns and therefore must slow down so any collision that does happen is at a relatively low speed reducing its consequences (SWOV, 2012).

Regarding motor vehicle capacity the roundabout scores comparable to the signalised junction when the traffic is divided relatively equally over the junction arms and arrives regularly. When traffic is however very imbalanced over the arms, or when traffic arrival patterns are irregular a signalised junction can handle traffic more efficiently and thus have a higher capacity than a roundabout (SWOV, 2012).

In an older study about small roundabouts without special bicycle provisions on the roundabouts, Hydén and Várhelyi (Hydén & Várhelyi, 2000) investigated the average time it took motor vehicles to approach and cross junctions. For fixed-time signalised intersections that were reconstructed into mini roundabouts the average time savings were 11 seconds per motor vehicle and 3,4 seconds per cyclist. The total amount of traffic at these junctions had not changed significantly after the reconstruction compared to before. The fact that the time spent was lower could be interpreted as roundabouts have a higher capacity than the time signalised junctions: the total traffic intensity remained constant whilst the time it took to handle the traffic decreased.

The previously mentioned study (Hydén & Várhelyi, 2000) also investigated the effect of changing junctions into roundabouts on emissions. They found that at the one junction with fixed time signals the introduction of a roundabout lead to a reduction of 29% in CO_2 emissions and a 21% reduction of NO_x emissions.

There is a big variation in roundabout designs apart from the different layouts of the bicycle infrastructure. An important variable is the amount of lanes on the roundabout; only one or two or more. Naturally a roundabout with more lanes has a higher capacity, especially when this is combined with double lanes on the approaches and exits. Two or more lanes on the roundabout and approaches increases the speed at which the roundabouts are taken and increases the number of conflict points. Therefore this type of roundabout is not as safe as single lane roundabouts. Therefore multi lane roundabouts are not recommended in urban areas (CROW, 1998). For that reason only single lane roundabouts are included in this research.

A special type of two lane roundabouts are turbo roundabouts. The motor traffic lanes at turbo roundabouts are constructed in a spiralling way. The motorists have to choose their lane, depending on where they have to exit the roundabout, before entering the roundabout. This means that there is no more weaving on the roundabout itself increasing car capacity and safety (CROW, 2008). This does also mean that the approaches to the roundabout need to have at least two lanes. This reduces safety for cyclists which now have to cross more lanes and thus face a larger conflict area. Furthermore vehicles on the inside approach lane may block the view for vehicles on the outside approach lane meaning drivers and bicyclists could overlook each other. Because of this lower safety for cyclists the CROW guidelines recommend that bicycle traffic should be handled grade separated (CROW, 2012). Because of this turbo roundabouts will not be included in this research.

For single lane roundabouts with single lane approaches and exits the rule of thumb maximum capacity is 25.000 PCE per day (CROW, 2008). This is the same maximum as the SGFC-junction so regarding the motor vehicle intensity the single lane roundabout and SGFC are applicable in the same locations, increasing the practical usability of a comparison between them.

The position of the cycle lane has a large influence on the traffic safety for cyclists. There are three main variants: an on road cycle lane with priority for cyclists, separate cycle path with priority for cyclists and a separate cycle path without priority for cyclists. Because the amount of accidents occurring with the first variant, with an on road cycle lane, are much higher than for the designs with a separate cycle path this design is no longer recommended (Minnen J. v., Rotondes en voorrangsregelingen, 1995). Although due to lack of space there still is a significant amount of roundabouts that have bicycle paths close to motor traffic lanes, or sometimes even on road bicycle lanes.

Several Dutch studies investigated the effect of cyclists' priority on separate cycle paths at roundabouts on traffic safety. All of them found that roundabouts without priority for cyclists are safer. For roundabouts within the urban area with a separate bicycle path (Minnen J. v., 1998) found that on locations without bicycle priority on average 2,02 accidents happen per year whilst for locations with bicycle priority this is 3,53 per year. A study by (Weijermars W. , 2001) looked specifically at accidents with injuries and or casualties. Therefore this study found lower accident numbers but the difference between roundabouts without priority and with priority remains intact: on average 0,83 accidents per roundabout per year for locations with bicycle priority compared to 0,29 accidents per year for locations without. A study by (Gerts, 2002) used the same 36 roundabouts without cyclist priority as Weijermars but compared them with 22 different roundabouts with cyclist priority. Again this study only looked at injury and fatal accidents. This study found that on average 0,59 accidents per year happen on roundabouts with priority compared to 0,28 average annual accidents on locations without priority.

A.3.1. Roundabout with priority for cyclists

Under the current recommendations roundabouts within the urban area should have priority for cyclists (CROW, 2008), even though it is known that this is less safe for cyclists. This recommendation is therefore not always followed, some provinces never give bicycles priority at roundabouts. It is estimated that around 60% of roundabouts within the urban area has priority for cyclists (CROW, 2008). One reason to implement bicycle priority anyway is that it was expected that the safety deficit could be diminished following certain design standards about the diameter and distance between the cycle path and the roundabout. Another reason was the idea that by giving cyclists priority cycling would become more convenient and would therefore get a higher modal share.

If motor traffic may have to yield for cyclists when entering or exiting the roundabout, the amount of cyclists will have an effect on the motor vehicle capacity of roundabouts. For their bachelor thesis (Dragtstra & Elferink, 2010) studied the magnitude of this effect. They limited their research to single lane roundabouts with segregated bicycle paths where cyclists can only cross in one direction. They concluded, based on field measurements extended with micro simulations, that the effect of cyclists on the waiting time for motor vehicles approaching the roundabout is relatively small: on average 2,4 seconds increase per 120 crossing cyclists per hour.

The extra time loss that motor traffic experiences will lead to an increase in emissions compared to roundabouts where motor traffic has priority over cyclists. However there is no research that specifically isolates the effect of the priority rules on emissions.

B. Indicators used for performance evaluation

In the chapter research framework it is already explained that junction performance is evaluated in three different quantities: traffic flow, traffic safety and emissions. This chapter describes with what units these quantities are measured.

B.1. Junction capacity

For this first junction performance criterion the goal is to evaluate how well the junctions can handle traffic. Obviously this ability to handle traffic is related to the maximum junction capacity. It is however not sufficient to just determine the maximum junction capacity because this may not be directly related to how efficient the junction handles traffic at lower intensities. Suitable ways to indicate how well the junction handles traffic over the whole range of traffic demand scenarios are: measuring the delay that occurs or the length of the queues that occur.

The traffic handling capability of a junction can be expressed through the total amount of time spent in the network summed over all traffic that (wants to) pass through the junction. So if for example ten vehicles pass through the junction during the simulation period and their average travel time is 80 seconds then the total time lost is 10*80 = 800 seconds. The less efficient the junction handles traffic the more time it will take a vehicle to cross the junction, and thus the higher the total time spent. Because it will always cost time to cross the junction not all the time spent is time 'lost' due to congestion. However because both junctions are placed in an exact copy of the same network where the distance between the networks entry and exits point are the same, the difference between the time spent in the network is directly related to the differences in traffic handling capability of the junction design. There are several other related delay indicators that are commonly used such as: average delay per vehicle or average speed in the network.

A limitation of each of these measures is that they cannot distinguish what traffic state caused the delays. Stop and go traffic may lead to the same average time spent on the network as moving at a slow but constant speed. It is however possible that the road capacity is influenced by the state of the traffic flow on it. Once traffic is in a congested state the capacity of the road is usually lower. This is known as the capacity drop (Yuan, Knoop, Leclercq, & Hoogendoorn, 2014). However scientific research is focussed on capacity drops on highways and not on single lane urban roads where the effects may have a different magnitude.

The other indicator to represent how well the junction handles traffic is the length of the occurring queues. When the junction handles traffic very efficiently vehicles are able to quickly pass the junction and therefore not pile up in long queues. The complexity of using queue length as an indicator is in the definition of a queue. When there is a row of stationary vehicles than it is obvious that there is a queue, but what about a row of vehicles moving at only 5 km/h on a road with a speed limit of 50 km/h or a row of vehicles moving at 20 km/h? In order to define a queue it is therefore necessary to define a speed threshold below which vehicles count as queued. Another threshold that can be used to define whether a vehicle is queued is the distance to the preceding vehicle. If this distance is small than the following vehicle is impeded by the leading vehicle and thus effectively has fallen in a queued state.

By applying the queue length indicator it is possible to distinguish stop and go traffic (the queue) from slow but constant moving traffic given that it travels faster than the threshold speed for defining a queue. A big disadvantage of using queue length is that this method does not capture the full effect of the junction on the traffic flow. When vehicles have to slow down because of the junction but keep travelling faster than the queue speed threshold this is not registered even though they are hindered by the junction.

For measuring the junction traffic flow the indicator total time spend is used because it can measure the full effect of the junction design on traffic flow. Although this indicator does not directly distinguish the state of the traffic flow, the effects of this state on road capacity are represented in the simulation and thus indirectly included in the measurements.

The two junction designs are intended to provide short waiting times for cyclists, which likely results in longer waiting times for motor vehicles. In order to analyse these effects the time spent is specified for motor vehicles and bicycles specifically. The fewer time is lost, the more efficient a junction is at handling the traffic flows and the better it thus scores regarding throughput.

B.2. Traffic Safety

Because traffic simulation software is not capable of simulating traffic accidents, alternative indicators for traffic safety have to be used (Aarts, 2011). Generally speaking there are three other types of road safety assessments (Dijkstra, En Route to Safer Roads, 2011). The first type is based on exposure, the amount of traffic, controlled for different road type characteristics like speed limit, types of road users and / or junction type. The second type is based on the number of conflicts that occur between road users. The third type is an based on expert judgement where a road safety auditor assesses road safety of a new design by using his / her experience (Dijkstra, En Route to Safer Roads, 2011).

Because this research is about comparing two different junction designs in great detail the first method of determining road safety is not the most suitable because it uses more macroscopic traffic indicators. The third method is not feasible either because of the same reason. The second, conflict based, method however fits well with this research. Because the interactions between different road users is key to scoring junction performance, detailed information about the junction layout and the behaviour of individual road users will have to be gathered. This same data can be used to deduce conflict based safety measures. Because the interactions between different road users are modelled the effects of the traffic state on the junction approaches is included. Stop-and-go traffic will increase the probability of an collision compared to traffic flow with a constant speed because the accelerations and decelerations lead to speed differences between vehicles.

B.2.1. Conflict based safety indicators

This type of safety indicator bases traffic safety on the number of conflicts that occur; the higher the amount of conflicts that occur the higher the accident risk and thus the lower traffic safety is. Quantifying the exact relation between the amount of conflicts and accident occurrence and / or accident severance is still the topic of ongoing research (Dijkstra, En Route to Safer Roads, 2011), and will not be part of this research. By comparing the amount of conflicts between different junction designs it is possible to obtain the relative traffic safety of one design compared to the other.

Several subtypes of conflict based safety indicators can be distinguished based on how a conflict is defined. Well known examples are the Time To Collision (TTC), Deceleration Rate (DR), Encroachment Time (ET) and Post Encroachment Time (PET).

The most used conflict based indicator is the Time To Collision (TTC). This indicates how long it would take, usually expressed in seconds, before two vehicles would crash into each other assuming that they don't adjust their current speeds and trajectories (Archer, 2005). If the minimum value for this measure during one potential conflict event between two vehicles gets below a certain threshold it would be difficult or even impossible for a human driver to react to this conflicting situation meaning that an accident would likely have happened in that situation in real life. By counting the number of times the TTC between any two vehicles is below the threshold value it is possible to count the number of conflicts and thus give an indication of traffic safety. The advantage of the TTC indicator is that it can be used for both conflicts between vehicles on the same road as well as conflicts between different traffic streams that occur at junctions.

A disadvantage of using the amount of TTC conflicts as a safety indicator is that the relation between the amount of conflicts and the amount of accidents is not clear (Archer, 2005). However several studies have demonstrated that there is a correlation between the number of simulated TTC conflicts and real world accidents (Saleem, Bhagwant, Shalaby, & Ariza, 2014), (Dijkstra, 2011) and (Astarita, Giofré, Guido, & Vitale, 2012). Because the research goal is to compare two junction types to each other and not to predict the exact number of accidents at a junction it is not a problem that the link between TTC conflicts and accidents is fully understood.

An important limitation of conflict based indicators is that, because it requires a conflict between two road users, single vehicle accidents are not included. However from 2002 through 2006 about 46% of the total registered accidents with motor vehicles were single vehicle accidents (Kuiken, Bolle, & Nägele, 2008). An older study specifically looked into single bicycle accidents and found that about 60% of the total number of bicycle accidents are single bicycle accidents (Schoon & Blokpoel, Frequentie en oorzaken van enkelvoudige fietsongevallen, 2000). It is not known whether this reported share of single bicycle accidents applies for accidents at junctions as well. At this moment simulation software is not capable to generate information about single vehicle accidents.

B.3. Environmental impact

The performance of a junction regarding environmental impact can be divided into three categories. The first indicator is the emission of Total Carbon (TC). The average concentration of TC in the atmosphere has an impact on global warming. Global warming contributes to global climate change. Because both the cause, average TC concentration, and the effects, climate change, are of global nature the specific location of the TC emission is not important. Although it is not possible to exactly calculate how much the average TC concentration will increase from the emissions at the junction it is certain that the higher the amount of emissions the worse the contribution to climate change. So the junction with the lowest amount of TC emissions scores best at this subject.

Second are several indicators that can partly explain the effect of traffic on local air quality. European emission standards for vehicles limit the emissions of: CO, NO_x, hydrocarbons and particulates (European Union, 2013). The relation between the emission of these substances by vehicles and local air quality is not direct because the local circumstances for example wind speed, precipitation and temperature influence how these substances disperse. The combination of emissions and dispersion determines the local air quality. Correctly simulating these dispersion effects is another field of study and for the simplicity only the amount of emissions is used as indicator for local air quality.

Third group is the emission of sound. As with the emissions effecting local air quality the amount of noise (dB) is not always directly correlated with the amount nuisance people experience. Local circumstances, for instance the distance between the road and housing play a role as well. In addition characteristics of the noise itself play a role as well. The noise frequency plays a role, the lower the noise the further it carries. In this research noise emission will however not be included.

The traffic state has a big impact on the amount of emissions produced. Because of the many accelerations and deceleration stop and go traffic will lead to more traffic emissions than traffic driving at a constant speed. Because the estimations of the amount of emissions produced are based on micro-simulation outputs the traffic state is explicitly included in the calculations.

C. Camera observations

The camera observations are conducted to quantify the performance of an existing junction which serves as a reference in the model calibration process. The video footage is used to gather data about several aspects of the traffic flow: vehicle types, traffic intensities, traffic arrival distributions, queue lengths in meters and queue lengths in number of vehicles. This chapter discusses the preparation for the camera observations.

Keypoint Consultancy B.V. provides the camera's for the observations. These camera's are mounted on lamp posts. This higher vantage point gives a good overview of the junction area and reduces the risk that the view is obstructed by passing lorries or vans. The cameras take a picture every second.

C.1. Observation locations

The observations will take place at both one roundabout and at one SGFC-junction. The selection of a suitable observation location is based on four different criteria: the distance to other controlled junctions, the presence of public transport lines, the junction geometry and practical considerations.

This research is aimed at evaluating junction designs on their local effects. Therefore individual independent junctions are simulated. The junction independence means that the signal timings are independent of other junctions and that the traffic arrives not in platoons. By observing junctions that are relatively far away from other controlled junctions this junction independence can be approached.

Buses that pass a signalised junction have the ability to influence the signal timings in their favour. This could for instance mean that the buses' green phase is extended or that it gets green in favour of another direction. The priority for the bus will have a negative impact on the junction capacity for other directions and modes which may lead to more queued vehicles. Because public transport is not included in the simulation model the realised traffic flow disturbances of the bus will cause differences between the observed and the simulated traffic flows. The model calibration, which is about matching the simulated traffic conditions with the observed traffic conditions, will in that case be less accurate. Therefore a junction without public transport is preferred.

Regarding junction geometry two aspects are considered. The first is the compactness of the junction. A compact junction is preferred because that makes it easier to observe all the traffic movements with a small amount of cameras. The second aspect regarding geometry considers the junction arms. Preferable the arms are connected at an angle close to 90°. This will make it easier to calculate the junction clearance times for simulating the junction signal timings.

The final consideration is purely practical: the location proximity. Because the cameras from Keypoint are stored in Enschede it would be easier to observe location in Enschede. Therefore only SGFC locations in Enschede are considered for observation.

In total eleven different SGFC-junctions are located in the municipality of Enschede (G. Spaan, personal communication, 16 December 2014). Of these eleven locations seven are not suitable because their signal timings are coupled with other junctions. From the remaining four junctions only one does not facilitate public transport, this is the junction of the Oldenzaalsestraat with the Lasondersingel/Laaressingel. The only drawback of this location is that the junction is relatively big; both the Singels have three lanes on the junction approach (and one on the exit) and a wide median. Overall the junction of the Oldenzaalsestraat with the Singels is the most suitable observation location.

For the roundabout it is no longer necessary to conduct new observations. Instead video footage previously made for a different research is used . These observations have been conducted at a roundabout with bicycle priority at the crossing of the Europaweg, Bruchterweg and Admiraal Helfrichstraat in Hardenberg, the Netherlands. In total about eight hours of video footage has been made.

C.1.1. Camera setup

At the SGFC-junction location several cameras are used, either aimed at the intersection area or at one of the junction approaches. The intersection area is monitored by two cameras. These cameras are set up at different corners of the junction area. In principle only the images of one camera are used to gather data. The other camera acts as a backup, mainly for the conflict observations, in case the view from the first camera is obstructed or at such an angle that makes analysing the images difficult.

Although the observation locations are chosen in such a way that traffic is likely to arrive relatively evenly spread out this cannot be guaranteed. The traffic arrival distribution can have a big influence on the performance of a junction. For a good model calibration it is therefore important that the arrival distribution is known. To gain information about the arrival distribution cameras are aimed at the junction approaches.

The traffic arrival distribution can be different on the different approaches to the junction therefore from an information standpoint ideally all four of the approaches are observed. However this will lead to a multiplication in the amount of images that have to be analysed making this unfeasible. As a compromise a total of five cameras is used. Apart from the two cameras aimed at the intersection area the two approaches of the Singel and the Northern approach of the Oldenzaalsestraat are monitored by one camera each. The Southern approach of the Oldenzaalsestraat is not filmed because it has only one approach lane. It is therefore not possible that the outlfow of one direction is hindered because a turn lane is blocked. A schematic overview of the used camera setup is displayed in Figure 34 below.

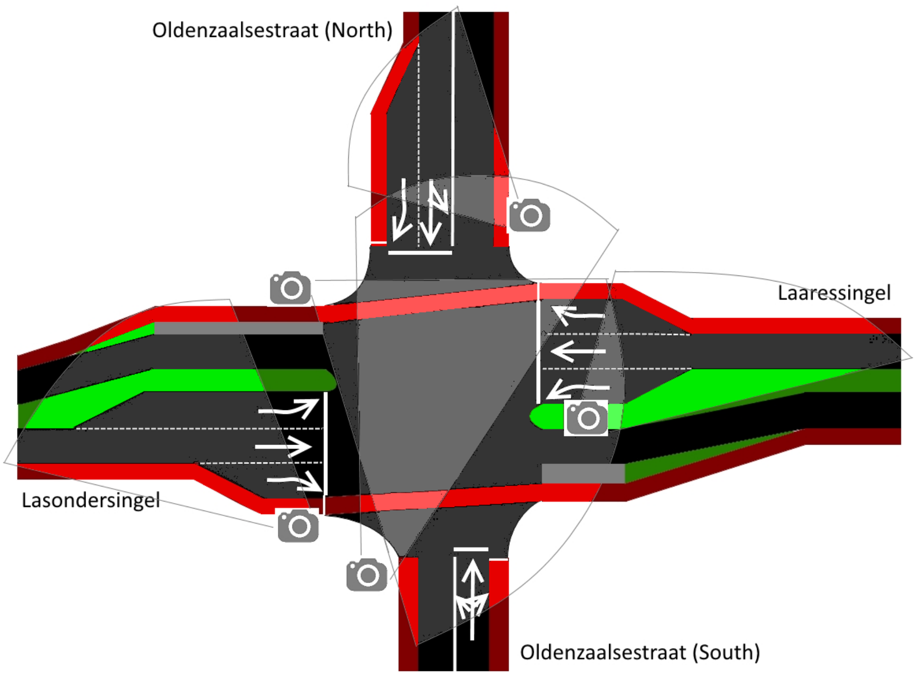

Figure 34: Schematic overview of the positioning of the cameras at the signalised junction

The previously conducted observations at the roundabout in Hardenberg were done with one camera. This camera pictures the traffic on the roundabout itself near the Europaweg and traffic on the Europaweg. A schematic overview of the camera placement is given in Figure 35 below.

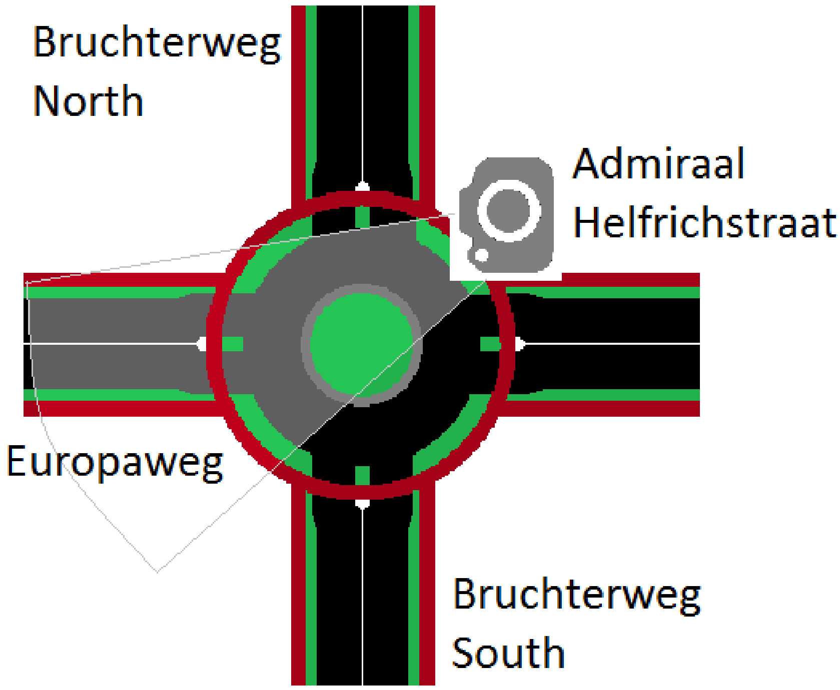

Figure 35: Schematic overview of the positioning of the camera at the roundabout

Because the observations were done with only one camera it will not be possible to determine the traffic arrival distribution on the other three of the four roundabout approaches. The observations from Hardenberg are however still useful. Firstly because all the traffic interactions at the Europaweg, both at the approach and the exit, are observed. It is assumed that traffic behaves in the same way at the other roundabout approaches. So if the simulated traffic behaves similar to the observed traffic at the Europaweg the simulation will realistically present traffic flows at a roundabout.

Secondly the headway distribution of traffic on the circulating lane is a compound of the arrival distributions on the three other approaching arms. Although it is not possible to deduce the arrival patterns at the individual arms based on this information, because it is unknown from which approach the vehicles came, the effects of different arrival patterns are included in the observed traffic flows.

C.1.2. Observation times

To enhance the quality of the calibration it is desirable to have observations for different traffic demand scenarios. Therefore each of the two locations is observed during a peak period and during an off peak period. Each observation period will last two hours. This should be enough time to get an overview of the regular behaviour of traffic at the junction.

In order to get representative observations the weather conditions cannot be extreme. Observations made during heavy precipitation or fog will therefore not be suitable. The reduced visibility leads to decreased driving speeds and an increased minimal acceptable gap, which both contributes to a lower junction capacity. Furthermore the rain will reduce the quality of the video images making the data processing more difficult and less accurate.

Because the camera observations are made during winter there is a risk at subzero temperatures. This in itself does not render the observations unsuitable. Provided that the road surface is cleared of ice it is assumed that drivers show their regular behaviour.

The final factor that may influence traffic behaviour at junctions is darkness. In dark conditions drivers will have more difficulty estimating distances, and thus gap sizes. To compensate for this greater uncertainty they generally will accept bigger gaps. For instance a study investigating left turn movements from a minor road onto a major road at unlit junctions concluded that the minimal accepted gap was 3,8 seconds during daylight and 4,7 seconds during dark conditions (Shokri, Ismail, Hafezi, Ganji, & Rahmat, 2012).

However for this research it is expected that (the lack of) daylight on driver behaviour is less important. Mostly because at both locations the junction area and the approaching roads are equipped with public lighting.

Furthermore specifically for the junction of the Oldenzaalsestraat with the Singel it helps that the number of uncontrolled conflicts for motor vehicles is relatively low. Only the left turning traffic from both approaches of the Oldenzaalsestraat depend on gap acceptance to navigate through the junction. Meaning that any potential influence of darkness on gap acceptance is relatively small. For bicycles gap acceptance plays a much more important role. Since all bicycle directions get green at the same time there are multiple potential conflicts between opposing bicycle flows. However of their lower speeds it is expected that darkness has only limited effect on their gap acceptance behaviour.

D. Extraction of model input data

Paragrapn 3.2.2 has explained what information about the traffic pattern needs to be extracted from the camera observations for the model calibration: traffic intensities per turn direction and vehicle type, vehicle arrival distribution and minimum gap between vehicles in a queued state, gap acceptance and signal timing principles . This chapter explains how this data can be extracted from the video footage.

The traffic intensities are extracted from the images of the camera aimed at the intersection area by simply counting the number of vehicles that pass per turning direction. These traffic intensities are summed at five minute intervals, because this is the smallest interval at which traffic intensities can be defined in S-Paramics (SIAS Limited, 2011).

The mean headway and headway distribution are observed using the cameras aimed at the junction approaches. First step in this process is to define a line on the approach. Ideally this line is placed at a location far enough upstream of the junction where the traffic flow is not disturbed yet by the presence of a queue. Based on the images the time at which a vehicle crosses the line is recorded, accurately to the seconds. With this information it is possible to calculate the time interval between vehicles. By registering the time at which a vehicle crosses a certain line on the approach.

To determine the headway distribution the frequency of predefined headway intervals between vehicles is counted. Because the frame rate of the video observations at the SGFC-junction is one frame per second the interval length is set at one second. So the intervals (bins) will be: between 0,5 and 1,5 s, 1,5 - 2,5 s, 2,5 - 3,5 s, etc. until the final bin from 8,5 s to 9,5 s. This means a total of nine intervals which corresponds with the number of sliders with which the aggression distribution can be defined in S-Paramics. Gaps greater than 9,5 seconds are not included. At such great gaps drivers can make fully autonomous decisions without having to take the vehicle in front of them into account. Therefore these long headways are not really a consequence of driver characteristics and thus they are rightfully not taken into account in determining the headway distribution. By counting how many measured gaps fall in a certain gap length interval the headway distribution can be quantified and visualised.

Apart from logging the traffic arrival distribution the cameras pointed at the junction approaches can be used to observe the distance between queued vehicles. Based on the road markings visible in the camera images an overlay of gridlines can be constructed. The gridlines are spaced at 2 meters apart. With help of this grid overlay the distance between queued vehicles can be observed.

The observed minimum gap in queue likely is different for bicycles compared to motor vehicles. A problem with the S-Paramics simulation software is that it is not possible to set a different global minimum gap in queue value for each mode separately. Therefore a value in between the measurements for each model is used.

D.1. Extraction of model output/calibration output

The first variable that is measured for the model calibration is the queue length in number of vehicles. Given that the traffic intensities and the arrival distributions used in the simulation are the same as in the observed situation the queue length is an important indicator to assess if the simulated junction has the same capacity as the observed junction. If the simulated queues are shorter, than junction capacity is too high. This could be solved by increasing the minimal acceptable gap time in the simulation.

The queue length in vehicles is deduced by counting the number of vehicles in the queue. In Paramics a vehicle is marked as being in a queue if it meets at least one of two criteria: the vehicles speed is below a threshold, by default 7,2 km/h or the distance to the vehicle in front drops below a threshold, by default 10 meters. For the analysis of the camera images the same rules are used. To assess the speed of a vehicle the same grid overlay used for determining the space between stationary vehicles is used. The speed threshold of 7,2 km/h corresponds to a speed of 2 m/s. So if a vehicle crosses more than one gridline, which are spaced 2 meters apart, between two images it is not in a queued state.

D.2. Extraction of simulation model limitations data

As explained in paragraph F.5 the camera observations are also used to gather information about the occurrence of several aspects that are not well represented in the simulation model. These are the red light violations, priority behaviour of cyclists and motor vehicles using the bicycle lane.

The observed red light violations are divided into three different categories. These are:

- Red light violations by motor vehicles
- Red light violations by cyclists or light mopeds when the parallel motor vehicle direction has green
- Other red light violations by cyclists or light mopeds

The relative amount of red light violations amongst motor vehicles is expected to be lower than the relative amount amongst bicyclists and light mopeds. The relative amount of red light violations is, both for motor vehicles and for bicycles and mopeds, strongly influenced by local factors. On average 27% of the cyclists at a junction will run through a red light (Meel, 2013) compared to a share between 1.6% and 8.2% for motor vehicles (Oei, Catshoek, Bos, & Varkevisser, 1997). Therefore violations by motor vehicles are counted separately from the bicycle and light moped violations. The bicycle and light moped violations are split into two categories. When the SGFC-junctions were first implemented one of the fears was that large amounts of cyclists would ignore the bicycle signal and cross the junction in accordance with the traffic signal for the parallel motor traffic because at most other junctions the bicycle signal would share its green phase with that of the parallel motor traffic. Therefore the occurrence of this type of red light violations is counted separately from the other red light violations by bicyclists and light mopeds.

For the analysis of the camera observations three different types of priority behaviour of bicyclists have been defined:

- **No specific priority behaviour**: cyclists can pass the junction without having to adjust their desired speed or trajectory to avoid other bicyclists
- **Priority for the cyclist from the right:** the bicyclist that comes from the right gets priority corresponding with the Dutch traffic law for uncontrolled junctions dictating that "drivers approaching from the right get priority" (RVV 1990). Bicyclists may behave this way because like at uncontrolled junctions all directions have equal importance because they all have a green traffic light.
- **Priority for the cyclist on the main road:** often signalised junctions have a set of priority markings as well to organise priority when the traffic lights are turned off. Bicyclists may follow these markings because the traffic lights do not govern priority between different directions.

At the observed SGFC-junction in Enschede motor vehicles sometimes drive on the bicycle path. This has a negative impact on the traffic safety at the junction because it increases the probability that a bicycle and a motor vehicle come into conflict. There is however a positive effect as well: right turning motorists can still access the right turn lane even when there is a long queue on the straight through lane which increases the junction capacity. To be able to assess both this positive and negative effect three different circumstances for a motor vehicle driving on the bicycle lane have been defined:

- **Right turning motor vehicle, right turn lane accessible:** in this case there is no positive effect on junction capacity because the normal entrance to the right turn lane was available. The negative impact on traffic safety however still remains. Motorists enter the bicycle lane out of convenience or because of a steering error.
- **Right turning motor vehicle, right turn lane blocked:** in this case there is a positive effect on junction capacity because the right turning vehicle does not add to the queue on the straight through lane. The negative impact on traffic safety is however still present. Motorists enter the bicycle lane to reduce their travel time.
- **Straight through driving motor vehicles:** in this case there is no positive effect on traffic flow because straight through going motorists cannot use the bicycle path to avoid a queue. The negative impact on traffic safety is however still present. Motorists enter the bicycle lane out of convenience or because of a steering error.

A vehicle is registered as 'driving on the bicycle lane' when at least half of the vehicle's width is above the bicycle lane. Left turning traffic is unlikely to drive on the bicycle path because these vehicles are moving to the outside of the central lane in anticipation of merging into the left turn lane.

D.3. Analysing images from the roundabout

The analysis of the camera observations at the roundabout results in the roundabout data spreadsheet. Per observed 50 minute interval one roundabout data spreadsheet is generated. These spreadsheets contain different types of information. Using Matlab scripts the information is aggregated into four datasets per analysed interval:

- Traffic intensity per vehicle type per direction per five minute interval
- Gap acceptance
- Queue lengths in number of motor vehicles per lane per five minute interval
- Gap distribution of inflowing vehicles

Paragraph D.3.1 describes the layout of the roundabout data spreadsheet. The next paragraph, D.3.2, illustrates the principles that each of the four Matlab scripts use when processing the roundabout data spreadsheet.

D.3.1. Roundabout data spreadsheet

Each row in the roundabout data spreadsheet contains information about one vehicle. Columns one and two are used to identify the vehicle with a unique number and identify the vehicle type. Columns three through seventeen are used to follow the road user through the junction. When a road user passes a certain point on the network the corresponding cell is filled with the frame ID number. Because the frame rate of the video observations is 25 fps dividing the frame ID number by 25 results in a time stamp in seconds. Out of all possible points of the network each road user only passes some of them. The cells of the points not passed remain empty. Columns eighteen through twenty are used to record the occurrence of an apparent conflict, with which vehicle the conflict occurs and if the vehicle leaving the roundabout was using its indicator respectively. If the road user does not adhere to the priority rules column 21 is filled with a '1' otherwise it remains empty. Column 22 is filled with a '1' if a bicycle is blocked by a motor vehicle standing on the bicycle lane. Column 23 indicates if the roundabout lane is blocked before the roundabout entry, meaning that motor vehicles entering the roundabout do not have to pay attention to motor vehicles on the circulating lane because they are stationary. Column 24 indicates that motor vehicles cannot enter the circulating lane because stationary motor vehicles are blocking the entry. Finally column 25 is filled with a '1' in case a bicycle drives on the motor vehicle circulating lane.

1	2	3	4	5	6	7	8	9	x
Vehicle ID	Vehicle type	Roundabout approach inflow	Queue start	Queue end	Front enter circulating lane	Rear enter circulating lane	Roundabout lane inflow front	roundabout lane inflow rear	x x x
[#]	1 to 8	[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	x

10	11	12	13	14	15	16	17	18	x
Exit roundabout front	Exit roundabout rear	Roundabout lane exit conflict front in	Roundabout lane exit conflict out	Roundabou t lane entry conflict in	Roundabou t lane entry conflict out	Roundabou t bike lane entry conflict in	Roundabout bike lane entry conflict out	Apparant conflict	x x x
[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	[FrameID]	'1' if true	x

19	20	21	22	23	24	25
Conflict partner ID	Use of indicator	Right of way violation	Roundabout bike lane blocked	Circulating lane blocked before approach	Circulating lane blocked after approach	Cyclist on the road
[#]	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true

Figure 36: Layout of the roundabout observation data spreadsheet

The numbers used to represent vehicle types are displayed in Table 23 below. The same vehicle type coding is used as well for all other data spreadsheets .

Distinguished vehicle types	
Description	**Coded number**
Car	1
Van	2
Lorry	3
Lorry with trailer	4
Motorbike	5
Bicycle	6
Light moped	7
Moped	8

Table 23: The number codes used to describe the observed vehicle type

D.3.2.Matlab scripts applied to roundabout data spreadsheet

This paragraph describes the four Matlab™ scripts applied to the roundabout observation data spreadsheet. These scripts are used to determine: : the traffic intensities per direction, the accepted and rejected gaps, the queue lengths on the approach and the headway distribution.

The first Matlab™ script is used to aggregate the observed roundabout data into traffic intensities per five minute interval per direction per vehicle type. First the script identifies the vehicle type, written in column two. Then the script checks if either column three or eight are filled to determine the origin of the road user. The entry frame ID is also used to determine in which five minute interval the vehicle should be counted. If the current road user entered the video image on the circle lane the script checks at what location this road user left the image. This is done by checking if either columns ten and eleven or columns fourteen and fifteen contain a frame ID number. Now that the vehicle type, time interval of inflow and route of the road user have been determined the script adds the road user to the correct cell in the intensity output matrix.

Matlab script gapacceptanceround.m
The second script presents two matrices: one containing all rejected and one containing all accepted gaps. The script first records during which times motor vehicles cannot enter the roundabout because a bicycle or motor vehicle on the roundabout is blocking the way. A dummy variable with time stamps in the first column and status updates in the second column is created for this. For each frame ID number in column fourteen a timestamp and a '1' (the status update) are posted on a new row of the dummy variable. For every frame ID number in column fifteen a timestamp and a '-1' are written on a new row of the dummy variable. After all observation data is processed the dummy variable is processed so that the timestamps are chronologically ordered. Now the dummy variable is processed from top to bottom. The second column each time gets a new value based on the value in the column above added or subtracted with one depending on the original content of this cell. Not the dummy matrix shows if a gap was available, a '0' indicates that no vehicle was blocking entry to the roundabout, or not, a value of '1'or higher means the entry is blocked. The second part of the script determines if these gaps were either accepted or rejected by motor vehicles on the approach.

The third script consists of two parts. The first part creates an output matrix with the observed queue lengths per five minute interval. The second part presents the shortest and longest queue per five minute interval out of all recorded queue lengths. The first step of part one of the script is to check for a frame number ID in column four. If there is a number than that motor vehicle was queued. The frame number ID is recorded in the first column of a dummy variable and a '1' is noted in the second column of the same row. One row below that the frame ID number found in column five is written down with a '-1' in the cell next to it. This way any change in queue length is recorded together with a timestamp on an individual row of the dummy variable. After all vehicles in the roundabout observation data spreadsheet are processed the dummy variable is sorted based on the frame ID number in the first column. Now the script processes the dummy variable from top to bottom and writes a timestamp, based on the frame ID number, and the queue length, by adding the '1' and '-1' values respectively, in a new queue matrix. After this the second part of the script picks the longest and shortest queue per five minute interval.

The final script determines the gap distribution of inflowing vehicles. The script is divided into two parts: the first part is about recording the gaps that occurred, the second part is about determining the distribution of these recorded gaps. The gaps between motor vehicles and those between bicycles are recorded in separate matrices. The first step of the script is therefore to check the vehicle type recorded in column five to determine in which matrix the gap should be recorded. Then the script checks for a value in column three or nine to be recorded in a dummy variable as the entry time of the leading vehicle. The script than continues to check the rows below in the datasheet until another vehicle of the same vehicle type as the leading vehicle, either motor vehicle or bicycle path user, is found. If the lead vehicle had a frame number in column three the script keeps checking column three for a following vehicle. If the leading vehicle had a value in column nine the script records the frame ID number in column eight. If both a leading and a following vehicles frame ID number are known the script proceeds to calculate the difference between them in frame numbers, calculates this into seconds and record this as a gap. After recording the gap the script empties the dummy timestamp values and continues one row below the row of the leading vehicle in the junction approach data spreadsheet.

After all gaps are recorded with the first part of the script the second part processes this gap data to determine the gap or headway distribution. In the script a fixed range of headway bins is defined. For each recorded gap the script records in which bin it belongs by checking if it is greater than the bin lower bound and smaller than the upper bound. When the correct bin is found the bin counter is increased with '1'. The final output is a matrix with different gap size bins over the columns.

D.4. Analysing images from crossing area of SGFC-junction

The analysis of the images from the two cameras recording the SGFC-junction crossing area results in the SGFC-junction crossing area data spreadsheet. Each observed hour results in a separate junction crossing area spreadsheet. These spreadsheets contain different types of information. Using different Matlab scripts the information is aggregated into four different datasets per analysed hour:

- Traffic intensity per vehicle type per direction per five minute interval
- Gap acceptance
- Priority behaviour of bicyclists and light moped riders
- Red light violations per vehicle type per direction

Paragraph D.4.1 describes the layout of the junction crossing area spreadsheet. The next paragraph, 6.1.1, illustrates the principles that each of the four Matlab scripts use when processing the junction crossing area data spreadsheet.

D.4.1. Junction crossing area data spreadsheet

Each row in the junction crossing area data spreadsheet contains information about one vehicle. The first three columns contain a timestamp, containing the hour, minute and second, when the vehicle passed the junction. The fourth column contains a unique vehicle ID-number. The fifth column contains a number that represents the vehicle type. These vehicle type numbers are the same as for the roundabout observations, see Table 23 in the previous paragraph.

The following twelve columns represent each of the possible (turn) directions a vehicle can take crossing the junction. The applicable column is filled with a '1' the other eleven remain empty. The eighteenth and nineteenth column are used to register the red light violations. The eighteenth column is filled with a '1' in case of any red light violation. In case a bicyclist violates the red light while the parallel motor vehicle traffic light was green a '1' is also written in the nineteenth column. The twentieth and twenty-first column are used to register TTC conflicts. The twentieth column is filled with a '1' in case of a conflict and the twenty-first column is filled with the unique vehicle ID of the other vehicle this vehicle came into conflict with. The last two columns are used for determining the priority behaviour of bicyclists and light mopeds in their green phase. In case the rider on the Singel gets priority a '1' is written in column twenty-two. If the rider from the right gets priority a '1' is written in column twenty-three. When riders did not clearly have to organise priority both columns will remain empty. An overview of the final layout of the table is given in Figure 37 below.

1	2	3	4	5	6	7	8	9	10	11	12	x
Hour	Minute	Second	Vehicle ID	Vehicle type	Laares Right	Laares Straight	Laares Left	Old. South Right	Old. South Straight	Old. South Left	Lasonder Right	x x
[Hour]	[Minute]	[S]	[#]	1 to 8	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	x

13	14	15	16	17	18	19	20	21	22	23
Lasonder Straight	Lasonder Left	Old. North Right	Old. North Straight	Old. North Left	Red light violation	Parallel car green	Conflict	Conflict partner ID	Priority for Singel	Priority for right
'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	'1' if true	[#]	'1' if true	'1' if true

Figure 37: The layout of the junction crossing area data spreadsheet containing information gathered from the cameras aimed at the junction crossing area

6.1.1. Matlab scripts applied to junction crossing area data

Four different Matlab™ scripts are applied to the junction crossing area data spreadsheet. These scripts are used to determine: the traffic intensities per direction, the accepted and rejected gaps, the priority behaviour of bicycles and the amount of red light violations. This paragraph describes the general working of each of these scripts.

The first Matlab™ script is used to aggregate the observed junction crossing area data into traffic intensities per five minute interval per direction per vehicle type. The first block of this script uses the time stamp in the first three columns of the database to check in which five minute interval this vehicle has to be added. The seconds block uses the vehicle type number in the fifth column to identify the vehicle type. The third block of script checks in which of the twelve columns corresponding with the twelve directions in which the junction can be crossed a '1' is written. The last step of the script is to add the observed vehicle to the previously determined interval, vehicle type and direction.

The second script gathers data about the gap acceptance behaviour. This data comes from a separate spreadsheet. Only motor vehicles on the Oldenzaalsestraat that want to make a left turn onto the Singels are observed for the gap acceptance. The data sheet contains per waiting vehicle the lengths of the gaps that occurred in the opposing motor vehicle flow and if the vehicle accepted this gap or not. The matlab script sorts this data into two matrices: one containing all rejected and one containing all accepted gaps.

The third script is used to determine the priority behaviour of bicycles. For each row in the junction crossing area data spreadsheet the script first checks the vehicle type number in column five. If this column contains a '6' or a '7' this row is about a bicycle user and the script starts to process it. The script checks for a '1' in column 22 or column 23 and increases the counter for the occurrence of priority for the Singel or priority for coming from the right respectively. If both columns 22 and 23 are empty it increases the counter for no priority behaviour needed is increased.

A fourth matlabscript is used to aggregate red light violations data. The script generates two matrices: one containing red light violations by motor vehicles the other regarding bicycles. The script checks if column 18 contains a '1' to indicate a red light violation. If a red light violation is found the script checks the vehicle type number in column five and based on this number determines if it is about a motor vehicle or a bicycle. The next step is to check which direction the road user was crossing the junction by checking which of the columns six through seventeen contains a '1'. If the red light violation was committed by a bicycle the script continues to record if the violation occurred when the parallel motor vehicle light has green. In that case column nineteen contains a '1' . The later part of the script writes the recorded number of each violation type in the respective output matrix and in the right matrix cell depending on the direction of travel of the road user. Finally the script calculates the percentage of red light violations of the total traffic volume for that direction.

D.5. Analysing images from approaches of SGFC-junction

Information from the cameras observing the junction approaches is stored in a different type of spreadsheet. The junction approach spreadsheet contains different types of information:

- Queue lengths in number of motor vehicles per lane per five minute interval
- Gap distribution of inflowing vehicles
- Number of bike lane invasions by motor vehicles

Paragraph D.5.1 describes the layout of the junction approach data spreadsheet. Paragraph D.5.2 illustrates the principles that each of the three Matlab scripts use when processing the junction approach data spreadsheet. Because there are three different approaches under observation there are three different junction approach data spreadsheets.

D.5.1. Junction approach data spreadsheet

Each vehicle will take up one row in the junction approach data spreadsheet. The first five columns are filled in the same way as the junction crossing area spreadsheet: a timestamp (hour, minute and second), unique vehicle ID number and vehicle type number. Columns six, seven and eight are used to determine the queue length. Each column represents one turn lane, so for the Oldenzaalsestraat North two are used and for the Singels three columns are used. In case a vehicle is queued a '1' is placed in the corresponding column. If the vehicle passes the junction without getting queued a '0' is placed in the corresponding column. This '0' serves as a marker to separate the different queues from each other. In case that no vehicles pass the junction unhindered between two different queues a dummy row with a timestamp and a '0' in the corresponding turn lane column, but without vehicle ID and vehicle type number is typed. Column nine records the queues in the bicycle lane in the same way as the motor vehicle queues.

Column ten is used to record if a motor vehicle drives on the bicycle lane. The column is filled with a '1' if this happens and remains empty when a motor vehicle does not enter the bicycle lane. Column eleven contains a '1' if the entrance to the right turn lane is blocked and remains empty otherwise. Column twelve records if the left turn lane is blocked in the same way as column eleven does this for the right turn lane. Finally column thirteen records if the vehicle experiences oversaturated conditions. If the vehicle under consideration has to wait for more than one traffic light cycle time a '1' is written otherwise the column remains empty.

1	2	3	4	5	6	7	8	9	x
Hour	Minute	Second	Vehicle ID	Vehicle type	Right turn lane queue	Through lane queue	Left turn lane queue	Bicycle lane queue	x x
[Hour]	[Minute]	[S]	[#]	1 to 8	'1' if queued '0' if not	'1' if queued '0' if not	'1' if queued '0' if not	'1' if queued '0' if not	x

10	11	12	13
Vehicle on bicycle lane	Right turn lane blocked	Left turn lane blocked	Saturated conditions
'1' if true	'1' if true	'1' if true	'1' if true

Figure 38: The layout of the junction approach data spreadsheet containing information gathered from the cameras aimed at the junction approaches

When traffic intensities are high it is possible that turning vehicles are unable to enter their desired turn lane because it is blocked by the queue on the straight through lane. In that case the queued vehicle has to be added to the queue of the through lane and not to the queue of the turn lane. For this purpose two other columns are used which each represent one of the turning lanes. If the turn lane is blocked the corresponding column is filled with a '1' otherwise it is left empty.

When direct entry to the right turning lane is blocked by a queue it is also possible that a vehicle drives on the bicycle lane to bypass the straight through queue and enter the right turn lane. For these cases another column is used. This column is filled with a '1' in case the vehicle drives on the bicycle path. In this case the vehicle has to be added to the right turn lane queue and not to the straight through queue even though the normal entry to the right turn lane is blocked.

D.5.2. Matlab scripts applied to junction approach data

This paragraph describes the three Matlab™ scripts that are used to process the junction approach data spreadsheets. Because the data spreadsheet for the Oldenzaalsestraat differs slightly from the other two data spreadsheets the scripts applied to this spreadsheets also differ slightly because different column numbers are used. Because the working principles are the same as the scripts applied to the other two approaches the scripts for the Oldenzaalsestraat North are not presented separately.

The first script consists of two parts. The first part creates an output matrix with the observed queue lengths per five minute interval. The second part presents the shortest and longest queue per five minute interval out of all recorded queue lengths. The first step of part one of the script is to check the time stamp in the first three columns of the observation data and determine in which five minute interval this queue belongs. The second step is to look for a '1' in the columns six through nine. If a '1' is found the relevant row counter is increased. If a zero is found the current value of the queue length counter is written to the output matrix and then the queue length counter is reset.

The second script determines the gap distribution of inflowing vehicles. The script is divided into two parts: the first part is about recording the gaps that occurred, the second part is about determining the distribution of these recorded gaps. The gaps between motor vehicles and those between bicycles are recorded in separate matrices. The first step of the script is therefore to check the vehicle type recorded in column five to determine in which matrix the gap should be recorded. The timestamp in the first three columns of this row is recorded in a dummy variable as the entry time of the leading vehicle. The script than continues to check the rows below in the datasheet until another vehicle of the same vehicle type as the leading vehicle, either motor vehicle or bicycle path user, is found. The time stamp of this vehicle is recorded in a dummy variable as the entry time of a following vehicle. If both a leading and a following timestamp are known the script proceeds to calculate the difference between them and record this as a gap. After recording the gap the script empties the dummy timestamp values and continues one row below the row of the leading vehicle in the junction approach data spreadsheet.

After all gaps are recorded with the first part of the script the second part processes this gap data to determine the gap or headway distribution. In the script a fixed range of headway bins is defined. For each recorded gap the script records in which bin it belongs by checking if it is greater than the bin lower bound and smaller than the upper bound. When the correct bin is found the bin counter is increased with '1'. The final output is a matrix with different gap size bins over the columns.

The last script records the number of bicycle lane invasions by motor vehicles. The first step is to look for a '1' in column ten to locate a bike path invasion. If a '1'is found the script checks for a '1' in either columns six, seven or eight to determine the travel direction of the motor vehicle. For the final categorization of the violation the script checks for a '1' in column eleven to check if the right turn lane was blocked during the violation.

E. Results of the camera observations

This chapter presents the findings from the camera observations. This chapter is split into two parts: the first part presents the observation results from the roundabout and the second part the results from the SGFC-junction. Both these sections follow a similar layout: first the traffic intensities are presented, second the number of TTC-conflicts, third the arrival distribution of traffic and finally the gap acceptance behaviour is presented. For the SGFC junction some more information is presented at the end of that section about: motor vehicles on the bicycle lane, red light compliance and the priority behaviour of cyclists during their green phase.

E.1. Results of the roundabout observations

This section presents the results of the analyses of the camera observations conducted at the roundabout at the crossing of the Europaweg, Admiraal Helfrichstraat and Bruchterweg in Hardenberg. The observations have been made on the 8th of February and the 9th of February in 2011.

E.1.1. Intensities per direction per vehicle type

Table 24 below shows the observed traffic intensities at the roundabout during a peak hour. The intensities are specified per five minute interval, vehicle type and route of travel. The route of travel is indicated in the different columns. The first two distinguish where a vehicle enters video images either on the approach upstream of the roundabout (far arm) or on the circulatory roundabout lane. The last two columns record where traffic left the video image either before crossing in front of the approach under observation (exit circular lane) or after (conflict flow on circular lane). The sum of the first two columns should equal the sum of the last two columns since every vehicle that enters the video frame also leaves it at some point. The other observed intensity data can be obtained via the author.

The data from Table 24 is used in three different ways. First the total vehicle intensities over all time periods is used to construct an Origin Destination (OD) matrix. Two OD matrices are made: one for motor vehicles and on for bicycles. Second the total numbers of different motor vehicle types are used to determine the distribution of different motor vehicle types within the total motor vehicles.

Interval start [minute]	Interval end [minute]	Vehicle type indicator	Intensities [# vehicles]			
			Far arm	entry on circular lane	Exit circular lane	conflict flow on circular lane
0	5	1	11	12	9	3
0	5	2	1	1	1	0
0	5	3	1	1	0	1
0	5	4	0	1	0	1
0	5	5	0	0	0	0
0	5	6	4	3	2	1
0	5	7	0	1	0	1
0	5	8	0	1	0	1
5	10	1	10	19	13	6
5	10	2	2	2	2	0
5	10	3	1	3	3	0
5	10	4	0	0	0	0
5	10	5	0	0	0	0
5	10	6	6	11	1	10
5	10	7	0	0	0	0
5	10	8	0	0	0	0
10	15	1	21	14	8	6
10	15	2	3	3	2	1
10	15	3	1	1	1	0
10	15	4	1	1	1	0
10	15	5	0	0	0	0
10	15	6	13	11	2	9
10	15	7	0	0	0	0
10	15	8	0	1	0	1
15	20	1	29	16	8	8
15	20	2	3	3	2	1
15	20	3	0	1	1	0
15	20	4	1	0	0	0
15	20	5	0	0	0	0
15	20	6	11	17	1	16
15	20	7	0	1	0	1
15	20	8	0	1	1	0
20	25	1	37	24	15	9
20	25	2	1	4	4	0
20	25	3	0	0	0	0
20	25	4	0	0	0	0
20	25	5	0	0	0	0
20	25	6	18	25	4	21
20	25	7	0	0	0	0
20	25	8	0	1	0	1
25	30	1	25	38	27	11
25	30	2	2	4	2	2
25	30	3	0	0	0	0
25	30	4	0	0	0	0
25	30	5	0	0	0	0
25	30	6	24	17	4	13
25	30	7	0	0	0	0
25	30	8	0	0	0	0
30	35	1	32	20	14	6
30	35	2	2	2	2	0
30	35	3	3	0	0	0
30	35	4	0	0	0	0
30	35	5	0	0	0	0
30	35	6	7	11	1	10
30	35	7	0	0	0	0
30	35	8	1	1	0	1

Table 24: Observed traffic intensities at the roundabout during a peak hour specified per five minute interval, vehicle type and flow direction

E.1.2. Mean headway and headway distribution of traffic

The average gap is calculated based only on the gaps that are smaller than 9.5 seconds. In Paramics the mean gap parameter is intended to indicate at what time distance vehicles follow each other. At very large gaps drivers no longer show vehicle following behaviour, instead they drive independently. It is therefore not right to take very long gaps into account when determining the mean headway. The cut off value of 9,5 seconds is chosen because it complies with the setup of nine gap frequency bins setup for determining the gap distribution, see below.

Table 25 below shows the mean headway as observed during the rush hour at the roundabout. The mean headway is calculated for motor vehicles and bicycles separately in column two and four respectively. For both modes the gaps are measured for two traffic flows: vehicles entering at the observed roundabout approach and vehicles entering the video on the roundabout circulatory lane. The mean headway is the weighted average of the observed headways at the two points.

Mean headway excluding gaps > 9.5 for roundabout during rush hour				
	Motor vehicles		Bicycles	
Location	Average gap	# observations	Average gap	# observations
Approach	3.19	185	3.06	33
Circulatory lane	3.55	137	3.07	35
weighted average	**3.35**		**3.07**	

Table 25: Mean headways excluding gaps bigger than 9.5 s for the roundabout during rush hour

In S-Paramics the mean headway is a global parameter meaning that it is applied to all vehicle types. It is however still possible to use a different mean headway for bicycles because each link has an adjustable headway factor. This factor indicates how the mean headway on that specific link relates to the global mean headway. For the roundabout calibration model the headway factor for each bicycle link is set at: $3{,}07 \div 3{,}35 = 0{,}92$.

The headway distribution is quantified by counting how many of the measured gaps fall in each of the nine gap length intervals, see further explanation in appendix D. The results of this measurement are shown in Table 26 below.

Distribution of motor vehicle traffic over different gap sizes for roundabout rush hour excluding gaps > 9.5 s										
Arms	**Relative importance of bins**									**Totals**
Bin lower bound [s]	**0.5**	**1.5**	**2.5**	**3.5**	**4.5**	**5.5**	**6.5**	**7.5**	**8.5**	
Bin higher bound [s]	**1.5**	**2.5**	**3.5**	**4.5**	**5.5**	**6.5**	**7.5**	**8.5**	**9.5**	
Approach	17%	36%	19%	6%	5%	5%	5%	4%	3%	100%
Circulating lane	24%	19%	14%	10%	13%	5%	5%	5%	4%	100%
Averages	**21%**	**28%**	**16%**	**8%**	**9%**	**5%**	**5%**	**4%**	**4%**	

Table 26: Headway distribution of motor vehicles for the roundabout rush hour

The percentages in the bottom row of Table 26 are direct input for the nine sliders with which the aggression distribution is determined in S-Paramics. The aggression distribution affects the headway distribution, desire speed distribution and gap acceptance distribution. In S-Paramics only one aggression distribution can be set which affects all vehicle types. The motor vehicle headway distribution is used for this and therefore the bicycle gap distribution is not determined.

E.1.3. Gap acceptance

The critical gap can be determined based on the recorded accepted and rejected gaps. In their paper Kay et al identify three methods for determining the critical gap based on curves representing the frequency of rejection and acceptance of gaps (Kay, Ahuja, Cheng, & Vuren, 2006). These are:

- Raff's critical gap
- An equal overlapping area gap
- 50th Percentile of rejected gap curve

They conclude that for congested roundabouts the 50th percentile of the rejected gap curve provides the best approximation for the gap acceptance. Under congested conditions the amount of rejected gaps will be greater than the number of accepted gaps, because each vehicle can only have one accepted gap, but likely multiple rejected gaps. However the roundabout observations conducted in this research have been conducted under low traffic conditions, the number of accepted gaps is about three times bigger than the number of rejected gaps. Therefore the 50th percentile of rejected gap curve is not suitable for this situation and Raff's critical gap method is used.

Raff's critical gap method uses cumulative share of accept or reject curves. On the x-axis the gap size in seconds is displayed. On the y-axis the share of gaps, from 0% to 100% is displayed. The reject curve shows what share of gaps would be rejected given the gap size. At a small gap size on the x-axis all rejected gaps are bigger, resulting in an Y value of 100%. As the gap size value on the x-axis increases the share of rejected gaps that is greater reduces. This represents the behaviour of the driver population: as gap size increases a lower share of the drivers will reject that gap. The cumulative share of accepted gaps shows an opposite curve. As the gap size increases the share of drivers that accepts this gap increases. Raff's critical gap method states that the gap size value found at the intersection of the rejects and accepts curve is the critical gap. Figure 39 below shows the accepted and rejected curve for one of the roundabout observations.

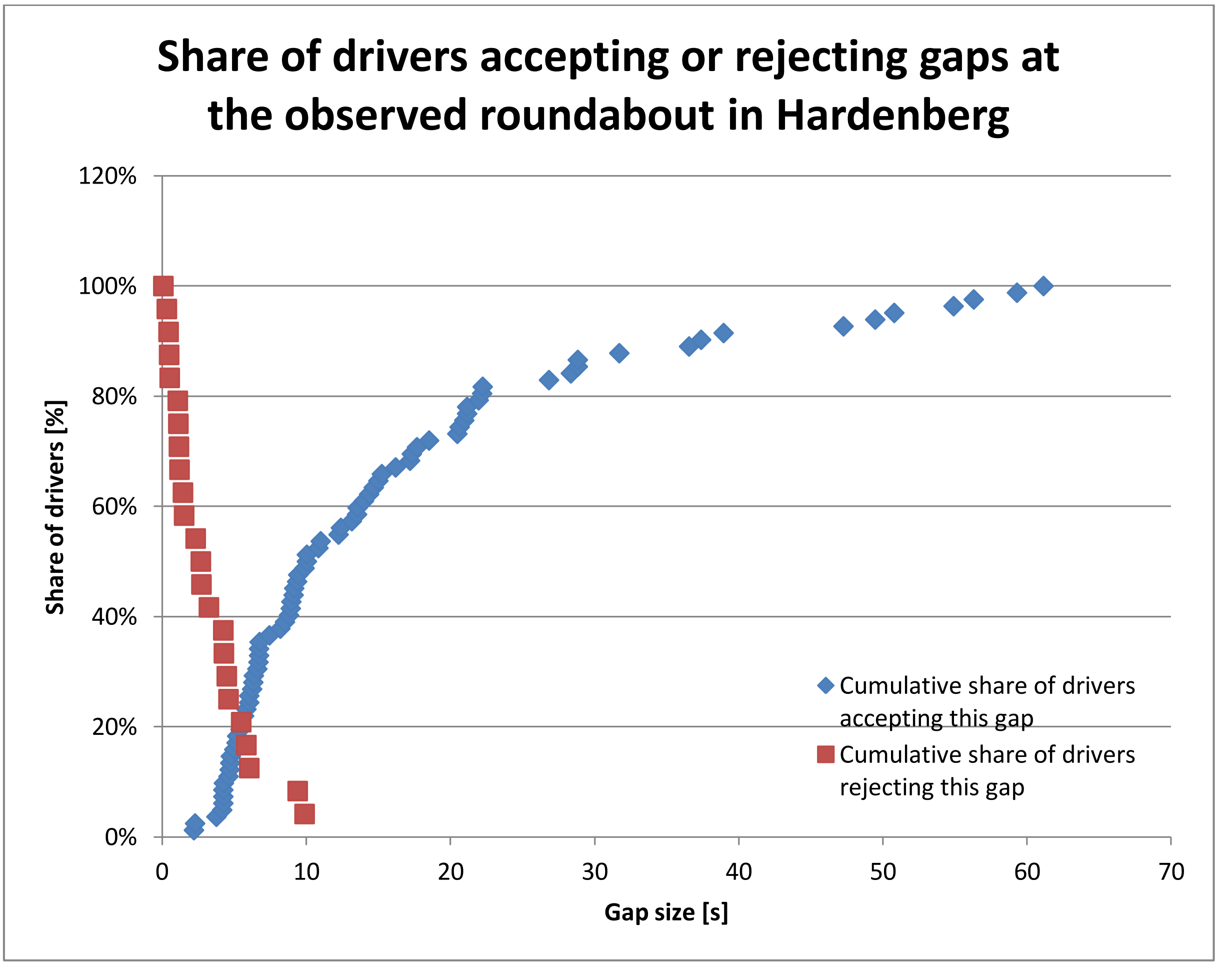

Figure 39: Share of drivers accepting or rejecting gaps at the observed roundabout in Hardenberg

Figure 39 above is based on the accepted and rejected gaps found during a fifty minute interval observation interval at the roundabout in Hardenberg. The intersection point of the accepted and rejected curve lies at a gap size of 5.43 seconds. This value is the default for the simulation model calibration.

E.1.4. Observed queue lengths

Table 27 below shows the observed minimum and maximum queue lengths per five minute interval. These values are used to compare the simulated queue lengths against in the calibration process.

Observed queue lengths at the roundabout in peak hour				
Interval #	Interval start minute	Interval end minute	Min. queue length [#vehicles]	Max. queue length [#vehicles]
1	0	5	1	2
2	5	10	1	1
3	10	15	1	2
4	15	20	1	3
5	20	25	1	3
6	25	30	1	3
7	30	35	1	4
8	35	40	1	2
9	40	45	0	0
10	45	50	0	0
11	50	55	0	0

Table 27: Observed minimum and maximum queue lengths at the roundabout during the peak period

The observed minimal and maximum queue lengths for the off-peak period are shown in below.

Observed queue lengths at the roundabout in off-peak hour				
Interval #	Interval start minute	Interval end minute	Min. queue length [#vehicles]	Max. queue length [#vehicles]
1	0	5	1	2
2	5	10	1	1
3	10	15	1	4
4	15	20	1	5
5	20	25	1	5
6	25	30	1	3
7	30	35	1	2
8	35	40	1	1
9	40	45	1	1
10	45	50	1	3
11	50	55	0	0
12	55	60	0	0

Table 28: Observed minimum and maximum queue lengths at the roundabout during the peak period

E.2. Results the SGFC-junction

This section presents the results of the analyses of the camera observations conducted at the SGFC-junction at the crossing of the Oldenzaalsestraat and the Singels in Enschede. For four different hours the traffic situation has been analysed. The two observed peak hours are: Tuesday 13th of January 2015 between 16 o'clock and 17 o'clock and Thursday 15th of January 2015 between 8 o'clock and 9 o'clock. The two observed off-peak hours are: Wednesday 14th of January 2015 between 12 o'clock and 13 o'clock and between 15 o'clock and 16 o'clock.

E.2.1. Intensities per direction per vehicle type

below shows part of the observed traffic intensities at the SGFC-junction on the Tuesday afternoon. Per five minute interval the passing vehicles have been classified per direction and vehicle type. This data directly serves as input for the simulation model in the calibration process.

E.2.2. Mean headway and headway distribution of traffic

Table 29 below shows the recorded mean headways for the SGFC-junction during peak hour. The headways are determined for each of the three observed approaches separately.

Mean headway excluding gaps > 9.5 for SGFC-junction during rush hour				
	Motor vehicles		Bicycles	
Location	Average gap	# observations	Average gap	# observations
Laaressingel	4.24	222	5.00	3
Oldenzaalsestraat N	3.99	302	3.63	8
Lasondersingel	3.30	541	2.64	11
Weighted average	3.69		3.32	

Table 29: Mean headway excluding gaps > 9.5 for SGFC-junction during rush hour

The headway factor for each bicycle link in the SGFC-junction calibration model is set at: $3{,}32 \div 3{,}69 = 0{,}90$.

The headway distributions for motor vehicles on the three observed SGFC-junction approaches during the rush hour are displayed in Table 30 below.

Headway distribution of motor vehicle traffic at the SGFC-junction during rush hour excluding gaps > 9.5 s										
Arms	Relative importance of bins									Totals
Bin lower bound [s]	0.5	1.5	2.5	3.5	4.5	5.5	6.5	7.5	8.5	
Bin higher bound [s]	1.5	2.5	3.5	4.5	5.5	6.5	7.5	8.5	9.5	
Laaressingel	33%	0%	0%	0%	33%	0%	0%	0%	33%	100%
Oldenzaalsestraat N	25%	25%	13%	0%	13%	0%	13%	13%	0%	100%
Lasondersingel	11%	33%	11%	22%	11%	11%	0%	0%	0%	100%
Average	**23%**	**19%**	**8%**	**7%**	**19%**	**4%**	**4%**	**4%**	**11%**	**100%**

Table 30: Headway distribution of motor vehicle traffic at the SGFC-junction during rush hour excluding gaps greater than 9.5 seconds

The percentages in the bottom row of Table 30 are direct input for the nine sliders with which the aggression distribution is determined in S-Paramics.

E.2.3. Gap acceptance

Figure 40 below shows the accepted and rejected gaps at the SGFC-junction in Enschede. Compared to the gap acceptance graph for the roundabout, see Figure 39, two differences stand out. First because the framerate of one image per second the accepted and rejected gaps can only be determined in whole seconds. This leads to a broken function. The second difference is that there are only very few usable accepted gaps observed. Usually vehicles wanting to turn left waited until the traffic light of the opposing flow had turned red. This means that their accepted gap has no end because there are no more vehicles coming from the opposing direction, and therefore these gaps are unsuitable to use for determining the critical gap acceptance.

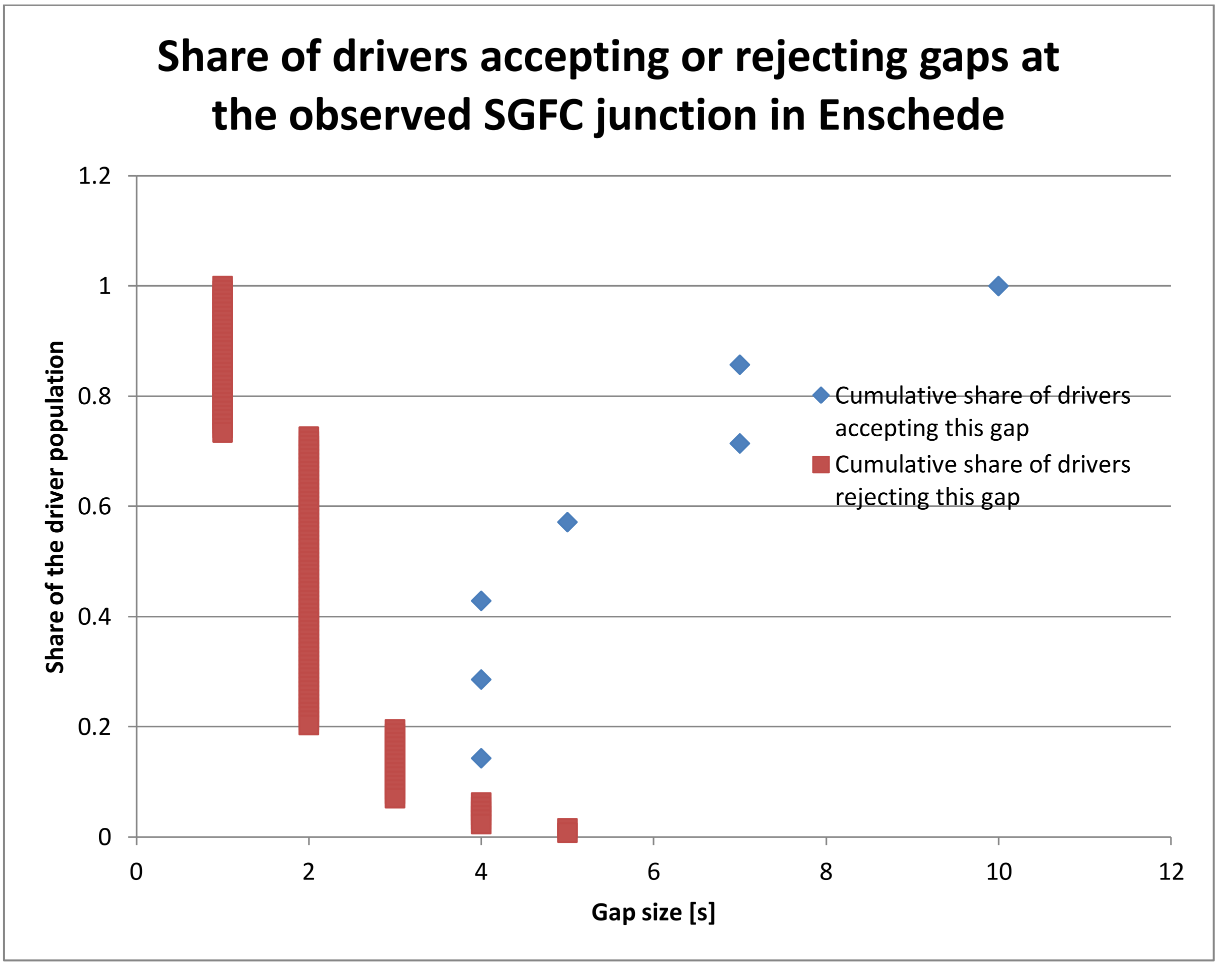

Figure 40: Share of drivers accepting or rejecting gaps at the observed SGFC-junction in Enschede

Because there are so little records of usable accepted gaps the accepted gap and rejected gap curve do not intersect anywhere. Raff's critical gap method is therefore unusable for determining an estimate of the critical gap. Therefore the same critical gap values as at the roundabout are tested during the SGFC-model calibration.

E.2.4. Observed queue lengths

The observed queue lengths have been aggregated into a minimum and a maximum number of vehicles per five minute interval per lane. Table 31 below shows the observations results for the Lasondersingel for the observed hour on Tuesday. The results as shown below are compared with the queue lengths generated as model output in the calibration process. The observed queue lengths for the other two observed junction approaches can be obtained through the author.

Minimal and maximal queue lengths in number of vehicles on the Lasondersingel on Tuesday between 16 and 17 o'clock										
Interval	Hour	Minute Start	Minimal queue right turn lane [#]	Maximal queue right turn lane [#]	Minimal queue straight through lane [#]	Maximal queue straight through lane [#]	Minimal queue left turn lane [#]	Maximal queue left turn lane [#]	Minimal queue bicycle lane [#]	Maximal queue bicycle lane [#]
1	16	0	1	3	2	7	1	7	1	1
2	16	5	1	3	3	9	4	6	1	1
3	16	10	1	1	3	13	1	8	0	0
4	16	15	0	0	1	16	2	8	1	1
5	16	20	3	3	2	17	1	9	1	2
6	16	25	1	1	2	10	2	14	1	2
7	16	30	1	1	3	4	1	7	0	0
8	16	35	1	1	2	11	1	10	1	2
9	16	40	1	1	4	10	6	11	2	3
10	16	45	1	1	1	11	5	11	4	5
11	16	50	2	2	3	7	5	6	1	3
12	16	55	1	2	5	8	1	8	3	3

Table 31: Minimum and maximum queue lengths in number of vehicles per five minute interval per lane during peak hour

The observed queue lengths for the off peak hour are shown in below. The observed queue lengths for the other two observed junction approaches can be obtained through the author.

Minimal and maximal queue lengths in number of vehicles on the Lasondersingel during the off peak hour										
Interval	Hour	Minute Start	Minimal queue right turn lane [#]	Maximal queue right turn lane [#]	Minimal queue straight through lane [#]	Maximal queue straight through lane [#]	Minimal queue left turn lane [#]	Maximal queue left turn lane [#]	Minimal queue bicycle lane [#]	Maximal queue bicycle lane [#]
1	12	0	1	1	5	12	2	9	1	1
2	12	5	1	3	2	9	2	5	1	2
3	12	10	1	3	5	8	2	8	1	2
4	12	15	1	1	3	9	1	6	1	2
5	12	20	1	3	1	5	1	4	2	3
6	12	25	4	4	1	16	4	5	1	1
7	12	30	1	1	2	14	1	4	1	2
8	12	35	1	2	7	14	2	9	1	2
9	12	40	1	2	2	21	2	4	1	3
10	12	45	1	2	1	9	2	14	1	1
11	12	50	0	0	1	7	1	4	1	3
12	12	55	1	2	2	10	2	6	1	3

Table 32: Minimum and maximum queue lengths in number of vehicles per five minute interval per lane during off peak hour

E.2.5. Red light compliance

As already mentioned in paragraph D.2 the red light violations are registered separately for motor vehicles and for bicyclists / light mopeds. In Table 33 below both the relative and absolute numbers of red light violations by motor vehicles split per direction are presented.

Absolute and relative red light violations by motor vehicles												
Direction	Laares Right	Laares Straight	Laares Left	OldZd Right	OldZd Straight	OldZd Left	Lasonder Right	Lasonder Straight	Lasonder Left	OldNrd Right	OldNrd Straight	OldNrd Left
Tue 16-17	0	1	0	0	0	0	0	0	1	0	0	0
Tue 16-17	0.000%	0.366%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.382%	0.000%	0.000%	0.000%
Wed 12-13	0	0	0	0	0	0	1	1	0	0	0	0
Wed 12-13	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	1.852%	0.280%	0.000%	0.000%	0.000%	0.000%
Wed 15-16	0	0	0	0	0	0	0	1	0	0	0	0
Wed 15-16	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.321%	0.000%	0.000%	0.000%	0.000%
Thu 8-9	0	0	0	0	0	0	0	3	2	0	0	0
Thu 8-9	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	0.000%	1.071%	1.117%	0.000%	0.000%	0.000%

Table 33: Absolute and relative number of red light violations by motor vehicles per direction

Table 33 shows that both in relative and in absolute numbers the amount of red light violations by motor vehicles is low. The highest share of red light violations is 1.852%, but this still corresponds with only one vehicle during one entire hour. The fact that red light violations by motor vehicles are not well represented in the S-Paramics simulation software will hardly cause deviations between the simulated junction performance and the observed performances. Therefore no workarounds or simulation model adaptations have to be done.

The red light violations by bicyclists or light mopeds are split into two categories: violations conducted when the parallel motor vehicle direction has green light and other violations. The observed violations on Tuesday between 16 o'clock and 17 o'clock are shown in Table 34 below. Because bicyclists are allowed to turn right on red, this direction is not included in this research.

Absolute and relative red light violations by bicycles and light mopeds on Tuesday between 16 and 17 o'clock								
Violation type	Laares Straight	Laares Left	Old. South Straight	Old. South Left	Lasonder Straight	Lasonder Left	Old. North Straight	Old. North Left
With parallel car green	1	0	0	0	0	0	0	0
Other	0	0	1	1	0	0	1	0
Total	1	0	1	1	0	0	1	0
Total	6.25%	0.00%	1.59%	12.50%	0.00%	0.00%	2.22%	0.00%

Table 34: Absolute and relative red light violations by bicycles and light mopeds on Tuesday between 16 o'clock and 17 o'clock

Table 34 shows that the amount of red light violations by bicycles and light mopeds is low. The relative amount of violations peaks at 12,50% which is a lot higher than the value for motor vehicles. However this still represents only one case. The higher relative amounts are merely caused by a lower amount of bicyclists. Because of the low amount of cases it is not possible to definitively determine which of the two types of violations occurs more often. For the other three observed hours the absolute amount of violations was comparable to the values for Tuesday as shown in Table 34. The relative amount of violations were lower during the other observed hours, partly because of higher total amounts of bicyclists. The results for these other hours can be obtained through the author. Overall the amount of red light violations by bicycles and light mopeds is so low that it will hardly cause a difference between the simulated and the observed junction performance. Therefore no workarounds or simulation model adaptations have to be done.

E.2.6. Priority behaviour of bicyclists and light mopeds

At the observed junction in Enschede the highest observed hourly intensity of bicycles and mopeds was 260, which is relatively low. In Table 35 below the occurrence of the three defined types of priority behaviour is shown for the four observed hours. The table shows that the vast majority of bicyclists did not have to give way to other bicyclists.

Priority behaviour of bicyclists and light mopeds			
	Priority for Singel	Priority for cyclist from the right	No need to organise priority
Tue 16-17	3	2	216
Wed 12-13	5	1	221
Wed 15-16	4	1	211
Thu 8-9	3	4	253

Table 35: Priority behaviour of bicyclists and light mopeds during their green phase

There is no clear favourite type of priority behaviour in the cases when it is needed to organise priority. There seems to be a slight preference for giving bicyclists on the Singel priority but the number of cases in which priority had to be organised is so small that it is not possible to draw a valid conclusion. Because giving priority to bicyclists on the Singel seems to be the most common way to organise priority this is the behaviour coded into the simulation model. At bicycle and light moped intensities up to at least 260 vehicles the programmed priority behaviour hardly affects junction performance because in almost all cases bicycles do not have to yield to others during their green phase.

E.2.7. Usage of bicycle lane by motor vehicles

At the observed SGFC-junction in Enschede it is possible that motor vehicles drive over the bicycle lane. With the camera observations the occurrence of motor vehicles driving on the bicycle lane has been recorded. Motor vehicles driving on the bicycle lane are classified in three different groups: right turning vehicles driving on the bicycle lane when the right turn lane was not blocked, right turning vehicles when access to the right turn lane was blocked and straight through driving vehicles. The observations for the Lasondersingel are shown in Table 36 below, the results for the Laaressingel in Table 37 and the results for the Oldenzaalsestraat North in Table 38. The first column in each of these tables notes the observed hour. The second and third column show the total number of motor vehicles on the bicycle lane and their relative share of total motor traffic on that arm. The fourth and fifth column show the absolute number of right turning vehicles that drove on the bicycle lane while the right turn lane was not blocked and their share in the total number of right turning vehicles that could freely enter the right turn lane. The sixth and seventh column show the absolute number right turning motor vehicles that drove on the bicycle lane while the normal access to the turn lane was blocked and their relative share in the total amount of right turning vehicles that were confronted with a blocked right turn lane. The eighth column shows the relative amount of right turning vehicles that drove on the bicycle lane of the total number of right turning vehicles. The ninth and tenth columns show the number of straight through travelling vehicles that drove on the bicycle lane and their share of the total number of through going motor vehicles.

Motor vehicles entering bicycle lane of Lasondersingel									
Observed hour	Total bike lane entries [#]	Relative bike lane entries of total traffic [%]	Bike lane entries by right turning traffic without blocked turn lane [#]	Relative bike lane entries of right turning traffic without blocked turn lane [%]	Bike lane entries by right turning traffic with blocked turn	Relative bike lane entries of right turning vehicles with blocked turn lane [%]	Relative bike lane entries by total right turning traffic [%]	Bike lane entries by through traffic [#]	Relative bike lane entries of through traffic [%]
Tue 16-17	6	0.9%	6	17.1%	0	0.0%	17.1%	0	0.0%
Wed 12-13	4	0.7%	3	6.0%	1	33.3%	7.5%	0	0.0%
Wed 15-16	4	0.7%	4	7.8%	0	0.0%	7.8%	0	0.0%
Thu 8-9	5	0.9%	5	6.6%	0	0.0%	6.6%	0	0.0%

Table 36: Motor vehicles entering the bicycle lane at the Lasondersingel approach

Table 36 shows that the absolute number of bicycle lane entries by motor vehicles on the Lasondersingel is small. The vast majority of bicycle lane entries are done by right turning vehicles when the normal access to the turn lanes was open. Only one right turning vehicle used the bicycle lane to evade a queue blocking the normal entry to the turn lane. This means that the motor vehicles using the bicycle lane hardly influence the capacity of the Lasondersingel approach of the junction. None of the through travelling vehicles drove in the bicycle lane. Based only on the observations at the Lasondersingel the fact that motor vehicles can use the bicycle lane will not cause a noticeable difference between the simulated and the observed results because of the low amount of vehicles that actually do this.

Motorvehicles entering bicycle lane at the Laaressingel									
Observed hour	Total bike lane entries [#]	Relative bike lane entries of total traffic [%]	Bike lane entries by right turning traffic without blocked turn lane [#]	Relative bike lane entries of right turning traffic without blocked turn lane [%]	Bike lane entries by right turning traffic with blocked turn lane [#]	Relative bike lane entries of right turning vehicles with blocked turn lane [%]	Relative bike lane entries of total right turning traffic [%]	Bike lane entries by through traffic [#]	Relative bike lane entries of through traffic [%]
Tue 16-17	15	3.8%	11	13.4%	3	60.0%	16.1%	1	0.3%
Wed 12-13	15	3.5%	7	9.1%	8	88.9%	17.4%	0	0.0%
Wed 15-16	21	5.4%	11	17.2%	7	87.5%	25.0%	3	1.0%
Thu 8-9	25	4.5%	2	4.3%	23	71.9%	31.6%	0	0.0%

Table 37: Motor vehicles entering the bicycle lane at the Laaressingel approach

On the Laarressingel the amount of motor vehicles that use the bicycle lane is both absolutely and relatively high, as shown in Table 37 above. Especially on Thursday morning a high number of right turning vehicles used the bicycle lane to pass the straight through queue blocking the right turn lane. The Laaressingel is the only junction approach where through going motor vehicles, although in limited numbers, drove on the bicycle lane. This does not affect the junction throughput or queue forming because these vehicles do not avoid a queue by driving on the bicycle lane. There is a limited negative impact on the traffic safety of bicyclists. Given the relative high usage of the bicycle lane on this approach it is necessary to adjust the simulation model used in the calibration process. To compensate for the higher junction capacity and shorter queue on the through lane the right turn lane in the simulation model is modelled longer than in reality. This way the simulated right turn lane is blocked less often and thus mimicking the higher accessibility of the right turn lane that in reality is achieved by motor vehicles using the bicycle lane.

Motorvehicles entering bicycle lane of Oldenzaalsestraat									
Observed hour	Total bike lane entries [#]	Relative bike lane entries of total traffic [%]	Bike lane entries by right turning traffic without blocked turn lane [#]	Relative bike lane entries of right turning traffic without blocked turn lane [%]	Bike lane entries by right turning traffic with blocked turn lane [#]	Relative bike lane entries of right turning vehicles with blocked turn lane [%]	Relative bike lane entries of total right turning traffic [%]	Bike lane entries by through/ left	Relative bike lane entries of through/ left traffic [%]
Tue 16-17	0	0.0%	0	0.0%	0	0.0%	0.0%	0	0.0%
Wed 12-13	1	0.2%	0	0.0%	1	20.0%	0.5%	0	0.0%
Wed 15-16	0	0.0%	0	0.0%	0	0.0%	0.0%	0	0.0%
Thu 8-9	11	1.4%	0	0.0%	11	61.1%	4.2%	0	0.0%

Table 38: Motor vehicles entering the bicycle lane at the Oldenzaalsestraat North approach

Table 38 shows that the amount of motor vehicles that enter the bicycle lane on the Oldenzaalsestraat is very low. Only right turning vehicles that were confronted with a blocked turn lane used the bicycle lane. Specifically during the hour on Thursday a large share of right turning vehicles did this. None of the through travelling vehicles drove in the bicycle lane. To compensate for the right turning vehicles that use the bicycle lane the junction model used in the calibration process is adjusted. The right turn lane is modelled longer than it is in reality.

The amount of motor vehicles using the bicycle lane varies greatly over the three investigated junction arms. This can be explained by the different layouts of the arms. The Laaressingel approach lies in a right turn. Drivers, especially those turning right, are inclined to turn right a bit too sharp or too early and therefore end up on the bicycle path. The Oldenzaalsestraat North lies in a slight left turn. Because drivers have to steer left here they turn away from the bicycle lane and thus almost never enter it. The approach on the Lasondersingel is almost straight and regarding the amount of bicycle lane entries this approach lies between the other two approaches. Another factor contributing to the high amount of bicycle lane entries on the Laaressingel is the length of the right turn lane. The right turn lane on the Laaressingel can hold about seven cars compared to ten for the right turn lane on the Lasondersingel and fifteen on the Oldenzaalsestraat.

The fact that motor vehicles drive on the bicycle lane only requires adjustments to the simulated junction model used for the model calibration. The junction design used for the final assessment of SGFC-junctions will have bicycle lanes fully separated from the motor vehicle lanes in accordance with the CROW guidelines published in the ASVV (CROW, 2012). With this design it is not possible that motor vehicles use the bicycle lane and therefore it is not necessary to make adjustments to the simulation model.

F. Calibration of the models

To increase the accuracy with which the simulations represent reality the models have to be adjusted to match the Dutch traffic situation. As described earlier in chapter 3.1 this study uses three different kinds of software: S-Paramics, AIRE emissions module and SWOV TTC module. The calibrations are based on camera observations conducted at one roundabout and at one SGFC-junction. Appendix 0 describes how these camera observations are conducted and how the data described in the current chapter is extracted from the images. Paragraph F.1 describes what steps need to be taken in preparation of the calibration. Paragraph F.2 describes how the S-Paramics model is calibrated regarding capacity. The output of the SWOV TTC module depends on the input generated by S-Paramics.

The AIRE emissions module cannot be calibrated because it there are no emissions measurements available. There are however some model inputs that have to be adjusted to the Dutch traffic situation. Paragraph F.4 describes how this is done. Like any model the Paramics simulation software is a simplified representation of reality and thus has limitations. Therefore not everything that influences junction performance in reality is included in the model. It is therefore not possible to exactly reproduced the observed situation and this can cause differences between the model outputs and observations and thus influence model calibration. The final paragraph describes these model limitations.

Figure 41 below presents a schematic overview of the calibration process. From the camera observations three types of data are extracted: values for certain model input variables (denoted by arrow 1.), values for the calibration target variables (denoted by arrow 2.) and first estimates for the critical gap (Gap acceptance box). Naturally the model input values extracted from the camera observations are implemented in the S-Paramics model, denoted by arrow 3. With these input values the S-Paramics simulation is run and the model produces, amongst others, output for the calibration objective variables, denoted by arrow 4. The comparison between the simulated values for the calibration target variables and the observed values is the main step from the calibration, denoted by arrow 5. In case that the simulated values do not mirror the observed values close enough the simulation model is run again with adjusted calibration adjustment variables, this is represented by arrow 6. Steps five and six are repeated until the model represents the observations close enough.

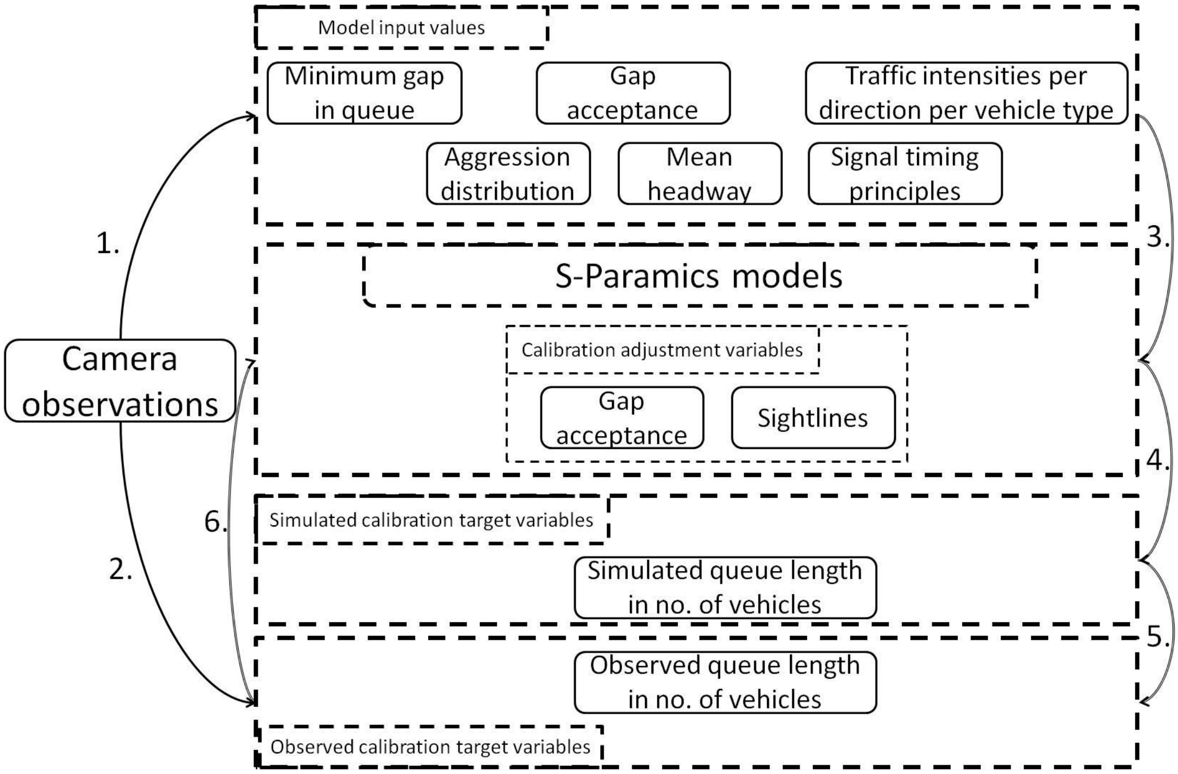

Figure 41: Schematic overview of the calibration process

F.1. Preparation for calibration

Before the calibration can be conducted the S-Paramics input has to correspond with the location and traffic situation of where the camera observations were taken. The geometric layouts of the two observed junctions have to be accurately represented in the simulation model. The junction layout is based on available aerial photographs from Google Maps.

Regarding the traffic situation several different characteristics need to be provided: the traffic intensities per turning direction, the headway distribution and the average distance between queued vehicles. All these traffic related inputs are based on camera observations.

The traffic intensities have to be gathered because the junction performance is related to these intensities. Higher intensities will lead to higher queue lengths, waiting times and a higher accident risk. It is important to specify the traffic intensities per turning direction because the turning direction determines the number of conflicts with other streams. The traffic intensities are specified by vehicle type as well. Because Heavy Goods Vehicles (HGV's) take up more space and have lower manoeuvrability and acceleration rates they have a negative impact on junction capacity (when expressed in vehicles per unit of time). The different vehicle types that are distinguished are: cars, vans, HGV's, articulated HGV's and bicycles. These traffic intensities are summed at five minute intervals, because this is the smallest interval at which a traffic intensity profile can be defined in S-Paramics (SIAS Limited, 2011).

The headway distribution describes the gaps that occur between vehicles in a traffic flow. This determines the likelihood that an acceptable gap occurs in a conflicting traffic flow and thus influences the capacity and delay at priority controlled junctions. The headway distribution also influences the performance at signalised junction. The headway distribution determines at what rate the queues grow and how that rate varies over time.

The length of a queue, in meters, influences the junction performance because it determines the distance from the N^{th} vehicle to the stop/yield line and whether or not access to certain lanes are blocked. The distance from the N^{th} vehicle to the stop/yield line influences the time it takes for that vehicle to reach the junction and thus the amount of vehicles that can pass during a green phase or during a gap in the roundabout flow. Specifically for the SGFC-junction the queue length determines whether or not entry to the turning lanes is blocked.

The queue length in meters is influenced by the number of vehicles in it, the lengths of the vehicles in it and the distance between the queued vehicles. The number of vehicles in the queue is not a model input but model output and can therefore not be entered by the user. The composition of vehicle types in the queue is determined by the intensities of the different vehicle types, see above. The distance between queued vehicles is model input as well and is determined based on the camera observations. The distance between queued vehicles is controlled by the "minimum gap in queue" parameter in S-Paramics (SIAS Limited, 2012).

Although the gap acceptance is one of the variables that is adjusted during the calibration process it is also adjusted as preparation for the model calibration. Especially for the roundabout it is important to observe the gap acceptance behaviour of drivers. At roundabouts vehicles willing to enter sometimes unnecessary wait for vehicles on the roundabout that turn out to leave the roundabout even though they did not signal this in advance. This is called an apparent conflict ("schijnconflict"). Even though there is no physical conflict point because the driver on the roundabout left before crossing the approach lane, the driver on the approach has waited for the vehicle on the roundabout and thus was hindered by it. Because the simulated drivers in S-Paramics will not 'forget' to signal before a turn this apparent conflict and the resulting lower roundabout capacity is not directly represented in the simulations. The apparent conflicts that occur in reality will increase the length of the average accepted gap. By setting this higher average accepted gap as input in S-Paramics the simulations will take the lower capacity caused by apparent conflicts into account.

For the SGFC-junction the signal timings are a further model input that the user has to match to the observed conditions. The difficulty of trying to match the signal timings is that these are not static but dynamically adapted to the traffic conditions. To represent this a signal plan that has similar logic, has to be written. The principles that the junction control logic follows are deduced from the observations. These principles include: the minimum and maximum green time per direction based on the presence of vehicles and the order of the phases. Constants that have to be implemented in the signal plan are the clearance times that occur between the different phases and the default amber time.

F.2. Capacity calibration

The goal of the calibration of Paramics is to make the simulation accurately represent traffic flows so that the capacity and the time losses occurring at junctions can be correctly simulated. The calibration of the roundabout model and the SGFC-junction are two parallel processes. For both the processes the goal is to match the simulated queue lengths, in number of vehicles, with the observed queue lengths.

If the model inputs, described in the previous paragraph, match with the observations and the model output, queues in number of vehicles, matches the observed values than this means that the junction capacity corresponds with reality. A realistically simulated junction capacity ensures that the model outputs regarding throughput are reliable.

If it turns out that the simulated queue lengths do not represent the observed queue lengths good enough adjustments to at least one of the following parameters will be made in order to improve the accuracy of the simulations.

The first parameter that to adjust is the sightline parameter. The parameter sightlines in Paramics defines from how far away drivers can see other, potential conflicting, vehicles coming. This variable therefore impacts at what time drivers are able to detect a gap in the conflicting traffic flow and thus indirectly influences gap acceptance. The larger the sightline distance the sooner vehicles can detect a gap in the conflicting traffic flow increasing the probability that a potential gap is used and thus influencing junction capacity.

The second variable is the gap acceptance. The gap acceptance behaviour has a big effect on roundabout capacity. The smaller the average gap in the roundabout traffic flow that drivers on the approach accept the more suitable gaps will occur and thus the higher the roundabout capacity. Even though the traffic lights separate several conflicting traffic flows in time there are still some conflicts left. In those cases the gap acceptance determines what time gap between conflicting vehicles is big enough to proceed and thus influences traffic flow and junction capacity.

F.3. Traffic safety adjustments

The traffic safety module uses input generated by S-Paramics. Therefore improving the accuracy of traffic safety requires changing settings in S-Paramics and not in the safety module itself. The gap acceptance is one of the S-Paramics variables that influences traffic safety. When the modeller sets a lower value the vehicles from conflicting traffic flows will cross/ merge closer to each other lowering the TTC of the involved vehicles and thus increasing the number of TTC conflicts. This parameter is already adjusted in the calibration process regarding junction capacity and is therefore already set at the right value.

Another parameter influencing the number of TTC-conflicts is the mean headway. This parameter mainly influences the probability of vehicles in the same traffic flow coming into conflict with each other (rear-end collisions).

A third parameter that influences the number of TTC-conflicts is the aggression distribution. This parameter describes how aggressiveness is distributed over the vehicle fleet. The level of aggressiveness assigned to a vehicle influences several aspects of vehicle behaviour: gap acceptance, desired headway, desired speed and flexibility in route choice. The first three of these aspects are relevant for this research. The distribution of aggressiveness thus determines how the accepted gaps, the headway and the desired vehicle speed are distributed around the user defined values.

The mean headway and the aggression distribution are adjusted to match the observed mean headway and mean headway distribution respectively.

F.4. AIRE Emissions module input

The AIRE emissions module cannot be calibrated to local Dutch circumstances because there are no emissions measurements available. However to improve the reliability of the outputs it is important that the model input is adjusted to match Dutch traffic conditions. The two input types that the AIRE software needs are: traffic flow data and vehicle fleet composition.

The traffic flow data is output generated by the S-Paramics model. By calibrating the S-Paramics model this input for the AIRE module is representative of Dutch traffic conditions.

The vehicle fleet composition input is described by three different variables: the fuel split, the vehicle size and the EU Emission standard (Transport Scotland, 2011). Data on these distributions for the total vehicle fleet in the Netherlands are available through the Central Bureau for Statistics (CBS). Several datasheets give information about the total kilometres travelled of different vehicle types.

There is a sheet available describing the total kilometres travelled of cars specified by fuel type; petrol, diesel or LPG (CBS, 2014). Another sheet describes the same information for vans (CBS, 2014). A similar sheet for lorries does not contain a distribution over fuel type since all lorries drive on diesel (CBS, 2014). This data sheet does contain divisions over different weight classes and the production year of the lorries. The production year information can be used to determine the EU Emission standard to which the lorries adhere.

F.5. Simulation model limitations

Like any other model the S-Paramics simulation model is a simplified representation of reality. This means that some aspects of traffic and driver behaviour that do occur in reality cannot be simulated.

One important aspect that is not represented well in the simulations software is rule compliance. In reality not all drivers will comply with the traffic laws. However the software is not always able to represent this. Regarding speed limits the rule compliance can be controlled by the user. The speed a simulated vehicle drives is based on four variables: the desired speed of the driver, the aggressiveness of the driver, vehicle characteristics and the posted speed limit. It is therefore possible that simulated drivers go faster than the posted speed limit.

Red light violations are however not well represented in the simulations. Only violations that happen within the first second after the traffic light turned from amber to red can occur in the simulation model. The amount of red light violations will affect the junction's performance: the traffic safety will decrease with a higher amount of red light violations. The occurrence of red light violations and thus an indication of how much the simulation results potentially overestimate traffic safety at the SGFC-junction is deduced from the camera observations.

Another aspect at which the simulation model might not represent well is the priority behaviour of bicycles during their dedicated green phase. Because all the bicycle traffic lights turn green at the same time all of the bicycle flows have equal priority. Road markings or traffic laws like "drivers from the right have priority" formally do not apply in this situation. It is therefore up to the bicyclists themselves to determine who gets priority. This may lead to variety in the priority rules that are applied. This variety is something that is not well represented in the simulation model, where only one type of priority can be applied. In order to determine which type of priority rule needs to be applied in the simulation model the occurrence of different types of priority is extracted from the video observations.

Specifically at the observed SGFC-junction in Enschede there is another type of traffic behaviour that occurs in reality but cannot be simulated in the model. The approaches to the observed junction have a bicycle lane connected with the carriageway, instead of a separate bicycle path. This means that motor vehicles can drive on the bicycle lane. On approaches with a separate right turn lane motorists can use the bicycle lane to bypass long queues on the through lane that would otherwise block the entrance to the right turn lane. In the simulation model motor vehicles are not able to drive on the bicycle path. Depending on the occurrence of this behaviour it is necessary to use a workaround for the calibration of the model. This workaround consists of a longer right turn lane in the simulation than at the observed location in Enschede. Because of the longer right turn lane the entry to this lane is blocked less often by queues on the trough lane thus mimicking the higher accessibility to the turn lane that in reality is achieved by motorists using the bicycle lane.

G. Results of model calibration and validation

The goal of the calibration process is to make the simulated junction capacity correspond as closely as possible to the observed junction capacity. This is done by matching the simulated queue lengths with the observed queue lengths. For each five minute interval the minimum and maximum queue length are compared.

This chapter discusses for both the roundabout and the SGFC-junction what tests are used to determine how well the simulated queues represent the observed queues and how each of the models scored. The results for the roundabout are discussed first, followed by the results for the SGFC-junction.

G.1. Calibration of the roundabout model

As already mentioned in paragraph F.2 the first variable that is adjusted in the calibration process is the visibility. On all the motor vehicle links where drivers have to make a gap acceptance decision the visibility is varied. Per situation twenty model runs are conducted. Both the average maximum and average minimum queue length per five minute interval as well as the individual values of each run are compared with the observations. The Mean Squared Error (MSE) is used to determine how well the simulated queue lengths fit the observations. The results of this comparison are shown in Table 39 below.

Variant description	Visibility				MSE for comparison with average of 20 runs		MSE for comparison with individual runs	
	Approaches to bike circle	Approaches to motor vehicle circle	circle lane	circle lane exit	Min. queue	Max. queue	Min. Queue	Max. queue
All 0 m	0	0	0	0	0.99	2.71	1.60	3.23
All 2,5 m	2.5	2.5	2.5	2.5	0.97	**1.42**	1.54	**2.90**
All 5,0 m	5	5	5	5	0.53	1.45	0.98	3.71
All 10 m	10	10	10	10	**0.24**	2.72	**0.79**	3.51
25 m and 10 m	25	10	10	10	0.51	3.09	1.04	4.46

Table 39: Mean Squared Error values for the comparison of sets of simulated queue lengths with different visibility values with observed queue lengths

The simulation model with the lowest MSE value has smallest difference with the observed values, and is thus the best fitted model. In Table 39 the lowest MSE value per column is highlighted. Table 39 shows that the model variant with a visibility of 2,5 meters scores best for the comparison with the maximum queue length. However the model that scores best for the comparison with the minimum queue length is the variant with a visibility of 10 meters. This indicates that the simulation model cannot be improved further by adjusting the visibility. A better fit with the minimum queue lengths results in a worse fit with the maximum queue lengths. For the analysis of the different junction designs it is more interesting to have an accurate prediction of the maximum queue length than an accurate prediction of the minimum queue length. Firstly because the maximum queue length is normative for the length of dedicated turn lanes. Secondly because it indicates if other junctions are at risk of being blocked. Therefore the visibility is set at 2,5 meter.

To further improve the fit of the simulated queue lengths with the observed queue additional simulations with different values for the Gap Acceptance parameter are run. The MSE values for these sets of runs are shown in Table 40 below.

Variant	MSE for comparison with average of 20 runs		MSE for comparison with individual runs	
Gap acceptance [s]	Min. queue	Max. queue	Min. Queue	Max. queue
2.00	1.19	1.75	**1.27**	3.29

2.68	**0.80**	1.94	1.29	3.14
5.00	0.96	1.75	1.68	3.56
5.43	0.97	**1.42**	1.54	**2.90**
6.00	1.17	1.49	1.66	3.04
6.50	1.29	1.48	1.83	3.92
7.00	0.96	1.97	1.33	2.91

Table 40: Mean Squared Error values for the comparison of sets of simulated queue lengths with different gap acceptance values with observed queue lengths

In Table 40 the lowest MSE values per column have again been highlighted. These values show that the critical gap value of 5.43, as found with Raff's critical gap method, provide the best fit for the maximum queue values. For the minimal queues lower values provide the best fit.

Because the absolute value of the MSE is related to the variables used in the calculation it is not suitable for determining if a simulation model fits the observations good enough. The MSE can merely determine which model variant fits the data best relative to other model variants. To determine if the simulation model fits the data good enough a statistical test has to be performed.

G.1.1. Statistical test

To determine if the roundabout simulation model represents the observed situation accurately enough a statistical test has to be performed. The first step is to determine which test is suitable for this situation. In this case the values of a group, the 20 simulation runs, are compared with one observed value. The data, queue lengths in number of vehicles, are of the numerical continuous type. According to a decision tree published by the University of Twente the default statistical test to us in this situation is the One-Sample T-test (Universiteit Twente, 2015).

An additional prerequisite for the One-Sample T-Test is that the data has to follow the normal distribution (Houghton Mifflin Harcourt, 2014). Several tests are available for checking if data follows a normal distribution for example: the Shapiro-Wilk test, Anderson-Darling test and Kolmogorov-Smirnov test for normality. A study by Stephens shows that in general the Shapiro-Wilk test has the highest statistical power (Stephens, 1974). However the Shapiro-Wilk test does not work as well in situations with a lot of ties, i.e. when the same number of queued vehicles present multiple times. Because there are a lot of ties in the queue length data the normality of the simulated data is tested with the Kolmogorov-Smirnov test (K-S test) for normality, which handles this better.

An overview of the results from the K-S test for normality is presented in Table 41 below. This table shows that for the majority of time intervals the simulated queue length outputs are not normally distributed. Therefore the One-Sample T-test cannot be used to determine if the simulations match the observations good enough.

Test for normal distribution of roundabout calibration simulation data				
Time interval	**Interval start minute**	**K-S for normality test value**	**K-S for normality critical value**	**Type of distribution**
1	**0**	0.499	0.294	Not Normal
2	**5**	0.387	0.294	Not Normal
3	**10**	0.247	0.294	Normal
4	**15**	0.308	0.294	Not Normal
5	**20**	0.171	0.294	Normal
6	**25**	0.179	0.294	Normal

7	**30**	0.210	0.294	Normal
8	**35**	0.292	0.294	Normal
9	**40**	0.366	0.294	Not Normal
10	**45**	0.475	0.294	Not Normal
11	**50**	0.438	0.294	Not Normal

Table 41: Test for normal distribution of roundabout calibration simulation data.

Given that the simulated queue lengths do not follow the normal distribution and that the data is numerical continuous the Kolmogorov-Smirnov test is suitable for determining if the simulated queue lengths follow the observed queue lengths close enough. The K-S test for normality conducted previously is an adapted form of the K-S test.

The results of the Kolmogorov-Smirnov test are shown in Table 42 below. The first column shows all the occurring maximum queue length per five minute interval values. In case of simulated values this is the average maximum of twenty runs. Columns two and three show the cumulative frequency of how often the observed values occurred for the observations and simulations respectively. Columns four and five present these cumulative frequencies as a share of the total number of observed five minute intervals, which is in both cases eleven. Column six presents the K-S test statistic which is the difference between the cumulative share of observations in column four and the cumulative share of simulations in column five. Finally column six presents the threshold value directly extracted from the K-S test table based on both the sample sizes. Table 42 shows that the K-S test statistic always lies below the threshold value. This means that the K-S test cannot prove that the simulated maximum queue lengths differ significantly from the observed values. Therefore the model calibration is successful.

(Average) Queue lengths [# vehicles]	Frequencies		Cumulative share of observations [%]	Cumulative % of simulations	K-S test statistic	Threshold value
	Cumulative observed	Cumulative Simulated				
0.00	1	0	0.090909	0.000000	0.090909	0.579906
1.00	4	1	0.363636	0.090909	0.272727	0.579906
1.20	4	2	0.363636	0.181818	0.181818	0.579906
1.25	4	3	0.363636	0.272727	0.090909	0.579906
1.47	4	4	0.363636	0.363636	0.000000	0.579906
1.61	4	5	0.363636	0.454545	0.090909	0.579906
1.95	4	6	0.363636	0.545455	0.181818	0.579906
2.00	6	6	0.545455	0.545455	0.000000	0.579906
2.15	6	7	0.545455	0.636364	0.090909	0.579906
2.40	6	8	0.545455	0.727273	0.181818	0.579906
3.00	8	8	0.727273	0.727273	0.000000	0.579906
3.10	8	9	0.727273	0.818182	0.090909	0.579906
3.75	8	10	0.727273	0.909091	0.181818	0.579906
4.00	9	10	0.818182	0.909091	0.090909	0.579906
4.30	9	11	0.818182	1.000000	0.181818	0.579906
5.00	11	11	1.000000	1.000000	0.000000	0.579906

Table 42: Results for the Kolmogorov-Smirnov test for the calibration of the roundabout model

G.1.2. Validation of the roundabout simulation model

The roundabout simulation model is validated with data from an off-peak observation period. The observation location is still the same. The Kolmogorov-Smirnov test is used again to check if the simulation results represent the observations accurately enough.

(Average) Queue lengths [# vehicles]	Frequencies		Cumulative share of observations [%]	Cumulative % of simulations	K-S test statistic	Threshold value
	Cumulative observed	Cumulative Simulated				
0	3	0	0.27272727	0	0.272727	0.579906
1	4	3	0.36363636	0.272727	0.090909	0.579906
1.11	4	4	0.36363636	0.363636	0	0.579906
1.27	4	5	0.36363636	0.454545	0.090909	0.579906
1.28	4	6	0.36363636	0.545455	0.181818	0.579906
1.31	4	7	0.36363636	0.636364	0.272727	0.579906
1.65	4	8	0.36363636	0.727273	0.363636	0.579906
2	7	8	0.63636364	0.727273	0.090909	0.579906
2.1	7	9	0.63636364	0.818182	0.181818	0.579906
2.45	7	10	0.63636364	0.909091	0.272727	0.579906
3	10	10	0.90909091	0.909091	0	0.579906
3.75	10	11	0.90909091	1	0.090909	0.579906
4	11	11	1	1	0	0.579906

Table 43: Results for the Kolmogorov-Smirnov test for the validation of the roundabout model

Table 43 below shows the results for the Kolmogorov-Smirnov test done for the model validation. The comparison between the K-S test statistic in the sixth column and the thresholds value in the seventh column shows that the test statistic is always smaller than the threshold value. This means that the K-S test cannot prove that the simulated queue lengths differ significantly from the observed queue lengths and therefore the model validation is successful.

G.2. Calibration of the SGFC-junction model

Just as with the roundabout model the first variable that is adjusted in the SGFC-junction model in the calibration process is the visibility. The MSE values in Table 44 below show the which visibility value offers the best fit of the modelled queue lengths to the observed queue lengths. At the SGFC-junction queue length data is gathered on three approaches and therefore three MSE values are calculated for the observed peak hour. The MSE values presented in Table 44 below are the averages over these three approaches.

Variant name	Visibility [m]	MSE for comparison with average of 20 runs averaged over three arms		MSE for comparison with individual runs averaged over three arms	
		Min. queue	Max. queue	Min. queue	Max. queue
Vis. 0,0 ALL	0.0	6.95	53.60	9.12	80.93
Vis. 2,5 ALL	2.5	6.98	45.87	**8.72**	69.12
Vis. 5,0 ALL	5.0	**6.84**	47.63	8.96	71.93
Vis. 10,0 ALL	10.0	6.86	**41.61**	9.15	**66.88**
Vis. 25,0 ALL	25.0	7.70	50.59	9.45	69.74

Table 44: MSE values for different visibility settings of the SGFC-junction

Table 44 shows that for the best representation of the maximum queue lengths a visibility setting of 10,0 m is required. For the best representation of the minimal queue length either settings of 2,5 m or 5,0 m give the best result. To ensure a good comparability between the two junction types' performances ideally all other settings should be the same for both models. For the roundabout a visibility of 2,5 m is chosen. Table 44 shows that for the maximum queue lengths a visibility of 2,5 m closely follows the goodness of fit of the 10,0 m setting. Given that it is desirable for the comparability of simulation outputs and acceptable regarding the goodness of fit a visibility value of 2,5 m is chosen for the SGFC-junction as well.

To further improve the fit of the simulated queue lengths with the observed queue additional simulations with different values for the Gap Acceptance parameter are run. The MSE values for these sets of runs are shown in Table 45 below.

Variant name	Gap acceptance [s]	MSE for comparison with average of 20 runs averaged over three arms		MSE for comparison with individual runs averaged over three arms	
		Min. queue	Max. queue	Min. queue	Max. queue
GA 2,00	2.00	7.46	52.32	9.25	72.91
GA 2,68	2.68	6.91	51.82	8.96	76.33
GA 5,00	5.00	7.03	52.18	8.83	77.15
GA 5,43	5.43	**6.62**	46.59	8.68	71.70
GA 6,00	6.00	7.16	47.39	8.77	66.83
GA 6,50	6.50	6.63	48.01	**8.55**	71.76
GA 7,00	7.00	6.74	**42.00**	8.78	**62.41**

Table 45: MSE values for different gap acceptance settings of the SGFC-junction

Table 45 indicates that for the most accurate representation of the maximum queue lengths a gap acceptance setting of 7,00 seconds is required. For the minimum queue lengths gap acceptance values of either 5,43 or 6,50 seconds score best. For the roundabout a gap acceptance value of 5,43 s offered the best results. Again given that it is desirable for the comparability of simulation outputs and acceptable regarding the goodness of fit a visibility value of 2,5 m is chosen for the SGFC-junction as well.

G.2.1. Statistical test

To determine which statistical test is applied to check if the calibration is successful the normality of the data is tested using the K-S test for normality. The results of the K-S test for normality are shown in Table 46 below.

K-S test for normal distribution of SGFC-junction model with visibility of 2,5m and gap acceptance of 5,43 s					
Time interval	Interval start minute	Junction arm	K-S for normality test value	K-S for normality critical value	Type of distribution
1	0	Lasondersingel	0.139669	0.294	Normal
1	0	Oldenzaalsestraat North	0.183902	0.294	Normal
1	0	Laaressingel	0.184414	0.294	Normal
2	5	Lasondersingel	0.135395	0.294	Normal
2	5	Oldenzaalsestraat North	0.16616	0.294	Normal
2	5	Laaressingel	0.122156	0.294	Normal
3	10	Lasondersingel	0.136898	0.294	Normal
3	10	Oldenzaalsestraat North	0.21513	0.294	Normal
3	10	Laaressingel	0.155364	0.294	Normal
4	15	Lasondersingel	0.149559	0.294	Normal
4	15	Oldenzaalsestraat North	0.160441	0.294	Normal
4	15	Laaressingel	0.157527	0.294	Normal
5	20	Lasondersingel	0.171663	0.294	Normal
5	20	Oldenzaalsestraat North	0.143754	0.294	Normal
5	20	Laaressingel	0.17675	0.294	Normal
6	25	Lasondersingel	0.165248	0.294	Normal
6	25	Oldenzaalsestraat North	0.168426	0.294	Normal
6	25	Laaressingel	0.109152	0.294	Normal
7	30	Lasondersingel	0.12134	0.294	Normal
7	30	Oldenzaalsestraat North	0.0963	0.294	Normal
7	30	Laaressingel	0.132922	0.294	Normal
8	35	Lasondersingel	0.120763	0.294	Normal
8	35	Oldenzaalsestraat North	0.189915	0.294	Normal
8	35	Laaressingel	0.160864	0.294	Normal
9	40	Lasondersingel	0.121933	0.294	Normal
9	40	Oldenzaalsestraat North	0.139508	0.294	Normal
9	40	Laaressingel	0.195654	0.294	Normal
10	45	Lasondersingel	0.216937	0.294	Normal
10	45	Oldenzaalsestraat North	0.145672	0.294	Normal
10	45	Laaressingel	0.15544	0.294	Normal
11	50	Lasondersingel	0.203397	0.294	Normal
11	50	Oldenzaalsestraat North	0.104018	0.294	Normal
11	50	Laaressingel	0.125371	0.294	Normal
12	55	Lasondersingel	0.188869	0.294	Normal
12	55	Oldenzaalsestraat North	0.144285	0.294	Normal
12	55	Laaressingel	0.154041	0.294	Normal

Table 46: K-S test for normal distribution of the simulated queue lenghts for the calibrated SGFC-junction model

Table 46 shows that for all approaches and all time intervals the K-S test for normality test value is lower than the threshold value. This means that all the simulated maximum queue lengths show a normal variation over the twenty different runs. This means that the one sample t-test is used to assess if the SGFC-junction model is calibrated good enough.

In Table 47 below the results of the one sample t-test are shown.

One sample t-test for the SGFC-junction model with a visibility of 2,5 m and a gap acceptance value of 5,43 s					
Time interval	**Interval start minute**	**Junction arm**	**One sample t-test value**	**One sample t-test critical**	**P-value**
1	**0**	Lasondersingel	-0.83	2.43	0.42
1	**0**	Oldenzaalsestraat North	-0.04	2.43	0.97
1	**0**	Laaressingel	-0.21	2.43	0.84
2	**5**	Lasondersingel	-0.65	2.43	0.52
2	**5**	Oldenzaalsestraat North	-0.49	2.43	0.63
2	**5**	Laaressingel	-0.11	2.43	0.92
3	**10**	Lasondersingel	-0.23	2.43	0.82
3	**10**	Oldenzaalsestraat North	-0.59	2.43	0.56
3	**10**	Laaressingel	0.09	2.43	0.93
4	**15**	Lasondersingel	-0.39	2.43	0.70
4	**15**	Oldenzaalsestraat North	0.08	2.43	0.94
4	**15**	Laaressingel	0.01	2.43	0.99
5	**20**	Lasondersingel	-0.02	2.43	0.99
5	**20**	Oldenzaalsestraat North	0.15	2.43	0.89
5	**20**	Laaressingel	0.11	2.43	0.91
6	**25**	Lasondersingel	0.50	2.43	0.62
6	**25**	Oldenzaalsestraat North	0.69	2.43	0.50
6	**25**	Laaressingel	0.49	2.43	0.63
7	**30**	Lasondersingel	0.14	2.43	0.89
7	**30**	Oldenzaalsestraat North	-0.46	2.43	0.65
7	**30**	Laaressingel	0.08	2.43	0.93
8	**35**	Lasondersingel	-0.29	2.43	0.77
8	**35**	Oldenzaalsestraat North	-0.32	2.43	0.75
8	**35**	Laaressingel	-0.29	2.43	0.77
9	**40**	Lasondersingel	0.02	2.43	0.99
9	**40**	Oldenzaalsestraat North	0.15	2.43	0.88
9	**40**	Laaressingel	-0.11	2.43	0.92
10	**45**	Lasondersingel	0.20	2.43	0.84
10	**45**	Oldenzaalsestraat North	0.00	2.43	1.00
10	**45**	Laaressingel	0.05	2.43	0.96
11	**50**	Lasondersingel	0.40	2.43	0.70
11	**50**	Oldenzaalsestraat North	0.20	2.43	0.84
11	**50**	Laaressingel	0.19	2.43	0.85
12	**55**	Lasondersingel	0.39	2.43	0.70
12	**55**	Oldenzaalsestraat North	0.53	2.43	0.60
12	**55**	Laaressingel	0.12	2.43	0.91

Table 47: Results of the one sample t-test indicating that the model calibration is successful

Table 47 shows that for all junction approaches and over all time intervals the absolute one sample t-test value is smaller than the threshold value. This means that the t-test cannot prove that the simulated maximum queue lengths differ significantly from the observed queue lengths which means that the model calibration is successful.

G.2.2. *Validation of the SGFC-junction model*

The SGFC-junction simulation model is validated with data from an off-peak observation period. The observation location is still the same. For the Lasondersingel and Laaressingel approaches the one sample t-test is used again to check if the simulation results represent the observations accurately enough. For the Oldenzaalsestraat North approach the K-S test is used because not all the queue length data was normally distributed on this arm.

One sample t-test for the SGFC junction model validation for the Lasondersingel					
Time interval	**Interval start**	**Junction arm**	**One sample t-**	**One sample t-**	**P-value**
1	0	Lasondersingel	-0.77	2.43	0.45
1	0	Laaressingel	-0.75	2.43	0.46
2	5	Lasondersingel	-0.27	2.43	0.79
2	5	Laaressingel	-0.14	2.43	0.89
3	10	Lasondersingel	0.01	2.43	1.00
3	10	Laaressingel	-0.01	2.43	1.00
4	15	Lasondersingel	0.17	2.43	0.87
4	15	Laaressingel	0.01	2.43	0.99
5	20	Lasondersingel	0.09	2.43	0.93
5	20	Laaressingel	0.01	2.43	0.99
6	25	Lasondersingel	0.55	2.43	0.59
6	25	Laaressingel	0.42	2.43	0.68
7	30	Lasondersingel	-0.03	2.43	0.97
7	30	Laaressingel	0.17	2.43	0.87
8	35	Lasondersingel	-0.46	2.43	0.65
8	35	Laaressingel	-0.47	2.43	0.65
9	40	Lasondersingel	-0.96	2.43	0.35
9	40	Laaressingel	-1.23	2.43	0.23
10	45	Lasondersingel	-0.37	2.43	0.71
10	45	Laaressingel	-0.27	2.43	0.79
11	50	Lasondersingel	0.32	2.43	0.75
11	50	Laaressingel	0.43	2.43	0.67
12	55	Lasondersingel	0.21	2.43	0.84
12	55	Laaressingel	0.54	2.43	0.60

Table 48: Results of the one sample t-test for model validation

Table 48 above shows that for both arms and all time intervals the absolute t-test value is smaller than the threshold value. This means that the t-test cannot prove that the simulated maximum queue lengths differ significantly from the observed queue lengths which means that the model validation for these two junction approaches is successful.

(Average) Queue lengths [#	Frequencies		Cumulative share of	Cumulative % of simulatio	K-S test statistic	Threshold value
	Cumulative	Cumulative				
10.00	2	0	0.166667	0.000000	0.166667	0.555218
11.00	3	0	0.250000	0.000000	0.250000	0.555218
12.95	3	1	0.250000	0.083333	0.166667	0.555218
13.00	6	1	0.500000	0.083333	0.416667	0.555218
13.40	6	2	0.500000	0.166667	0.333333	0.555218
13.80	6	3	0.500000	0.250000	0.250000	0.555218
14.00	7	3	0.583333	0.250000	0.333333	0.555218
14.35	7	4	0.583333	0.333333	0.250000	0.555218
16.00	7	5	0.583333	0.416667	0.166667	0.555218
17.50	7	6	0.583333	0.500000	0.083333	0.555218
18.85	7	7	0.583333	0.583333	0.000000	0.555218
19.95	7	8	0.583333	0.666667	0.083333	0.555218
20.50	7	9	0.583333	0.750000	0.166667	0.555218
21.00	8	9	0.666667	0.750000	0.083333	0.555218
21.40	8	10	0.666667	0.833333	0.166667	0.555218
22.00	10	10	0.833333	0.833333	0.000000	0.555218
23.00	11	10	0.916667	0.833333	0.083333	0.555218
23.10	11	11	0.916667	0.916667	0.000000	0.555218
25.00	12	11	1.000000	0.916667	0.083333	0.555218
26.65	12	12	1.000000	1.000000	0.000000	0.555218

Table 49: Results of the Kolmogorov-Smirnov test for model validation of the Oldenzaalsestraat

Table 49 above shows that the K-S test statistic is lower than the threshold value for all of the time intervals. This means that the K-S test cannot prove that the simulated maximum queue lengths differ significantly from the observed queue lengths which means that the model validation for the Oldenzaalsestraat North approach is successful as well.

H. Generation of traffic demand scenarios

The performance of the junction designs is evaluated over a wide range of traffic demand scenarios, because the traffic demand scenario may influence the relative performance of the junction designs. The traffic demand scenario is described with three different variables: hourly intensity, distribution of traffic over the main road and the side road and the percentage of left turning traffic. These three variables have been chosen because they all influence the amount of conflicts that occur at the junction. In turn the amount of conflicting traffic movements occurring at a junction impacts the junction performance

The probability that vehicles from conflicting traffic streams meet at the junction will increase with increasing traffic intensities. So with increasing traffic intensities the probability that vehicles have to (be forced to) take action to evade each other at the junction increases. These evasive actions will increase delays and/or congestion at the junction. Because the traffic intensity influences the probability that vehicles meet at the junction the traffic intensity is related to junction safety as well. The amount of emissions is related to the motor traffic intensity in two ways. Firstly in a direct way: more vehicles will lead to more emissions. Secondly because a higher traffic intensity will lead to more congested traffic flows which result in more emissions per vehicle.

The distribution of traffic over the main and side road influences the amount of conflicts, and thus the junction performance, as well. When almost all the traffic is handled on the main road there is little conflict at the junction because the amount of traffic from the side road is minimal.

The amount of left turning traffic has an important impact on the amount of conflicts at a junction as well. Because the left turning movement does not only conflict with traffic flows from the other road but with the oncoming traffic flows from the same road as well. Because of this large impact on the amount of conflicts at the junction the amount of left turning traffic has a large impact on junction performance as well.

The further distribution of traffic over straight trough and right turning traffic is less important. Straight through traffic only conflicts with traffic flows from the other road (apart from the opposing left turn). Its effect on the amount of conflicts is therefore already accounted for in the distribution of traffic over the main and the side road. The right turning movement conflicts with the least other movements and is therefore not explicitly included. Because this distribution is has a low impact on junction performance it will not be included as a variable in this research, instead it is set at a fixed distribution used in all simulations.

Regarding bicycle safety at junctions the amount of right turning traffic generally speaking is important because right turning motor vehicles can come into conflict with through going bicycle traffic. Especially with right turning lorries there is a risk for blind spot accidents. Because these type of collisions have severe consequences for bicyclists they should be explicitly taken into account (SWOV, 2012). For this research this bicycle safety aspect does however not play a role. At the SGFC-junctions the green time for bicycles is separated from the green time for motor vehicles and thus eliminating the risk for right hook and blind spot accidents. At roundabouts with priority for bicycles 'right hook' accidents can still occur. However they occur when a motor vehicle exits the roundabout. The location where that motor vehicle entered the roundabout, which specifies if it turns right, straight through or turns left at the junction level, is irrelevant.

Important for the feasibility of this research is that the amount of potential junction design traffic demand scenario combinations is limited. The traffic demand scenario is described with three continuous parameters with potentially large ranges meaning that there are thousands of possible traffic demand scenarios. Multiply this with a large number of potential junction designs and the number of possible combinations is already too great to research within the time constraint.

To ensure a feasible amount of simulations an estimation of the time it takes is made. This is made under the assumption that each simulation run takes 1 minute. This estimation also gives an overview of the variables and the number of levels per variable used in creating the traffic demand scenarios. As can be seen in Table 50 below the planned total number of simulations is 8000. This equals about 3,5 working weeks. The variables and the values used for the different levels are described throughout the rest of this chapter.

	Traffic Patterns						Desing type	Runs	Total
Levels	Total motor vehicle intensity [mv/h]	Total bicycle intensity [b/h]	Percentage of motor vehicles on main road [%]	Percentage of bicycle on main road [%]	Percentage of left turning traffic motor vehicles [%]	Percentage of left turning traffic bicycles [%]	Number of junction design types	Number of runs per situation	Total number of simulations
Levels per variable	5	5	2	2	2	2	2	10	*8000*
Level values	1000	200	60	60	5	5			
	1500	600	80	80	15	15			
	2000	1000							
	2500	1400							
	3000	1800							

Table 50: Overview of the variables used to describe the traffic demand scenario, the junction design types and number of runs and their corresponding number of levels resulting in the total number of required simulations

Because of the time restriction other variables that do have an impact on junction performance cannot be fully included in this research. These are the vehicle arrival distribution and the vehicle fleet distribution.

H.1. Traffic intensity levels

For junctions with low traffic volumes no special traffic control measures are needed. Therefore the range of total traffic intensity that is used for investigating the performances of the two different junction designs has a minimum value.

The maximum value for traffic intensity used in this research are based on the recommended maxima for the single lane roundabout and the SGFC-junction. For a single lane roundabout the CROW gives a theoretical capacity of 2.700 PCE/hour (CROW, 2008). This capacity is the sum of the traffic flows on all of the approach lanes of the roundabout. However the CROW states that in practice a capacity value of 2.000 PCE/hour is realistic (CROW, 2008). There is however another important capacity measure: the conflict point capacity. The conflict point intensity is calculated by summing the intensity of traffic on the roundabout at the conflict point of an approach and the traffic intensity on the approach lane. Figure 42 below illustrates the conflict point capacity: the traffic intensity at the roundabout is noted with arrow 1. and traffic intensity on the approach is noted with arrow 2. A roundabout connected with four roads thus has four points at which the conflict point intensity can be calculated. For single lane roundabouts this conflict point capacity is at 1.500 PCE/ hour (CROW, 2012).

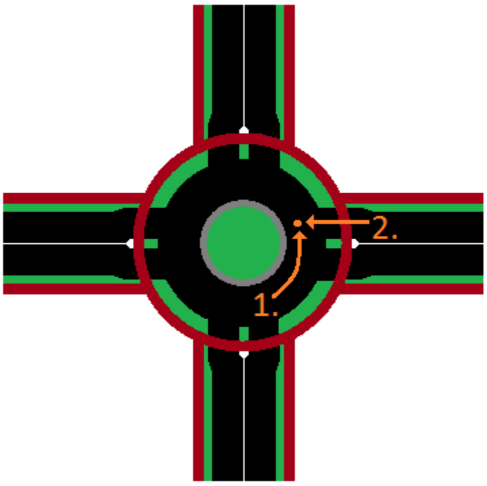

Figure 42: The conflict point intensity at a roundabout is determined by adding the intensity of traffic already on the roundabout (1) and the intensity of traffic on the approach (2)

For signalised junctions it is more difficult to find a suitable maximum hourly traffic intensity because the number of turn lanes can still vary during the research. For junctions with three lanes on each approach, one for turning left, one for through traffic and one for turning right, the theoretical capacity is 4.000 PCE/h but in practice 3.500 is more realistic (CROW, 2008). The conflict point capacity is 3.800 PCE/h (CROW, 2008). However a rule of thumb for the maximal hourly intensity states that this is 10% of the maximum intensity per day (CROW, 2008). For a junction with SGFC this would mean an peak hour capacity of $25.000 * 0.10 = 2.500$. The maximum hourly intensity that is simulated is set at 3.000 PCE/h. This value is lower than the practical maximum value of 3.500 PCE for a signalised junction with three lanes on each approach because this value is meant for junctions without SGFC and the motor vehicle capacity of a SGFC-junction is likely to be lower.

Including the previously mentioned maximum capacity the traffic intensity is distributed over five different levels. The traffic intensity is increased with 500 veh/h increments. This results in traffic intensity levels of 1.000, 1.500, 2.000, 2.500 and 3.000 vehicles/h.

The maximum capacity of bicycle lanes is very high: a cycle lane of 1,80m width can handle about 4.700 bicyclists per hour (CROW, 2006). Because a bicycle lane width of 2,00m is used in this research the capacity at the simulated junctions may be even higher, dependant on the exact signal timings. For this research the maximum total bicycle intensity is set lower because these values are seldom seen in practice. For instance for the city of Enschede only on the two busiest signalised intersections the bicycle count exceeds the value of 100 bicycles per 15 minutes (Veenstra, Thomas, & Geurs, 2013), which would result in 400 bicycles per junction approach per hour. Assuming an even distribution of bicycles on all the junction approaches this would lead to a total intensity value of 1.600 bicycles per hour for a four armed junction. The same study states that for the city of Enschede a count of 50 cyclists per 15 minutes can be considered as a typical volume for the main corridors in Enschede (Veenstra, Thomas, & Geurs, 2013). This would result in a value of 200 cyclists per approach per hour and thus in a total intensity of 800 bicycles per hour for the total junction.

The variable bicycle intensity will, just like the motor vehicle intensity, have five levels. These will be: 200, 600, 1.000, 1.400 and 1.800 bicycles/h. This range offers some values below the bicycle intensities measured on Enschede's main bicycle corridors and a value at about the same intensity as the busiest bicycle junctions in Enschede and should therefore offer a complete range of common intensities.

H.2. Distribution over major and minor road

This variable determines what percentage of the hourly traffic intensity will approach the junction on the main road. Automatically the rest of the traffic will approach the junction from the side road. The minimum value for the variable is 50%. Lower values are illogical. For example 40% of traffic on the main road, which for example runs North-South, and thus 60% of traffic on the side roads de facto makes the side road the 'main' road because it has the highest traffic volume now. Given that the junction design is adapted to the traffic demand scenario this traffic demand scenario would give the same result as the traffic demand scenario with 60% traffic on the North-South main road and therefore it is unnecessary to simulate this traffic demand scenario.

The maximum value of this variable should be below 100%. A value of 100% of traffic on the main road would mean that there is no cross traffic and thus no need for a junction. The maximum value is therefore set at 90%.

This variable will have two levels. These will be: 60% and 80% of traffic on the main road. For bicycle traffic the same levels are used.

H.3. Share of left turning traffic

Because left turning traffic has a conflict with through traffic from the opposite direction the amount of left turning traffic is an important factor for junction capacity and therefore in junction design. Another reason to look into left turning traffic, in this case specifically for the cycling mode, is that the SGFC-junction is specifically recommended for locations with a high percentage of left turning cycle traffic.

Theoretically the minimum value for this variable is 0%. In practice it is unlikely that this theoretical minimum occurs but values close to 0% are realistic. The theoretical maximum value is 100%. It is however unlikely that this value will occur in practice. The variable of percentage of left turning traffic will have two levels. These are 5% and 15%. It is recommended that SGFC-junctions should be applied at locations where at least 10% of bicycle traffic makes a left turn, see section A.2.1. By choosing values both above and below this recommended amount it is possible to find out if this recommendation is a valid argument in choosing between a roundabout and a SGFC-junction.

I. Selection of results to present

Because of the great number of simulated traffic demand scenarios it is not feasible to present all results in this report. The first paragraph explains how the graphs used to present the results are setup and how many graphs would be necessary to show all results in this report. The second paragraph explains the reasoning behind reducing the number of graphs presented to five per performance indicator as presented in chapter 4.

To ensure a good readability of the results the number of lines is limited to four lines per graph. Within this limitation each graph can show several effects of the traffic demand scenario and junction design type:

- the effect of one traffic demand variable by displaying the levels of this variable on the x-axis
- the effect of the junction type by displaying different lines for each junction type
- the effect of a second traffic demand variable by displaying two lines per junction type

Given the previous graph layout limitations 15 different types of graph can be made to show the effect of a traffic demand scenario on a certain junction performance indicator, see Table 51 below. Each of the columns in Table 51 represents one of the six traffic demand variables that can be displayed on the x-axis of a graph. Each of the three traffic demand variables is defined separately for motor vehicles and bicycles, therefore there are six variables. The rows represent the variable that can be displayed by using two different lines per junction type. Each cell in Table 51 represents a potential graph. It is not sensible to create a graph with the same variable displayed on the x-axis and in the different lines therefore the cells in the diagonal from top left to bottom right are filled with an 'x'. Graphs represented by each cell above and/or to the right of this diagonal are mirrored from one of the graphs represented by cells below and/or left of this diagonal. Because it is illogical to create both of these mirrored graphs the top right half of the cells is filled with an 'x' as well. This results in 15 open cells and thus potentially 15 graphs per junction performance indicator.

Variable displayed with different lines		Variable on the x-axis					
		intensity		main-side road distrib.		left turning share	
		motor vehicle	bicycle	motor vehicle	bicycle	motor vehicle	bicycle
intensity	motor vehicle	x	x	x	x	x	x
	bicycle	1	x	x	x	x	x
main-side road distribution	motor vehicle	2	3	x	x	x	x
	bicycle	4	5	6	x	x	x
left turning share	motor vehicle	7	8	9	10	x	x
	bicycle	11	12	13	14	15	x

Table 51: The possible number of graphs per performance indicator to demonstrate the effect of a certain traffic demand

Because each graph can only show the effects of two traffic demand variables the four others have to remain constant. The level at which they remain constant is chosen as follows:

- the middle level for the traffic intensities (where there are 5 levels)
- the level that generates the most conflicts for the main-/side road distribution and left turning share (where there are 2 levels).

For example the graph with motor vehicle intensity on the x-axis and bicycle intensity represented in the graph lines, option number 1 in Table 51 above, the variables that are not included are:
-main-/ side road distribution for motor vehicles, thus put at 60%
-main-/ side road distribution for bicycles, thus put at 60%
-left turning share for motor vehicles, thus put at 15%
-left turning share for bicycles, thus put at 15%

I.1. Selection of graph types to present

Presenting 15 graphs per junction performance indicator would reduce the readability of the report too much. This paragraph explains how this number is reduced to 5 graphs per performance indicator as presented in chapter 4.

Each of the two traffic intensity variables is preferably put on the x-axis of a graph because that way each of their five levels are clearly included in the graph. Each of the main- / side road distribution and left turn share variables has only two levels and can therefore easily be displayed with different lines. Displaying these variables on the x-axis is not efficient because they only have two levels. This reduces the potential number of graphs per junction performance indicator to nine, see Table 52 below.

The disadvantage of only displaying traffic intensities on the x-axis is that the combined effects of a main- / side road distribution and left turn share are not presented. This is acceptable because in general the effect of these two variables is small compared to the effects of vehicle intensities. Furthermore the individual effects of each of these variables is still presented in the remaining graphs for instance in graph numbers 1, 2, 3, 4, 5, 7, 8, 11, 12. The red cells in Table 52 below show which graphs will no longer be presented.

Variable displayed with different lines		Variable on the x-axis					
		intensity		main-side road distrib.		left turning share	
		motor vehicle	bicycle	motor vehicle	bicycle	motor vehicle	bicycle
intensity	motor vehicle	x	x	x	x	x	x
	bicycle	1	x	x	x	x	x
main-side road distrib.	motor vehicle	2	3	x	x	x	x
	bicycle	4	5	6	x	x	x
left turning share	motor vehicle	7	8	9	10	x	x
	bicycle	11	12	13	14	15	x

Table 52: The potential number of graphs per junction performance indicator to demonstrate the effect of a certain traffic demand scenario after limiting traffic demand variables with only two levels to only be displayed in the lines

Analysis of several graphs presenting travel times reveals that some traffic demand variables hardly influence the travel times. Given that the effect on travel time is negligible it is likely that these traffic demand variables also hardly influence the emission and safety performance of the junctions.

Firstly this is the case for the bicycle main- / side road distribution. This is illustrated with a type 5 graph in Figure 43 below. Both of the roundabout lines and both of the SGFC-junction lines are very similar to each other which means that the variable main- / side road distribution for bicycles hardly affects junction performance. Therefore no type 4 or type 5 graphs are presented in paragraph 4.2 about safety and paragraph 4.3 about emissions.

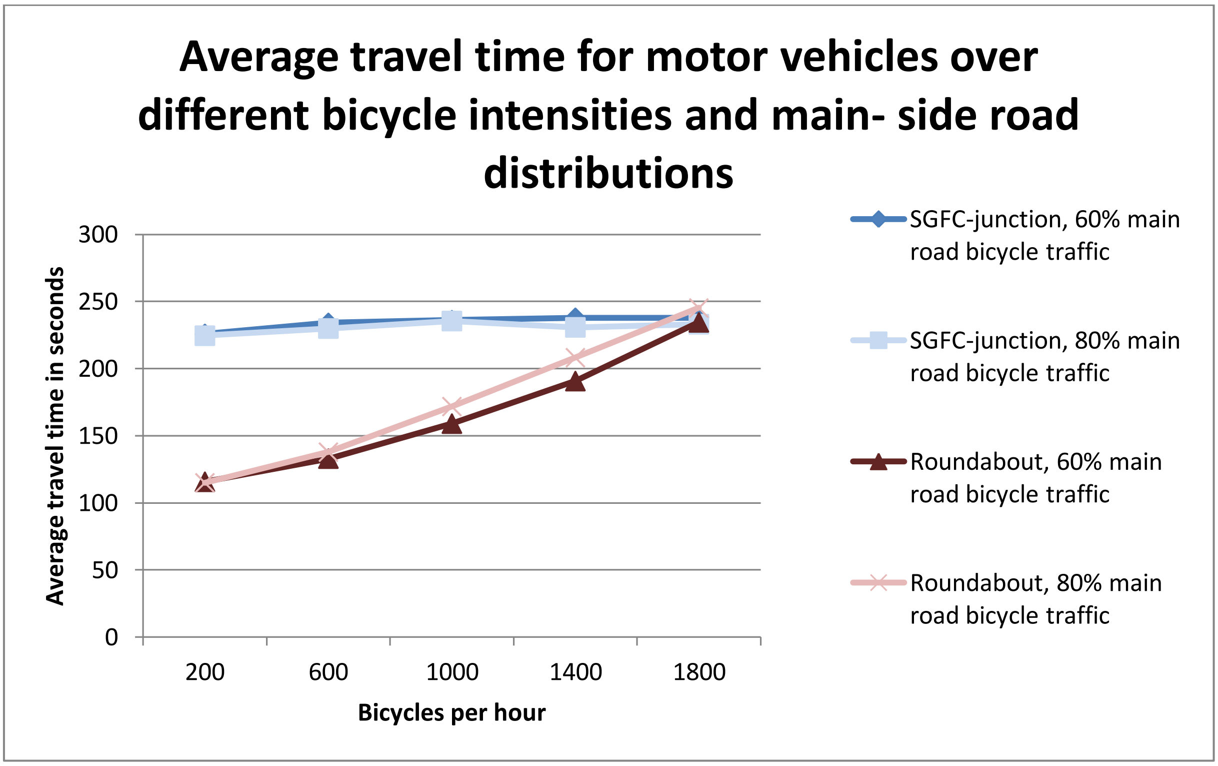

Figure 43: Average travel time for motor vehicles over different bicycle intensities and bicycle main- / side road distributions

For the SGFC-junction this is explained by the set-up of the traffic light control logic. A bicycle green phase is implemented if a bicycle is detected regardless of the junction approach this bicycle is on. The distribution of bicycles over the arms therefore does not affect the occurrence of a bicycle green phase and thus has no impact on how long motor vehicle traffic is stopped.

At the roundabout there is a slight impact of bicycle main- / side road distribution on motor vehicle travel time. In contrast to the expectations increasing the bicycle main road traffic fraction to 80% slightly increases the travel time for motor vehicles.

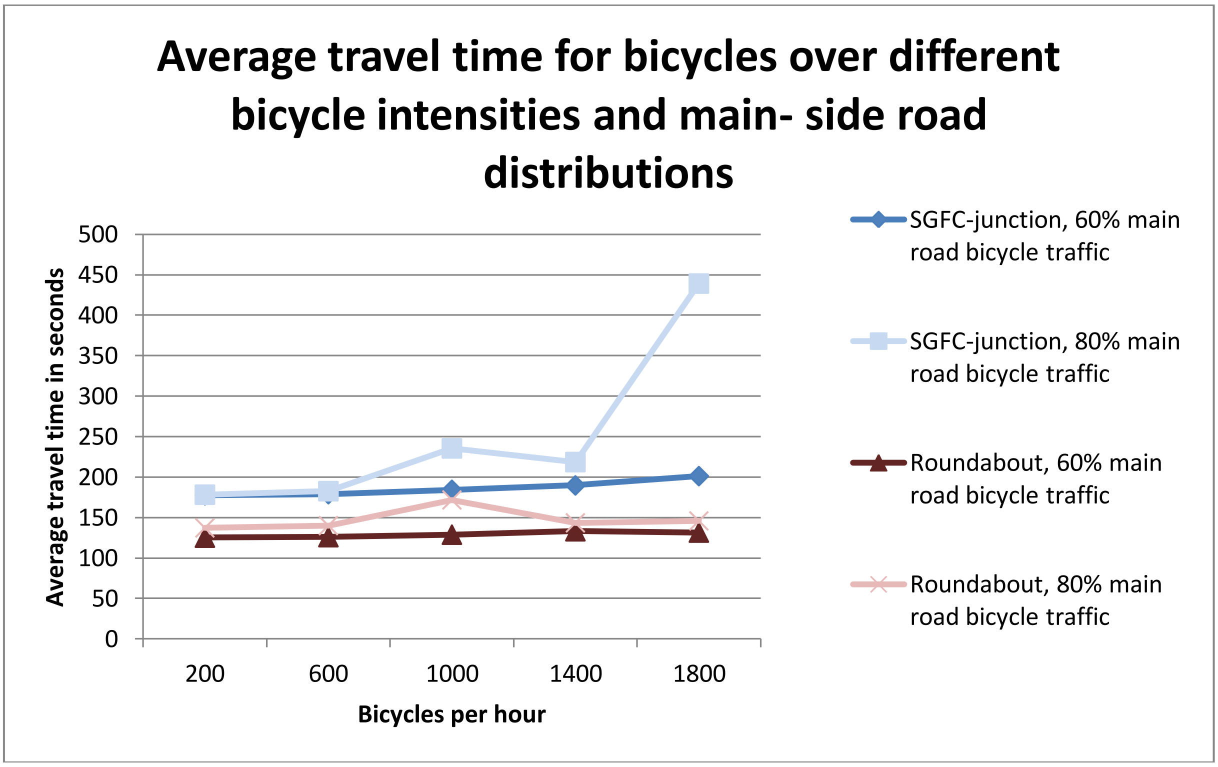

Figure 44: Average travel time for bicycles over different bicycle intensities and bicycle main and side road distributions

The second traffic demand variable that turns out to have little effect on junction performance is the left turning share of bicycles. This is illustrated with the two type 12 graphs in Figure 45 and Figure 46 below. In each of the two graphs both of the roundabout lines and both of the SGFC-junction lines are very similar to each other. This means that the left turn share of bicycles hardly influences the travel time for motor vehicles and or for bicycles. Therefore no type 11 or type 12 graphs are presented.

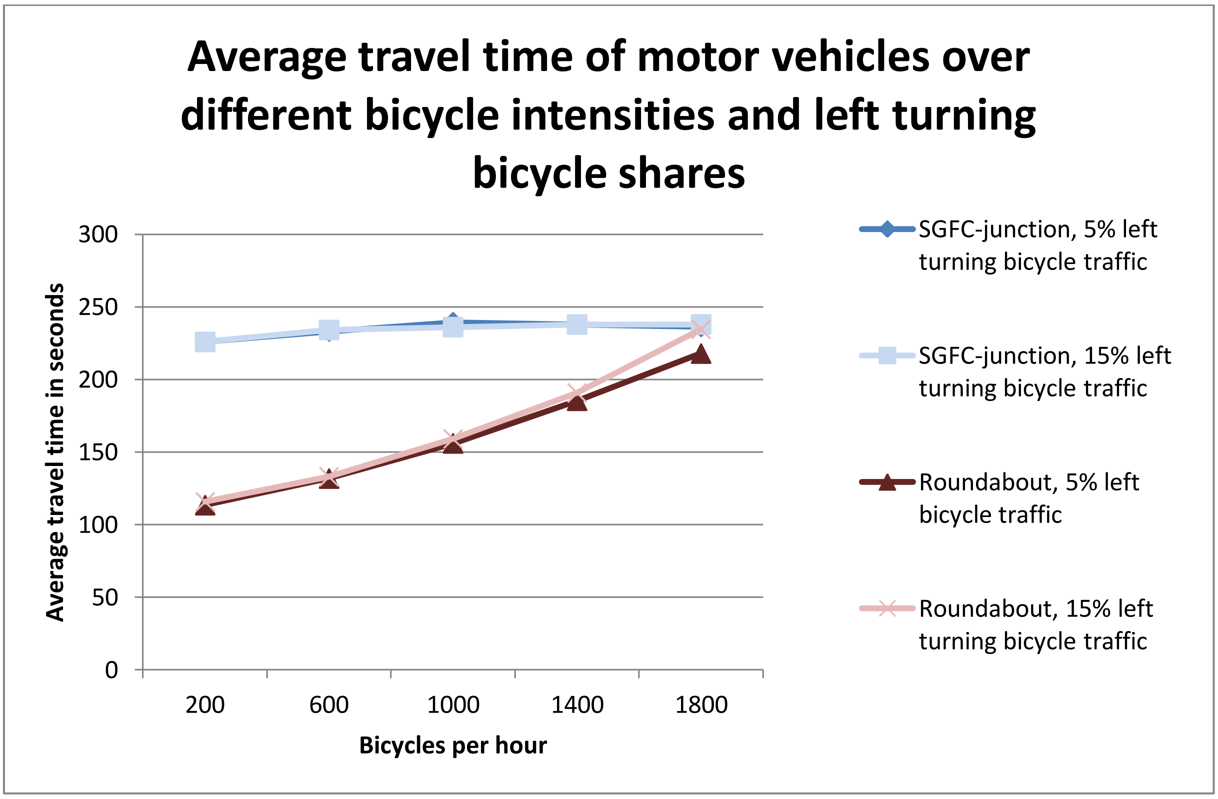

Figure 45: Average travel time of motor vehicles over different bicycle intensities and left turning bicycle shares

The left turn share of bicycles does not impact on motor vehicle travel time at the SGFC-junction because all bicycle movements happen within the same green phase. A redistribution of bicycle movements across the junction does not impact the length of this green phase and therefore does not impact motor vehicle average travel time.

Bicycle left turn share does have a slight impact on motor vehicle travel time at the roundabout. Because left turning bicycles have to cross, and thus potentially hinder motor vehicles, at two roundabout arms compared to hinder at only one arm for straight through going bicycles.

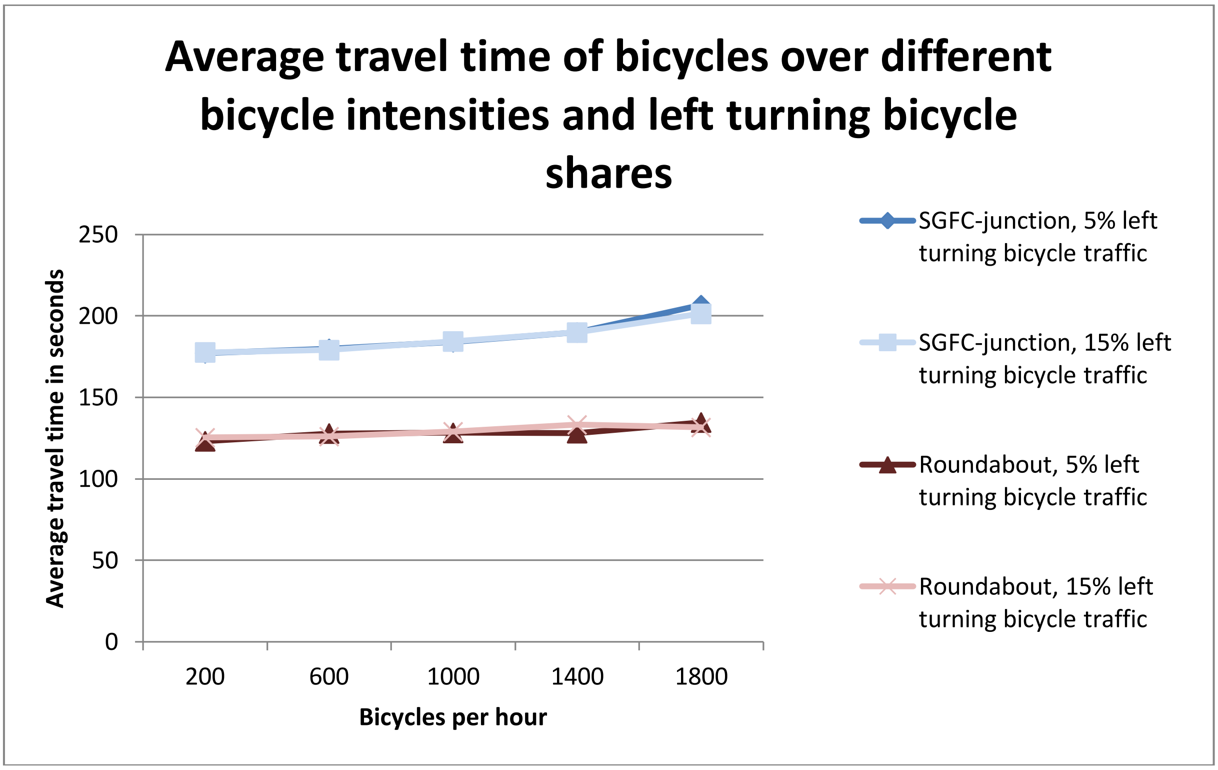

Figure 46: Average travel time of bicycles over different bicycle intensities and left turning bicycle shares

The left turn share of bicycles hardly impacts the travel time for bicycles at the SGFC-junction for two reasons. Firstly the left turn share does not impact how often a bicycle green phase is implemented and therefore the time between bicycle green phases remains constant. During the bicycle green phase the left turning bicycles rarely come into conflict with bicycles from the opposite junction arm, unless bicycle intensities get very high, because of the different distance to the conflict point.

By excluding the graphs with the bicycle main- / side road distribution and the left turn bicycle share five different graph types remain, see the overview in Table 53 below.

Variable displayed with different lines		Variable on the x-axis					
		intensity		main-side road distrib.		left turning share	
		motor vehicle	bicycle	motor vehicle	bicycle	motor vehicle	bicycle
intensity	motor vehicle	x	x	x	x	x	x
	bicycle	1	x	x	x	x	x
main-side road distrib.	motor vehicle	2	3	x	x	x	x
	bicycle	4	5	6	x	x	x
left turning share	motor vehicle	7	8	9	10	x	x
	bicycle	11	12	13	14	15	x

Table 53: Remaining number of graphs that is displayed per junction performance indicator after excluding traffic demand variables with only two levels from the x-axis and removing graphs with variables that have shown to have little or no impact

I.2. Selection of the amount of graphs per graph types

In the previous paragraph the number of graph types to be presented in chapter 4 has been reduced to five different types. Per graph type still eight different graphs have to be presented:

- motor vehicle travel time
- bicycle travel time
- NO_x emissions

- PM_{10} emissions
- total carbon emissions
- motor vehicle only conflicts
- bicycle only conflicts
- motor vehicle bicycle conflicts

This means a total of $8 \times 5 = 40$ different graphs. To improve the readability of the report it is necessary to further reduce this number. This paragraph describes the reasoning behind reducing the presented number of graphs per graph type.

The number of graphs in regarding junction safety can best be reduced by omitting the motor vehicle only conflicts and the motor vehicle bicycle conflicts. Figure 47 below shows that the average number of motor vehicle conflicts at the SGFC-junction is almost zero. For other traffic demand scenarios the number of motor vehicle only conflicts for the SGFC-junction are almost zero as well. This means that the SGFC-junction always has the best safety score and therefore it is not interesting to show graphs for this indicator for other traffic demand scenarios. For motor vehicle bicycle conflicts the SGFC-junction always scores zeros by definition, as illustrated in Figure 48 below. Therefore graphs for this indicator are not presented in the results chapter. Regarding safety only the graph about bicycle only conflicts is in this report.

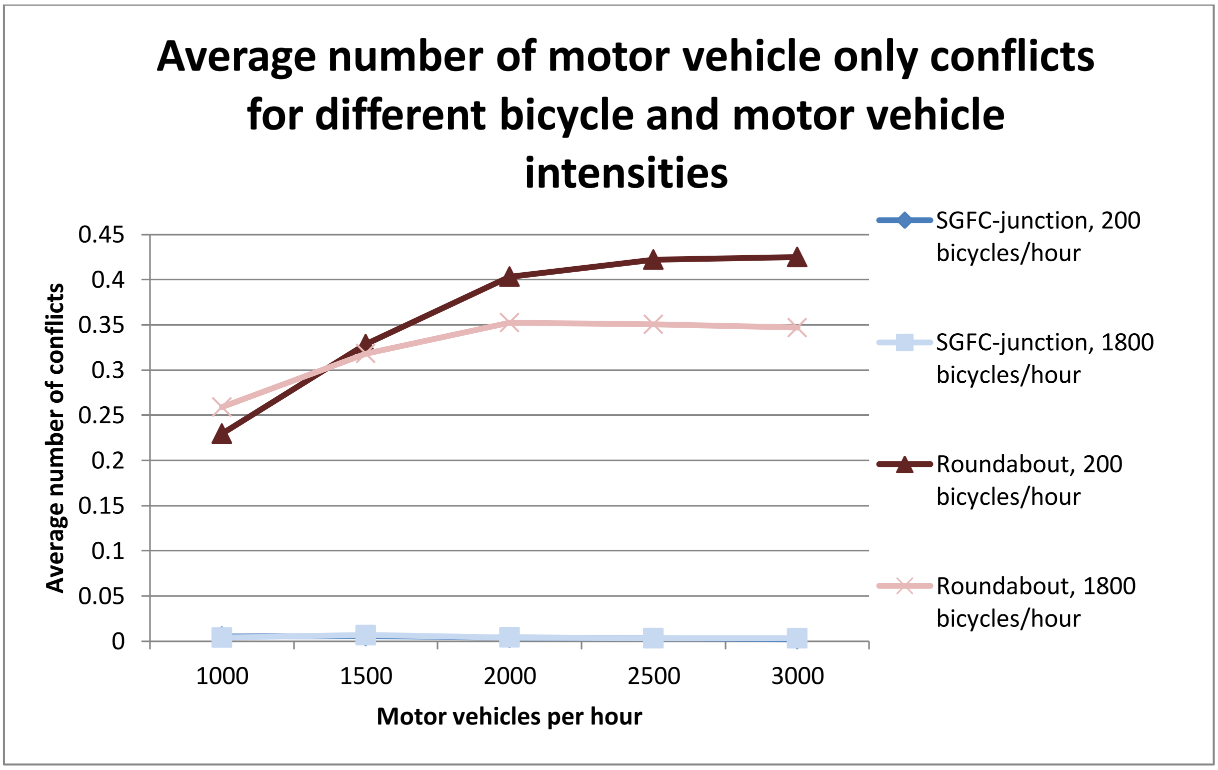

Figure 47: Average number of motor vehicle only conflicts for different bicycle and motor vehicle intensities

Figure 47 above shows that the SGFC-junction always has a lower average motor vehicle only conflict score than the roundabout. The Mann-Whitney u test indicates that over all motor vehicle intensity levels these differences are statistically significant. Both SGFC-junction lines follow a similar path meaning that the bicycle intensity does not impact the amount of motor vehicle only conflicts. Since the bicycle intensity hardly influences the amount of bicycle green phases implemented it also does not influence the amount of braking actions, which can lead to conflicts, drivers on the motor vehicle approach have to make.

Both of the roundabout graphs follow an increasing curve that levels out at high motor vehicle intensities. With an increasing motor vehicle intensity the probability that a vehicle on the approach has to give way to a vehicle on the circulating lane, and thus the probability that they come into conflict with each other increases. However as the motor vehicle intensity increases congestion on the approaches and on the circulating lane also increase. Because of this motor vehicles willing to enter the circulating lane arrive at the roundabout at a lower speed which reduces the probability of a conflict occurring. These two effects combined result in the presented increasing lines that level out at higher motor vehicle intensities. Figure 9 in chapter **Fout! Verwijzingsbron niet gevonden.** shows that a higher bicycle intensity leads to a higher average travel time and thus a higher amount of congestion for motor vehicles. Therefore the line for the traffic demand scenarios with 1.800 bicycles per hour levels out at a lower average conflicts per motor vehicle level.

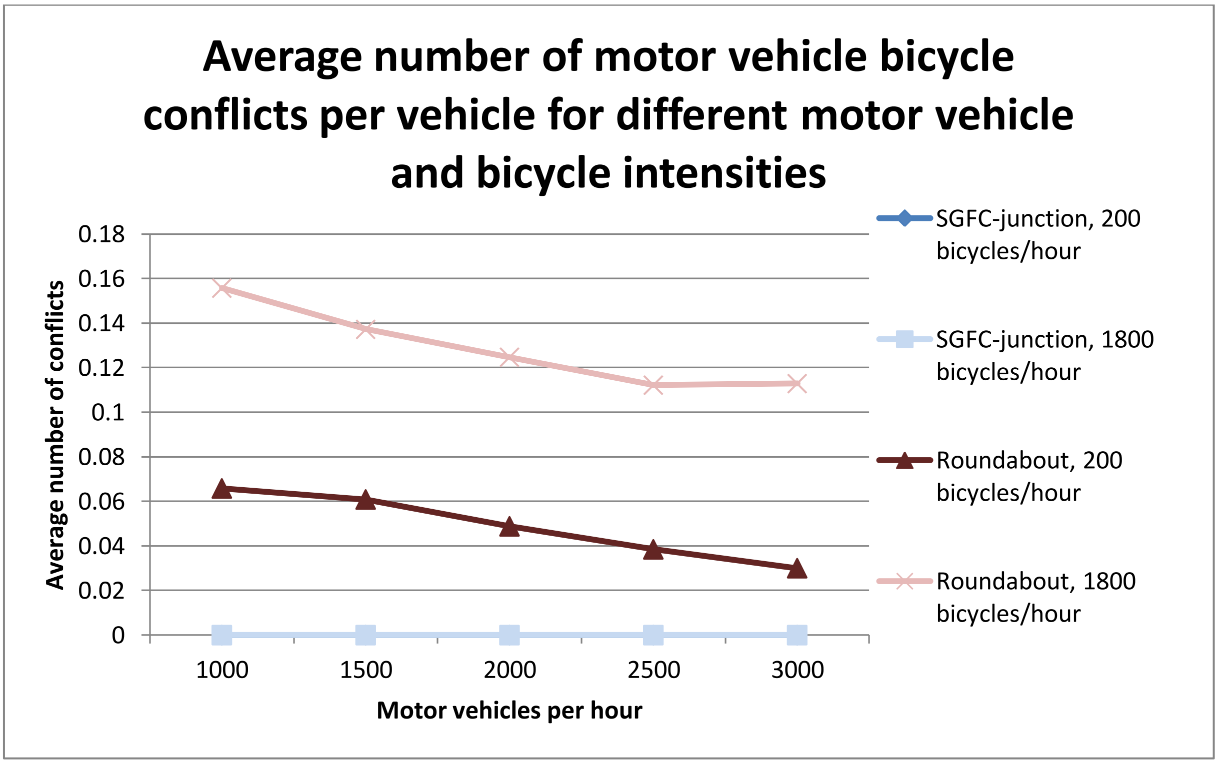

Figure 48: Average number of motor vehicle bicycle conflicts per vehicle over different motor vehicle and bicycle intensities

Figure 48 shows that the SGFC-junction always has an average of zero motor vehicle bicycle conflicts. This is the result of the junction design. The traffic light control logic completely separates motor vehicles and bicycles in time and because in the simulations all drivers adhere to the red light motor vehicles and bicycles simply cannot come into conflict with each other. For all motor vehicle intensities and for both bicycle intensity levels the roundabout score significantly higher average motor vehicle bicycle conflict numbers.

The fact that both roundabout lines show a decreasing trend contradicts the theory that as the number of motor vehicles increases the probability that a motor vehicle meet and thus the probability that they get into conflict increases. This can however be explained by the forming of congestion. As the motor vehicle intensity increases queues on the roundabout approaches increase. This decreases the speed at which motor vehicles approach the crossing with the bicycle path and thus reduce the probability of a conflict occurring. On the other hand an increase in the bicycle intensity does not lead to congestion forming on the circulatory bicycle path. Therefore an increase in the number of bicycles does lead to a higher average conflict rate as is illustrated by the two roundabout lines. To conclude for the safety performance indicator only the bicycle only conflicts are shown.

Regarding emissions three different indicators are available. Because the graphs for NO_x, PM_{10} and total carbon all follow the same pattern, as illustrated in Figure 49, Figure 50 and Figure 51 below, only the graphs about total carbon are presented in this report.

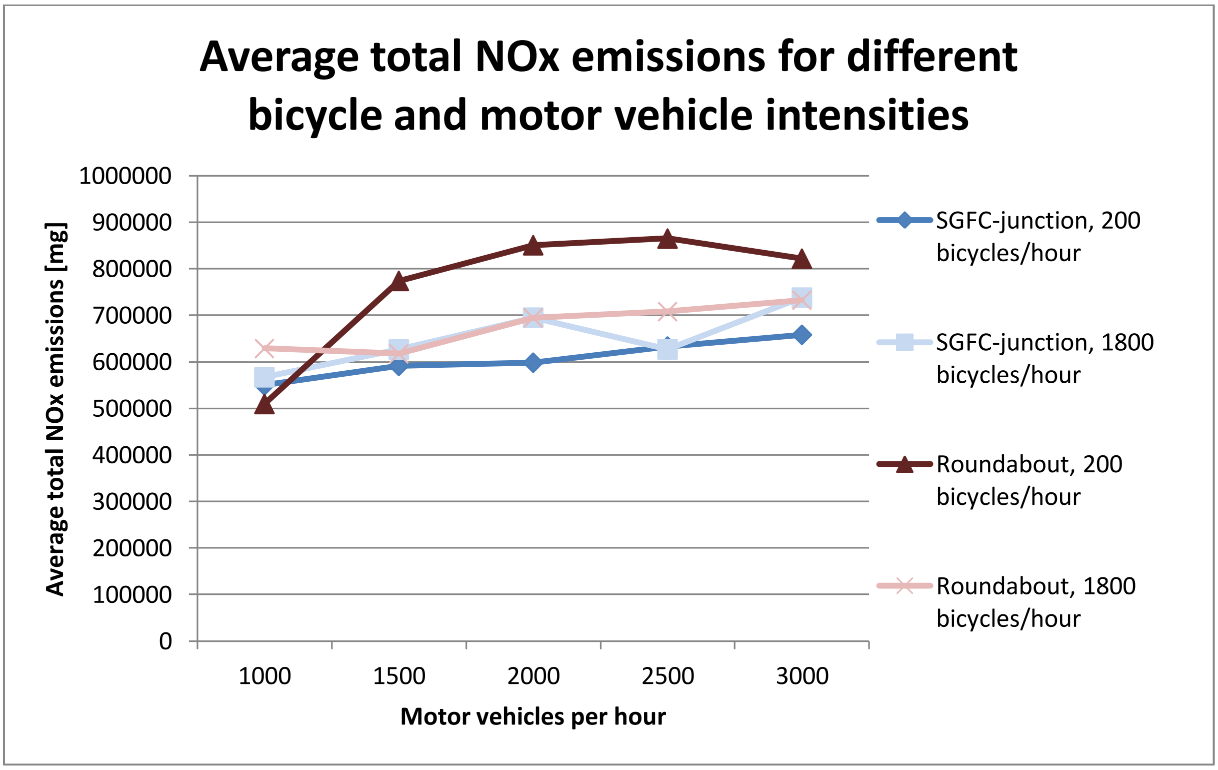

Figure 49: Average total NOx emissions for different bicycle and motor vehicle intensities

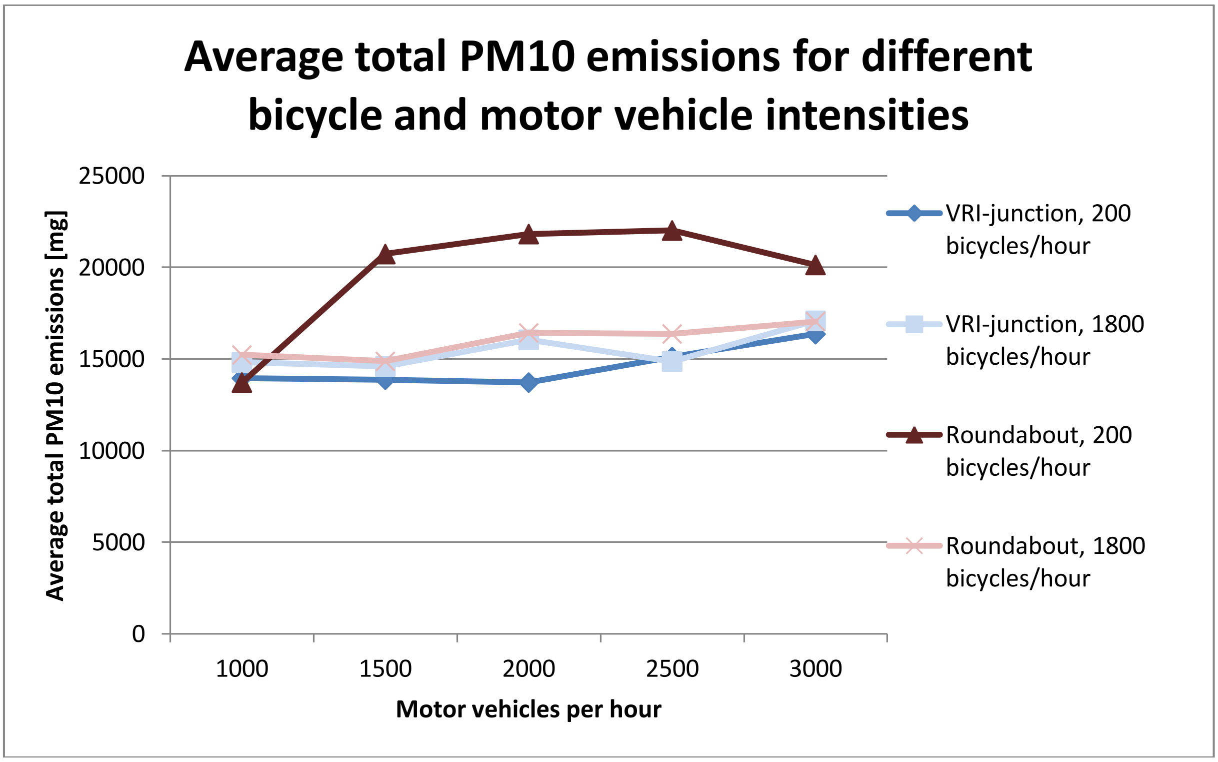

Figure 50: Average total PM10 emissions for different bicycle and motor vehicle intensities

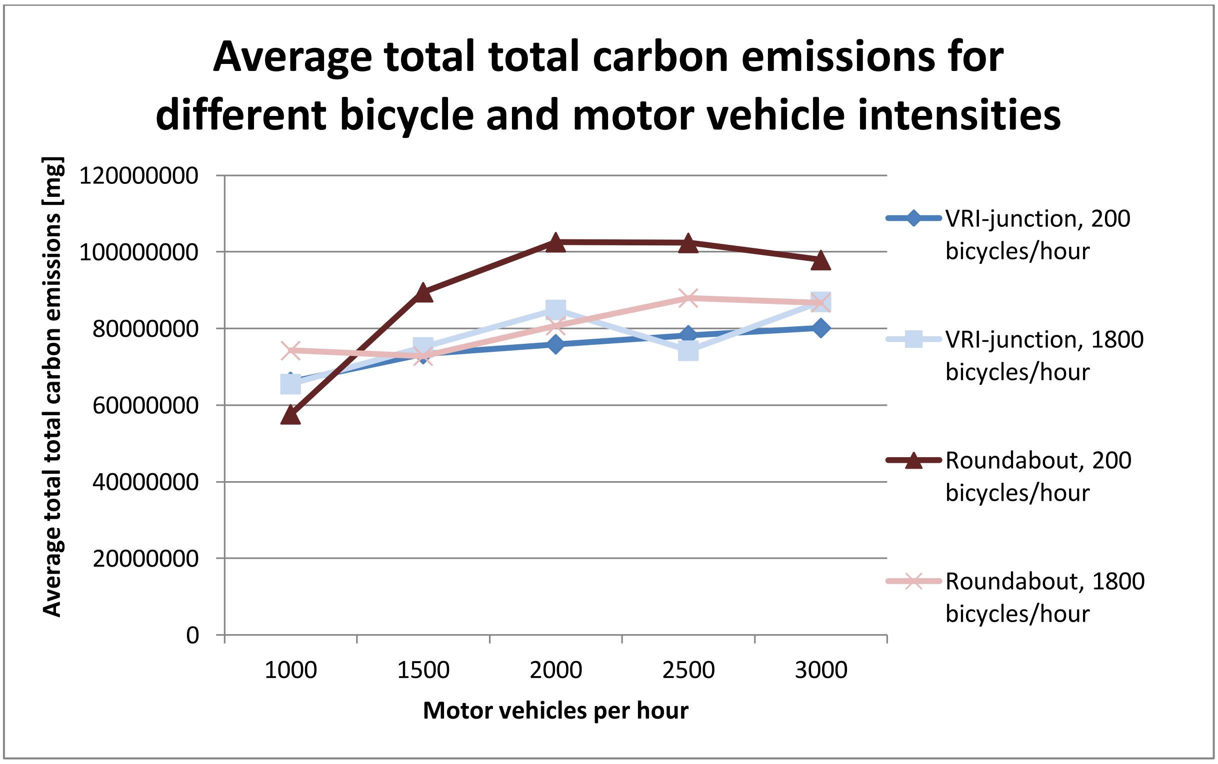

Figure 51: Average total total carbon emissions for different bicycle and motor vehicle intensities

Given the arguments presented in this chapter only graph types 1, 2, 3, 7 and 8 are presented in the report. Per graph type one graph for each junction performance indicator, motor vehicle travel time, bicycle travel time, bicycle only conflicts and total carbon emissions, is presented. This results in a total of $5 \times 4 = 20$ graphs presented in chapter 4.

J. Used Matlab script for comparing simulated junction performances

The general format of the Matlab script used in the analyses of the junction performances is shown below.

```
This file compares the total travel time of the VRI and the Roundabout for a given
%traffic demand scenario.
clear all,clc
%tic
% ---> Provide requested input below --------------------------------------
%Enter the complete path (includig filename) of the first run of the
%traffic demand scenario that you want to analyse
rotpatternfolderpath = 'I:\TTC results SWOV batch2\MV and bike intensities_20-18-60-60-15-
15_1.5_Conflicten_Run_1.xls';
VRIpatternfolderpath = 'H:\TTC resultaten VRI\VRI files nieuwe lichting 16-09-
2015\VRI_H3_S1_20-18-60-60-15-15_1.5_Conflicten_Run_1.xls';
trafficpatternname = {'20-18-60-60-15-15'};

%Define where the output has to be written
startcelrow = 316; %<--add 35 for each added traffic pattern 1 36 71 106 141 176 211 246 281
316

outputfilename = '20_2-18_60_60_5-15_15.xlsx';

%Define amount of runs and significance level
no_of_runs = 10;
siglev = 0.05;

% ---> End of input section -----------------------------------------------


%- - - 1) Loading inputdata - - -

%{
First step is to load the simulation outputs and prepare them for the
further calculations. In this case we want to specify the total time spent
(measure of junction capacity) in three ways: total, motor vehicles and
bicycles. The function prepcapdata is used for this.
%}

%Load the roundabout and VRI files
[rotttcconv,rotttcangl,rotttcfron,rotttchtli] = ttcreadrot(rotpatternfolderpath);
[VRIttcconv,VRIttcangl,VRIttcfron,VRIttchtli] = ttcreadvri(VRIpatternfolderpath);

%Calculate the total time spent specified into: motor vehicles, bicycles
%and total. To be able to separate into these classes it is necessary to
%load the files that contain all: motor vehicle links, bicycle links, motor
%vehicle only nodes, bicycle only nodes and mixed nodes.
rotMVLinksfilename = 'MVLinksRot.txt';
rotBikeLinksfilename = 'BikeLinksRot.txt';
VRIMVLinksfilename = 'MVLinksVRI.txt';
VRIBikeLinksfilename = 'BikeLinksVRI.txt';

rotMVonlynodefilename = 'MVnodeRot.txt';
rotBonlynodefilename = 'BikenodeRot.txt';
rotMIXnodefilename = 'MixnodesRot.txt';
vriMVonlynodefilename = 'MVnodeVRI.txt';
vriBonlynodefilename = 'BikenodeVRI.txt';
```

```
%vriMIXnodefilename =

formattinglinks = '%q';
formattingnodes = '%q';

%Load rotMVLinks
fileid = fopen(rotMVLinksfilename);
rotMVlinks = textscan(fileid,formattinglinks,'Headerlines',1);
fclose(fileid);

%Load rotBikeLinks
fileid = fopen(rotBikeLinksfilename);
rotBikelinks = textscan(fileid,formattinglinks,'Headerlines',1);
fclose(fileid);

%Load VRIMVLinks
fileid = fopen(VRIMVLinksfilename);
VRIMVlinks = textscan(fileid,formattinglinks,'Headerlines',1);
fclose(fileid);

%Load VRIBikeLinks
fileid = fopen(VRIBikeLinksfilename);
VRIBikelinks = textscan(fileid,formattinglinks,'Headerlines',1);
fclose(fileid);

%Load rotMVnodes
fileid = fopen(rotMVonlynodefilename);
rotMVnodes = textscan(fileid,formattingnodes,'Headerlines',1);
fclose(fileid);

%Load rotBikenodes
fileid = fopen(rotBonlynodefilename);
rotBikenodes = textscan(fileid,formattingnodes,'Headerlines',1);
fclose(fileid);

%Load rotMIXnodes
fileid = fopen(rotMIXnodefilename);
rotMIXnodes = textscan(fileid,formattingnodes,'Headerlines',1);
fclose(fileid);

%Load VRIMVnodes
fileid = fopen(vriMVonlynodefilename);
vriMVnodes = textscan(fileid,formattingnodes,'Headerlines',1);
fclose(fileid);

%Load VRIBikenodes
fileid = fopen(vriBonlynodefilename);
vriBnodes = textscan(fileid,formattingnodes,'Headerlines',1);
fclose(fileid);
```

Processing the previously loaded TTC data

```
%Process the raw ttc data
[ MVconflictsrot,Bikeconflictsrot,mixconflictsrot ] =
ttcprocessrotg(rotttcconv,rotttcangl,rotttcfron,rotttchtli,rotMVlinks,rotBikelinks,rotMVnodes,
rotBikenodes,rotMIXnodes,no_of_runs);
```

```
[ MVconflictsVRI,BikeconflictsVRI,mixconflictsVRI ] =
ttcprocessvrig(VRIttcconv,VRIttcangl,VRIttcfron,VRIttchtli,
VRIMVlinks,VRIBikelinks,vriMVnodes, vriBnodes, no_of_runs);
```

Comparing the TTC data for the roundabout and the VRI junction

```
% -----------------------------------------------------------------------

%Now that the data has been processed it is time for the comparison
%betweeen the two designs.

%In order to determine which statistical test is suitable to compare the
%two designs the first thing to check for is if the data is normally
%distributed.

%In order to be able to detect errors in this code or in the processing we
%nog predefine the normality indicators
rotMVnormality = 8;
rotBikenormality = 8;
rotsumnormality = 8;
VRIMVnormality = 8;
VRIBikenormality = 8;
VRIsumnormality = 8;

%Check for normality of roundabout ttc's
[rotMVnormality, rotMVnormalityp] = shapirowilknormality(MVconflictsrot,siglev);
[rotBikenormality, rotBikenormalityp] = shapirowilknormality(Bikeconflictsrot,siglev);
[rotsumnormality, rotsumnormalityp] = shapirowilknormality(mixconflictsrot,siglev);

%Check for normality of VRI ttc's
[VRIMVnormality, VRIMVnormalityp] = shapirowilknormality(MVconflictsVRI,siglev);
[VRIBikenormality, VRIBikenormalityp] = shapirowilknormality(BikeconflictsVRI,siglev);
[VRIsumnormality, VRIsumnormalityp] = shapirowilknormality(mixconflictsVRI,siglev);

%Check if all six datasets have been tested for normality. If not report
%error.
if (rotMVnormality + rotBikenormality + rotsumnormality + VRIMVnormality + VRIBikenormality +
VRIsumnormality) >= 8
    disp('Error: for at least one of the six input arrays the normality test has failed')
end

%In order to detect possible errors in this program we now predefine
%several variables.
MVdifferentttest = 9;
Bdifferentttest = 9;
Sumdifferentttest = 9;
MVdifferentMWtest = 9;
BdifferentMWtest = 9;
SumdifferentMWtest = 9;

%If all the six datasets are normal their respective normality variables
%should all be one. In this case the two sample t test can be performed
if (rotMVnormality + rotBikenormality + rotsumnormality + VRIMVnormality + VRIBikenormality +
VRIsumnormality) == 6
    %Now to determine which type of two sample t-test has to be done. To
    %destinguish between which test has to be applied we need to know if
```

```
    %the variances of the two samples are similar or not. We use the F-test
    %for this.

    %In case the variances are equal the test value is '0', if the
    %variances are different the null hypothesis is rejected and the test
    %result is '1'
    MVvar = vartest2(MVconflictsrot,MVconflictsVRI);
    Bvar  = vartest2(Bikeconflictsrot,BikeconflictsVRI);
    Sumvar= vartest2(mixconflictsrot,mixconflictsVRI);

    %Run the two sample t test assuming equal variances. In case there
    %is a statistically significant difference between the two averages
    %the test will return a '1'
    if MVvar == 0
        MVdifferentttest = ttest2(MVconflictsrot,MVconflictsVRI);
    end

    if Bvar == 0
        Bdifferentttest = ttest2(Bikeconflictsrot,BikeconflictsVRI);
    end
    if Sumvar == 0
        Sumdifferentttest = ttest2(mixconflictsrot,mixconflictsVRI);
    end

    %Run the two sample t test assuming unequal variances.
    if MVvar == 1
        MVdifferentttest = ttest2(MVconflictsrot,MVconflictsVRI, 'Vartype','unequal');
    end
    if Bvar == 1
        Bdifferentttest = ttest2(Bikeconflictsrot,BikeconflictsVRI, 'Vartype','unequal');
    end
    if Sumvar == 1
        Sumdifferentttest = ttest2(mixconflictsrot,mixconflictsVRI, 'Vartype','unequal');
    end
end
```

```
Error in Shapiro Wilk test: provided input matrix is empty or contains only zeros
```

Mann Whitney u test checking for differences between roundabout and VRI TTC conflicts

```
if (rotMVnormality + rotBikenormality + rotsumnormality + VRIMVnormality + VRIBikenormality +
VRIsumnormality) < 6
    %At least one of the six data sets is not normally distributed.
    %Therefore we have to use another statistical test for checking for
    %differences: the Mann Whitney u test.
    [MVdifferentMWtest, uMV] = mannwhitneyutest(MVconflictsrot,MVconflictsVRI);
    [BdifferentMWtest, uB ] = mannwhitneyutest(Bikeconflictsrot,BikeconflictsVRI);
    [SumdifferentMWtest, uSum ] = mannwhitneyutest(mixconflictsrot,mixconflictsVRI);
end

%Check for errors in code or processing
if (MVdifferentttest + MVdifferentMWtest) > 12
    disp('Error: The two motor vehicle ttt matrices have not been statistically compared')
end
if (Bdifferentttest + BdifferentMWtest) > 12
```

```
    disp('Error: The two bicycle ttt matrices have not been statistically compared')
end
if (Sumdifferentttest + SumdifferentMWtest) > 12
    disp('Error: The two summed ttt matrices have not been statistically compared')
end

%Now time to write outputs to excel
%In order to be able to write multiple traffic patterns below eachother on
%the same excel sheet the cel from where information is written is made
%variable with the statements below.
titlerow = strcat('A',num2str(startcelrow));
patternrow = strcat('D',num2str(startcelrow));

titlerot1row = strcat('A',num2str(startcelrow + 1));
titlerot2row = strcat('A',num2str(startcelrow + 12));
titlerot3row = strcat('A',num2str(startcelrow + 23));
titlevri1row = strcat('D',num2str(startcelrow + 1));
titlevri2row = strcat('D',num2str(startcelrow + 12));
titlevri3row = strcat('D',num2str(startcelrow + 23));

rottttMVrow = strcat('A',num2str(startcelrow + 2));
rotttBrow = strcat('A',num2str(startcelrow + 13));
rottttsumrow = strcat('A',num2str(startcelrow + 24));
VRItttMVrow = strcat('D',num2str(startcelrow + 2));
VRItttBrow = strcat('D',num2str(startcelrow + 13));
VRItttsumrow = strcat('D',num2str(startcelrow + 24));

statsrotttMVrow = strcat('B',num2str(startcelrow + 2));
statsrotttBrow  = strcat('B',num2str(startcelrow + 13));
statsrottttsumrow = strcat('B',num2str(startcelrow + 24));
statsvritttMVrow = strcat('E',num2str(startcelrow + 2));
statsvritttBrow = strcat('E',num2str(startcelrow + 13));
statsvritttsumrow = strcat('E',num2str(startcelrow + 24));

%Write the six matrices with the ttt outputs
xlswrite(outputfilename,MVconflictsrot,'Blad1',rottttMVrow)
xlswrite(outputfilename,Bikeconflictsrot,'Blad1',rotttBrow)
xlswrite(outputfilename,mixconflictsrot,'Blad1',rottttsumrow)
xlswrite(outputfilename,MVconflictsVRI,'Blad1',VRItttMVrow)
xlswrite(outputfilename,BikeconflictsVRI,'Blad1',VRItttBrow)
xlswrite(outputfilename,mixconflictsVRI,'Blad1',VRItttsumrow)

%Provide titles for the excel output file
title = {'Traffic pattern'};
titlerot1 = {'Roundabout number of ttc conflicts MV only'};
titlerot2 = {'Roundabout number of ttc conflicts Bike only'};
titlerot3 = {'Roundabout number of ttc conflicts Mixed'};
titlevri1 = {'VRI number of ttc conflicts MV only'};
titlevri2 = {'VRI number of ttc conflicts Bike only'};
titlevri3 = {'VRI number of ttc conflicts Mixed'};

%Write titles into excel
xlswrite(outputfilename,title,'Blad1',titlerow)
xlswrite(outputfilename,trafficpatternname,'Blad1',patternrow)
xlswrite(outputfilename,titlerot1,'Blad1',titlerot1row)
xlswrite(outputfilename,titlerot2,'Blad1',titlerot2row)
xlswrite(outputfilename,titlerot3,'Blad1',titlerot3row)
```

```
xlswrite(outputfilename,titlevri1,'Blad1',titlevri1row)
xlswrite(outputfilename,titlevri2,'Blad1',titlevri2row)
xlswrite(outputfilename,titlevri3,'Blad1',titlevri3row)

%Calculate the statistics
%Calculate averages of the ttt outputs
statsrottttMV(1,1) = mean(MVconflictsrot);
statsrottttB(1,1) = mean(Bikeconflictsrot);
statsrottttsum(1,1) = mean(mixconflictsrot);
statsvritttMV(1,1) = mean(MVconflictsVRI);
statsvritttB(1,1) = mean(BikeconflictsVRI);
statsvritttsum(1,1) = mean(mixconflictsVRI);

%Calculate the variance
statsrottttMV(2,1) = var(MVconflictsrot);
statsrottttB(2,1) = var(Bikeconflictsrot);
statsrottttsum(2,1) = var(mixconflictsrot);
statsvritttMV(2,1) = var(MVconflictsVRI);
statsvritttB(2,1) = var(BikeconflictsVRI);
statsvritttsum(2,1) = var(mixconflictsVRI);

%Calculate the standard deviation
statsrottttMV(3,1) = std(MVconflictsrot);
statsrottttB(3,1) = std(Bikeconflictsrot);
statsrottttsum(3,1) = std(mixconflictsrot);
statsvritttMV(3,1) = std(MVconflictsVRI);
statsvritttB(3,1) = std(BikeconflictsVRI);
statsvritttsum(3,1) = std(mixconflictsVRI);

%Log the test results
statsrottttMV(4,1) = MVdifferentttest;
statsrottttMV(5,1) = MVdifferentMWtest;
statsvritttMV(4,1) = MVdifferentttest;
statsvritttMV(5,1) = MVdifferentMWtest;

statsrottttB(4,1) = Bdifferentttest;
statsrottttB(5,1) = BdifferentMWtest;
statsvritttB(4,1) = Bdifferentttest;
statsvritttB(5,1) = BdifferentMWtest;

statsrottttsum(4,1) = Sumdifferentttest;
statsrottttsum(5,1) = SumdifferentMWtest;
statsvritttsum(4,1) = Sumdifferentttest;
statsvritttsum(5,1) = SumdifferentMWtest;

%Determine the number of motor vehicles and the number of bicycles
[MVtext,remain] = strtok(trafficpatternname,'-');
Btext = strtok(remain,'-');
MVnum = str2num(MVtext{1,1}).*100;
Bnum = str2num(Btext{1,1}).*100;

%Calculate the average time spent per vehicle
statsrottttMV(6,1) = statsrottttMV(1,1)./MVnum;
statsrottttB(6,1) = statsrottttB(1,1)./Bnum;
statsrottttsum(6,1) = statsrottttsum(1,1)./(MVnum+Bnum);
statsvritttMV(6,1) = statsvritttMV(1,1)./MVnum;
statsvritttB(6,1) = statsvritttB(1,1)./Bnum;
```

```
statsvritttsum(6,1) = statsvritttsum(1,1)./(MVnum+Bnum);

%Write the statistics to excel file
xlswrite(outputfilename,statsrottttMV,'Blad1',statsrottttMVrow)
xlswrite(outputfilename,statsrottttB,'Blad1',statsrottttBrow)
xlswrite(outputfilename,statsrottttsum,'Blad1',statsrottttsumrow)
xlswrite(outputfilename,statsvritttMV,'Blad1',statsvritttMVrow)
xlswrite(outputfilename,statsvritttB,'Blad1',statsvritttBrow)
xlswrite(outputfilename,statsvritttsum,'Blad1',statsvritttsumrow)
%toc
%}
```

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