Studies on coordinated traffic control: Using variable speeds and testing demand predictions

by

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PREFACE

This Master thesis is the final part of my journey to obtain the degree of Master of Science in Civil Engineering at Delft University of Technology. Civil Engineering is a broad field of study and I feel like this is reflected in my Master Thesis. Throughout the research I have been challenged on various fronts. At times I have felt like a mathematician, at times I have felt like a software developer and most of the time I have felt like a traffic engineer. In all of these fronts I was able to extend my knowledge and do things that, a year ago, I would not have thought to be within my capabilities. Now I would like to take this opportunity to thank the many people that made this thesis possible.

Thanks are in order for the members of the committee for their critical questions and feedback. Especially Maria Salomons, who has spent a lot of time reading through my chaotic initial writings. Thanks to Vialis for the opportunity to conduct this research at their company and providing a nice working environment. Thanks in particular to the traffic engineers and software developers, who were very welcoming and always made time for questions or a chat. Special thanks to Ronald van Katwijk, whose extensive network allowed me to connect with many people. Amongst those people were Martin Barto, Martijn Machielsen and Maarten Strating, whom I have to thank for sharing their knowledge and expertise about traffic signal controllers.

Special thanks also to my family, my friends and my girlfriend who are always there for me when I need them. Studying can be stressful and can sometimes feel like a rollercoaster of ups and downs. This reflects on those nearest to you and only their love and support has been able to keep me on the tracks.

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Abstract

Increasing levels of urbanisation threaten urban mobility. Cities are tasked with accommodating an increase in traffic demand, improving their living environment and reducing their emissions. These tasks conflict when it comes to road traffic, as an increase in traffic volume will lead to more emissions and especially widening the roads in the city dedicated to motorized traffic is not a good solution as it further increases demand and reduces the quality of the living environment. With this in mind, improving the performance of the current infrastructure, in a way that contributes to a reduction in emissions and deals with demand, is desirable. An area where the performance of the current infrastructure in urban areas can be improved is on the urban arterials controlled by coordinated traffic controllers. These performance improvements are sought in a general indicator: the number of stops. Reducing the number of stops leads to less acceleration and deceleration, which amongst other things, is related to a reduction in emissions, noise, fuel consumption and crashes. A time-based indicator like delay has to be considered as well, because the duration of stops may increase by a large amount, which may lead to a different conclusion about the performance.

Reducing the number of stops and delay in the network controlled by the coordinated traffic controller can potentially be achieved by overcoming some limitations of current coordinated traffic controllers. A few of these limitations were found to be: The inability to coordinate between unequal cycle times, the inability to coordinate in two directions with locally optimal signal timing plans and the accuracy of real-time traffic demand information. This thesis investigates two ideas that can potentially overcome the aforementioned limitations. These two ideas are investigated in two separate research directions. The scope of the research is limited to undersaturated demand scenario's, as coordination (with the aim of providing a green wave) becomes a lot more difficult when oversaturation occurs.

A variable speed for more efficient coordinated signal timings

The first research direction is based on an idea that uses a variable speed in the network of the coordinated traffic controller. This speed is variable in space between coordinated intersections, per segment and per direction, and in time, per cycle. For safety reasons, the variable speed is not allowed to exceed the speed limit. With new technologies, like intelligent speed assistance (ISA), realizing this variable speed in practice may be possible in the near future. In this research, an attempt is made to use the variable speed to overcome some limitations of coordinated signal timing plans. In particular, this research direction considers the feasibility of coordination in one direction between unequal cycle times and coordination in two directions between locally optimal signal timing plans. With a variable speed, more offsets are possible that lead to coordination, such that coordination can be achieved more often and aforementioned limitations can

be overcome. The research question regarding this research direction is as follows:

> *What are the benefits, for the coordination of traffic controllers, in terms of reduced stops and delay, when optimizing signal timing plans with variable speeds compared to fixed speeds in an undersaturated demand scenario?*

A theoretical analysis regarding the optimization of signal timing plans with variable speeds showed that the variable speed does not make coordination of the starts of green in one direction under different cycle times feasible. Coordinating in one direction under different cycle times with a (time-varying) variable is speed is not expected to lead to significant benefits, in terms of stops and delay, over a fixed speed. However, for coordination in two directions, the variable speed was shown (in theory) to be able to coordinate between locally optimal signal timing plans, in cases where the starts of green of the outbound and inbound coordinated directions are (nearly) simultaneous, in a way that a fixed speed is not able to. By using the space-varying variable speed, coordination can be established in two directions, in cases where starts of green of outbound and inbound coordinated directions are (nearly) simultaneous and the segments in the network are of varying length. To test these findings in a more practical setting, a model is needed that can utilize the variable speed and include all other variables at play, to optimize the coordinated signal timings. An extended version of the MAXBAND model was found most suitable to create a coordination that uses the variable speed to optimize the signal timings. Both the MAXBAND model and theoretical analysis resulted in the finding that the relative benefits of the variable speed over the fixed speed for coordination between efficient timing plans largely depends on the differences in segment lengths and the allowed internal time difference between outbound and inbound coordinated green windows. A set of timings using a variable speed, generated by MAXBAND, as well as a set of timings based on a fixed speed, generated by MAXBAND, were tested in a simulation environment. Simulation results show that the variable speed leads to a reduction in stops of 25.5% on the whole network, compared to the fixed speed, as a consequence of fewer stops on the coordinated directions. The delay, expressed in total time spent in the network, increased by 1.2% compared to the fixed speed. The root cause of this increase in delay is the slower (variable) travel speed on the segments between the coordinated directions. On the side directions, delay decreased by 19.6%, while no significant change in the number of stops was measured. The delay decrease on the side directions is a consequence of the lower cycle time, enabled by the variable speed. Overall, using a variable speed for a coordination in two directions, where the starts of coordinated green windows are (nearly) simultaneous, has been shown to be able to benefit the performance greatly in terms of stops.

TESTING THE POTENTIAL OF DEMAND PREDICTIONS IN TOPTAC

The second research direction is aimed at improving the performance of the TopTrac coordinated traffic controller by Vialis. For this controller, potential improvements were sought in the form of demand predictions, such that the controller's control decisions change from a reactive nature to a proactive nature. Currently, the controller receives traffic flow measurements from the two preceding cycles and uses this measurement to optimize the cycle time, green times and offsets for the coming two cycles. Performance

improvements, in terms of reduced stops and delay, are expected, when the controller uses a flow measurement (prediction) of the coming two cycles to optimize for these cycles. Before any prediction models are developed and tested, it is interesting to know the maximum potential performance improvements obtainable, when given near-perfect information. The research question regarding this research direction is as follows:

For TopTrac, what are the maximum benefits, in terms of reduced stops and delay, obtainable by using near-perfect demand predictions compared to normal operations in undersaturated conditions?

Two different methodologies were laid out for testing the potential of near-perfect demand predictions in TopTrac. The first methodology based demand 'predictions' on the inputs TopTrac received in a reference simulation. These inputs were shifted forwards in time by one optimization period. The second methodology based demand 'predictions' on the true traffic volume measured in a reference simulation. These traffic volumes were directly supplied to the traffic model in TopTrac, to provide the model with the most accurate information possible. The results of the simulations show that no significantly different results were obtained in the simulations with either of the prediction methodologies, compared to the reference simulation. As this result was not as expected, the research was extended. An analysis of the model output of the TopTrac traffic model showed that, for the investigated network, the control decisions did not change significantly over time in any of the simulations. This, despite the traffic pattern changing significantly during the investigated morning rush hour. This observation can be explained as follows: the traffic model tries to find signal timings that create coordination. To achieve this, longer green times on the coordinated directions and a larger cycle time is needed. Consequently, increased green times and cycle times are already achieved in the beginning of the simulation, when demand has not yet peaked. When the demand spikes, the traffic is already controlled with sufficient green time and a large enough cycle time (which were based on the established coordination). So the model sees no reason to make a different control decision based on the spike in traffic volume. Further explanations look at the side directions, which have such little demand, that even at the largest demand, the queue can be fully cleared with just the minimum green time. So there is no reason for the model to increase the green time on these directions and it was observed that the green time stays constant across the whole simulation. Logically, when no significantly different control decisions are made, no significant difference in the final results is observed. The conclusions regarding the demand predictions are based on a single test case. It is unclear whether this test case, where the control decisions do not change significantly over time, is an isolated case, or whether this occurs on other arterials as well. This research has shown that in general, it is important to be aware about the decision making process of the controller before testing the use of demand predictions. Using demand predictions will only affect performance when control decisions fluctuate based on fluctuating demand.

CONTENTS

VARIABLE DEFINITIONS

Table 1: Definition of variables

Notation	Definition	Unit
I	The total number of Traffic Light Controllers (TLC's) in the network, $I \in \mathbb{N}_3$	[-]
i	The TLC number, numbered increasing along the outbound direction, $i \in \{0, I\}$	[-]
$\hat{i}$	The segment number of the segment between TLC i and TLC $i+1$, $\hat{i} \in \{0, I-1\}$	[-]
n_i	An integer number of cycles at TLC i, $n_i \in \mathbb{N}^+$	[-]
$d_{\hat{i}}$	Distance between TLC_i and TLC_{i+1}, the length of segment $\hat{i}$	[m]
d_{min}, d_{max}	The distance $d_{\hat{i}}$ of the shortest, longest segment in the network	[m]
C_i	Cycle time of TLC i, $C_i \in \mathbb{N}^+$	[s]
C_{min}, C_{max}	The minimum and maximum values for the cycle time C_i	[s]
v	The (fixed) speed across the network	[m/s]
$v_{\hat{i},n} (\overline{v_{\hat{i},n}})$	Outbound (inbound) variable speed on segment $\hat{i}$ at cycle n	[m/s]
v_{min}, v_{max}	The minimum, maximum speed allowed on the arterial	[m/s]
l	A factor of the maximum speed used to denote the lower bound of the variable speed, $0 < l \leq 1$	[-]
$T_{off,\hat{i},n}$ $(\overline{T_{off,\hat{i},n}})$	Outbound (inbound) offset on segment $\hat{i}$ between coordinated signal groups of TLC's i and $i+1$ at cycle n	[s]
$\Delta T_{\hat{i}}$	The maximum outbound time difference allowed by the minimum and maximum speed and distance on segment $\hat{i}$	[s]
$SG_{i,n}$ $(\overline{SG_{i,n}})$	The start green time of the outbound (inbound) coordinated direction at TLC i in cycle n	[s]
$R_{\hat{i}}$	The repetition number for the number of cycles n after which the offset values on segment $\hat{i}$ repeat, $R_{\hat{i}} \in \mathbb{N}_2$	[-]
m	An integer used to signify the multiples of $\gcd(C_i, C_{i+1})$, $m \in \{0, R_{\hat{i}} - 1\}$	[-]
$p_{\text{fixed},i}$ $(p_{\text{fixed},i+1})$	The percentage of upstream (downstream) cycles that can be coordinated with a fixed speed between C_i and C_{i+1}	[%]
$p_{\text{variable},i}$ $(p_{\text{variable},i+1})$	The percentage of upstream (downstream) cycles that can be coordinated with a variable speed, based on the maximum time difference allowed by the minimum and maximum speed limits and distance on segment $\hat{i}$ (given $C_i < C_{i+1}$)	[%]
$C_{c,\hat{i}}$	The cycle time needed at TLC i and TLC $i+1$ for coordination in two directions based on properties of segment $\hat{i}$	[s]

O_i	The (fixed) internal time difference between the start of green of the two coordinated directions at TLC i, $O_i \in \mathbb{R}$.	[s]
G	An upper limit on the fixed internal time difference O_i applicable to all intersections	[s]
$n_{\hat{i}}$	An integer used for denoting a unit fraction of the cycle time on segment $\hat{i}$, $n_{\hat{i}} \in \mathbb{N}^+$	[-]
n_{min}, n_{max}	The $n_{\hat{i}}$ value corresponding to the shortest segment $\hat{i}$ and longest segment $\hat{i}$ in the network.	[-]
$G*$	A temporary fictional limit which can be compared to G to draw conclusions	[s]
G_i	An upper limit on the fixed internal time difference O_i that can differ per intersection	[s]
$\mathbf{O}$	The vector containing all O_i, starting with O_1. $\mathbf{O}$ has dimensions $(I \times 1)$	-
δd	The vector containing all segment differences starting with $d_1 - d_2$. δd has dimensions $(I - 2 \times 1)$	-
A	The matrix used to describe coordination in two directions, relating $\mathbf{O}$ to δd. A has dimensions $(I - 2 \times I)$	-
$\mathbf{O}_p$	Any particular solution for $\mathbf{O}$	-
$\mathcal{N}(A)$	The Null-Space of A	-
u	Any vector in the Null-Space of A, $u \in \mathcal{N}(A)$	-
z	Name of the objective function of an optimization problem	-
C_q	The network cycle time needed for fulfilling the demand	[s]
r	The 'freedom' in internal offset O_i used on the normative intersection	[s]
R	The number of repetitions needed	[-]
R_i	The number of repetitions performed so far	[-]
$S_{R_i}(y_i)$	The standard deviation in the observed data (y_i) based on the number of replications performed so far. The unit depends on the unit of (y_i)	-
$t_{\frac{\alpha}{2}}$	The critical value of the t-distribution at significance level α	[-]
μ	The mean of the result data. Unit depends on the result quantity	-
σ	The standard deviation of the result data. Unit depends on the result quantity	-
x	The number of simulation runs performed	[-]
$Z_{\alpha/2}$	The Z-score corresponding to the two-sided probability of a standard normal distribution with significance level α	[-]
t	The student's t-test value.	[-]
$\mu_{\text{base}}, \mu_{\text{pred}}$	The mean of the data from the reference simulation and simulation with predictions, respectively. Unit depends on the unit of the data	-
$s^2_{\text{base}}, s^2_{\text{pred}}$	The unbiased variance of the data from the reference simulation and simulation with predictions, respectively. The unit depends on the unit of the data	-

Table 2: Definition of MAXBAND-specific variables

Notation	Definition	Unit
$b(\overline{b})$	The bandwidth on the outbound (inbound) direction	[cycles]
c	The signal frequency (1/cycle time) of all TLC's in the network	[cycles/s]
$w_i(\overline{w_i})$	Time, as fraction of the cycle time, from the right (left) side of red at TLC i to the left (right) side of the outbound (inbound) green band	[cycles]
$t_i(\overline{t_i})$	The outbound (inbound) travel time between TLC$_i$ and TLC$_{i+1}$ as fraction of the cycle time	[cycles]
$\delta_i(\overline{\delta_i})$	Binary variables used to decide whether the outbound (inbound) left-turn should lead (=0) or lag (=1)	[-]
m_i	Integer offset in number of cycles at TLC$_i$	[#]
k	A weight used to favor the outbound (if < 1) or the inbound (if > 1) direction.	[-]
$r_i(\overline{r_i})$	The outbound (inbound) red time at TLC$_i$ as fraction of the cycle time	[cycles]
$l_i(\overline{l_i})$	The green time of the outbound (inbound) left-turn at TLC$_i$ as fraction of the cycle time	[cycles]
$\tau_i(\overline{\tau_i})$	The outbound (inbound) queue clearance time at TLC$_i$ as fraction of the cycle time	[cycles]
h, g	Lower and upper limits on the reciprocal speed difference allowed on the outbound and inbound direction	[s/m]

1
Introduction

This chapter introduces the topic. Firstly, background information is provided, starting with the reason for signal control and thereafter discussing why, when, where and how coordination is applied in traffic signal control. Secondly, an elaborate problem description is given. Finally, the structure of the report is laid out.

1.1. A background in coordinated traffic control

Traveling and transporting have always been a core human activity during most of humanity's history. Delays during travel have always been a resulting problem. With increasing development of technologies came different modes of transportation, including motorized vehicles. Since the 20th century, motor vehicles have been central to human mobility and also to city planning. Ever since the same century, automated traffic lights have been installed at intersections, to solve problems regarding traffic safety, capacity and delay. These problems mainly arise from conflicts, essentially caused by traffic from different directions wanting to use the same piece of road at the same time. To this day, traffic light controllers (TLC's) are designed to solve the same types of problems, with an increase of attention for environmental issues as well as equity (Hendriks, 2021).

With the large scale implementation of TLC's, especially on urban arterials, came the idea to coordinate the controllers on multiple subsequent intersections. Coordination is characterized by TLC's from nearby intersections considering each others control decisions and the effects of these decisions. By coordination, in this thesis, the creation of a green wave is meant. Throughout the report the term coordination is used (and not: 'green wave') as a green wave implies that the intersections give a green light to all vehicles before arriving at the stop line. However, sometimes coordination functions better when the first few vehicles have to brake, which will be explained further in section 1.1.2. Finding the right timings for coordinated traffic control can be quite the endeavor. Therefore, programs like the TRAffic Network StudY Tool (TRANSYT) and MAXBAND were created by Robertson (1969) and Morgan and Little (1964), respectively. Programs for coordinated traffic control further developed from pre-timed signals into real-time systems, like SCOOT (Robertson & Bretherton, 1991), SCATS (Sims & Dobinson, 1980) and many others.

1.1.1. WHY COORDINATE?

There exist programs that can coordinate traffic signals in real-time, but why would you really want to coordinate signals in the first place? An example that can help to illustrate the idea behind coordination is discussed using the fictional network shown in Figure 1.1. In Figure 1.1 the different directions are named according a standard coding convention in the Netherlands, where directions are numbered 01 to 12 in the clockwise direction. Typically starting with 01 as the east to north right-turn. Coding practice for different modes of transport or more complicated intersections can be found in the Dutch manual for traffic signal control: 'Handboek Verkeerslichtenregelingen' (CROW, 2014)

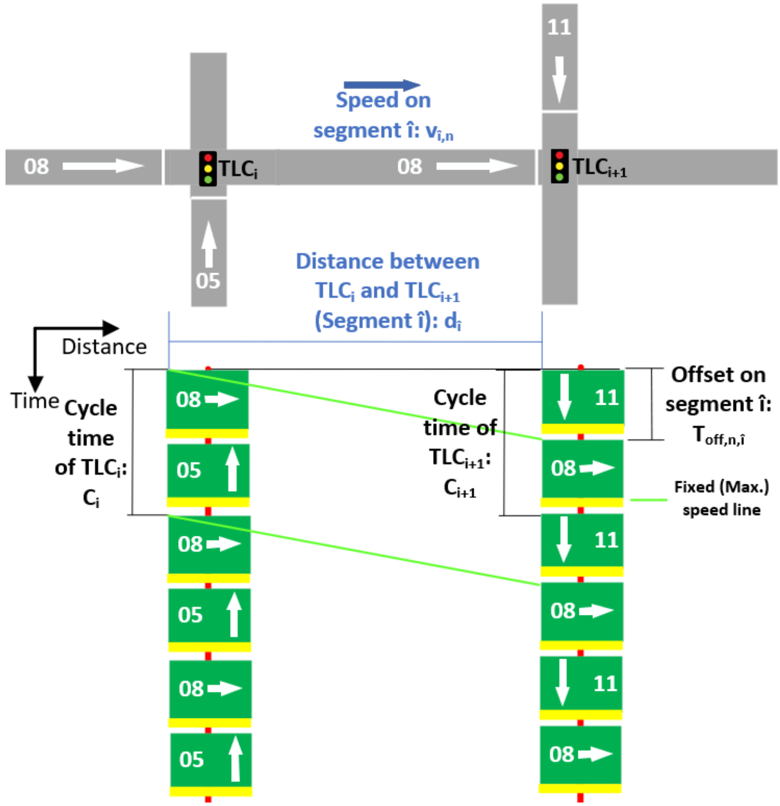

Figure 1.1: A fictional network intended to show the idea behind coordination of traffic light controllers

The basic idea behind coordination is that by combining the design and information of subsequent TLC's, the extra information that comes with this combination can be used to control the traffic more effectively. The main gain in information is related to the arrivals of vehicles, which depends, to some degree, on the neighbouring TLC's.

When, in Figure 1.1, direction 08 at TLC_i receives green it is apparent that vehicles making use of this green will end up at the 08 direction of TLC_{i+1}. In this case it holds for all vehicles departing from direction 08 at TLC_i. When TLC_i communicates the green timing of direction 08 with TLC_{i+1}, TLC_{i+1} will know to expect arriving vehicles at direction 08 at some time after the reported green time of 08 at TLC_i. Instead of timing the start of green of 08 at TLC_{i+1} at some random moment in time, if TLC_{i+1} is able to time the green of its own 08 direction accordingly, the arriving will vehicles will not have to stop or even slow down. In this case, we call direction 08 the coordinated direction. We call the time difference between the green timings of a coordinated direction the offset. To guarantee coordination in subsequent cycles and both directions (02 and 08), TLC_i and TLC_{i+1} will have to operate under the same cycle time, called the network cycle time. Coordination is not limited to just two TLC's, entire arterials or networks, consisting of many TLC's, can and have been coordinated.

Why, when, where and how to apply coordination is not always an easy question to answer as will become clear from this section and sections 1.1.2 and 1.1.3. Starting with why coordination is applied, coordinating traffic signals can be done for a variety of reasons, as stated in Patel et al. (2011), the most common being:

- **To reduce the number of stops vehicles have to make.** Having to stop when driving is logically undesirable for the driver. It can lead to delays, irritations, discomfort and other negative side effects. Reducing stops leads to less deceleration and acceleration, which in turn reduces noise, fuel consumption, emissions and, when near other traffic, can decrease the amount of rear-end collisions and crashes in general (Williamson et al., 2018). Both reduced emissions, noise, collisions, and improved safety (Yue et al., 2022) are reasons for applying coordination, by themselves. Reducing stops mainly applies to the coordinated directions. Uncoordinated directions actually tend to experience more stops when coordination is applied (Salomons, 2021b), which will be further elaborated on in section 1.1.2.

- **To improve progression on an arterial.** Sometimes a reason for choosing to apply coordination is to improve progression, especially for vehicles leaving the city. This should not be confused with lowering delay. Coordination can lead to lower delay for the coordinated directions (Cesme & Urbanik, 2021). However, the coordinated directions experience more delay at the first intersection (Salomons, 2021b). Often, non-coordinated directions and/or modes experience an increase in delay (Cesme & Urbanik, 2021).

- **To motivate drivers to drive a certain speed.** When coordination is successfully applied between two intersections, driving a certain (design) speed will make sure that you receive green on the next intersection. Driving too fast, which is most often seen as problematic, is discouraged, as it will only make the driver have to stop (Leeuwarden Vrij-Baan, 2016). Prerequisites are that drivers are made aware of this (design) speed and the presence of coordination on the road segment.

- **To reduce the queue size on some direction.** By coordinating some specific directions, the queue size on those directions can be reduced. This can be interesting when queue spill-back is a concern, for example when an intersection is located just beyond a movable bridge and queues could spill back onto the bridge deck.

- **For a historical reason.** Coordination might have been applied historically and people got used to it. Even though the old reasons for implementing coordination no longer hold, the fact that coordination was applied in the past might be a reason to keep applying it.

Aforementioned reasons are not exhaustive for nor exclusive to coordinated control. In general it holds that coordination, when forcefully imposed on the traffic control, introduces benefits for the coordinated directions, often at the expense of the uncoordinated directions. It is a political decision whether the reasons to apply coordination somewhere, outweigh the investment costs and whether applying coordination is in line with policy objectives. As an example of political reasons, before 2009, in the UK, implementation of coordination was discouraged by the department of Transport, as the reduction in fuel consumption would lead to less tax revenue (BBC News, 2009).

Besides certain policy objectives, network characteristics largely determine the reasons for applying coordination, which will be discussed in section 1.1.2.

1.1.2. When and where to coordinate?

Whether coordination is actually useful for a specific case and between which directions it should be applied, depends on a number of factors. The reasons why to coordinate over not coordinating, mentioned in section 1.1.1, are only applicable under certain conditions. The network characteristics of the network in which coordination is applied, determine to a great extent the conditions for the usefulness of coordination (TRB and NASEM, 2015). Three characteristics in particular can be identified:

Demands per direction in the network. Section 1.1.1 mentioned that, in general, applying coordination results in benefits for the coordinated directions, at the expense of the uncoordinated directions (TRB and NASEM, 2015). When the demand on the coordinated directions largely exceeds the demand on the uncoordinated direction, applying coordination can be seen as beneficial, for example in terms of reducing stops or queue length, to more people overall. Whether it is ethical to provide benefits to one group (direction) at the expense of another is an interesting question and research topic of its own, however this research will not discuss it further. There are ways to evaluate the traffic performance in the network as a whole, indicating if benefits on the coordinated directions outweigh the losses on the uncoordinated directions. For example, when the number of stops on the coordinated directions is reduced by 10, and the number of stops on the uncoordinated directions is increased by 7, the traffic performance of the network as a whole is improved by 3 stops. From this example it can be seen that demand per direction, especially the difference in demand between the coordinated and uncoordinated directions, is an important characteristic in the decision to apply coordination.

The amount of platoon dispersion between intersections Benefits, in terms of reducing stops or queue length, of coordination rely on the predictability of arrivals on the coordinated direction of the intersection downstream. When green is given upstream, a set of vehicles is released from this intersection and starts travelling downstream. This set of vehicles is called the platoon. When initially receiving green, the vehicles are all spaced closely together, since they were tightly queued up before the stop line. When travelling along the road between intersections, distances between vehicles in the platoon (their headway) increases. This phenomenon is called 'platoon dispersion' by some and 'platoon diffusion' by others. Platoon dispersion occurs for a variety of reasons, as explained in Mathew (2019):

- *Desired speed.* Each driver in the set of vehicles that make up the platoon has a preferred speed of travel. Over time, differences in travel speed cause an increasing difference in headway, increasing platoon dispersion.
- *Noise in the departure process.* When green is given, noise in the departure process may cause platoon dispersion even before vehicles have crossed the stop line. Different reaction times, turning vehicles that brake for a turn or block the through traffic, varying acceleration rates and shockwaves are all examples of things that can cause noise during the departure process.
- *Noise in the travel process.* Similar to noise in the departure process, noise encountered during travel between intersections can increase platoon dispersion. For example merging or lane-changing in multi-lane roads, on-street parking, traffic to and from uncontrolled side-streets and crossing pedestrians.

When travel time between intersections increases, the time and opportunities for (more) platoon dispersion to occur increases as well (Mathew, 2019). This makes travel time between intersections an important network characteristic. Because of platoon dispersion, at some point, arrivals at the downstream intersection no longer consist of a 'compact' platoon, but are scattered through time. Potential benefits, in terms of reducing stops or queue length, of coordination (creating a green wave) no longer apply. According to TRB and NASEM (2015), for any distances greater than 800 meters (half a mile), further review of the platoons is needed to determine if coordination may be useful. Methods have been developed to try to overcome this limitation. One of these is discussed in section 2.1.2. If coordination in two directions is desired, travel times are even more important. Namely, they will largely determine if it is physically possible to coordinate between locally optimal signal timing plans in two directions. This is further discussed in section 1.2.2 and researched in section 4.2. Platoon dispersion is the reason that an overall better coordination can be achieved when the first few vehicles have to brake (compared to a 'green wave' for all vehicles). When the first few vehicles brake, the platoon is compacted, such that less green time is required (also at downstream intersections).

Cycle times of intersections in the network. As mentioned in section 1.1.1, to guarantee coordination in both directions and subsequent cycles of the traffic controller, all controllers in the network are required to operate under the same cycle time. This network cycle time is at least equal to the largest minimal cycle time of the intersections in the network. It should be noted that cycle times of a unit fraction (1/n with n as any integer) of the network cycle time are also allowed on the intersections. For reasons related to traffic performance, as low a cycle time as needed to serve the demand is desired (CROW, 2014). A higher cycle time (than needed) is associated with higher (average) waiting times, more inequality between directions, lower credibility, more stops and delay, lower compliance and longer queues, leading to more queue spill back. When coordination is applied to a network of 6 intersections with minimum cycle times: 66, 61, 105, 63, 60, 65 [sec], the network cycle time will be set to 105 seconds. Consequently, all intersections in the network will operate under a 105 second cycle time. This happens even though a cycle time of 66 seconds would be enough to serve the demand on 5 of the 6 intersections. This brings along all the negative effects associated with a (too) large cycle time on 5 intersections. From this example it becomes clear that similar minimal cycle times are desired in the network of coordinated intersections. The minimal cycle time of an intersection is determined by the maximum cycle time over all conflict groups of the intersection (= the cycle time of the critical conflict group). That is when the 'Classical Method' is applied, which is not the only available method (Stolz & Veroude, 2015). Another popular method is the method of the critical path cycle time. More complex intersections, with more possible movements, or relatively higher demand will have larger minimal cycle times. Coordination will be more beneficial in a network of intersections with similar levels of complexity.

Despite the many potential benefits mentioned in section 1.1.1, coordinated traffic controllers will not always deliver on these benefits. Besides the aforementioned practical limitations, two other reasons are relevant to mention. The first relates to the tough competition that modern local (uncoordinated) traffic adaptive controllers provide. The second relates to the challenge of predicting traffic.

STRUCTURE-FREE LOCAL (UNCOORDINATED) TRAFFIC CONTROL WITH A LOT OF SENSORS

Over the years, local traffic controllers have been improved greatly. Nowadays, modern local traffic controllers are completely structure-free and base their control decisions on a variety of sensors and information. Structure-free meaning that the controllers are able to change green timings and phase orders, adapting to the current traffic situation.

When local controllers are coordinated, they can no longer be completely structure-free. After all, the coordinated directions are to be given green at a certain (predetermined) moment in time. Structure-free coordinated traffic controllers are possible, however, these lose the possibility to guarantee the coordination in every cycle. Not being able to guarantee coordination (a green wave) versus the advantages of a structure-free controller is a dilemma that is decided on a political level. There are also controllers that are somewhere between completely structure-free and completely fixed. Think of this in between as a coordinated controller imposing bounds of some strictness on the local controller, allowing the local controller to optimize freely within these bounds. This will be further discussed in sections 1.1.3 and 2.1.

Guaranteeing coordination thus removes some of the 'freedom' of the local traffic controllers, resulting in sub-optimal control on a local level. This is particularly relevant in the Netherlands (NL), where intersections contain a lot of sensors and modern Intelligent Traffic Light Controllers (iTLC's) also use floating car data. The information added, by applying coordination, becomes relatively less impactful in this case. Despite this, coordination is frequently applied in practice in NL, showing that the information gained by applying coordination can still be of great value.

Uncoordinated TLC's often use some assumptions in their optimization, for example random[1] arrivals or unconstrained outflow (Guo et al., 2019). When situated in a network with closely spaced intersections, these assumptions do not hold, because arrivals are largely dictated by neighbouring controllers and bottlenecks may be present that limit the outflow of certain directions. The performance of a local controller, when situated in a network of closely spaced intersections, can be worse than when the same controller is applied to an isolated intersection. As stated in van Katwijk et al. (2009), intersections in urban areas are spaced closely together, having direct effects in each other's region of influence. Coordinated traffic controllers can account for these effects.

TRAFFIC STATE ESTIMATION AND PREDICTION.

Coordinated traffic control relies on the predictability of arrivals at downstream intersections. Arrivals can be scattered through time and space (directions) and the aforementioned platoon dispersion phenomenon plays a large role here. Furthermore, the amount of arrivals at all intersections also depend on estimations of demand, including the amount of turning traffic (traffic that leaves the coordinated 'main' direction, to make a right- or left-turn).

The coordinated controller makes predictions or assumptions about the propagation of traffic through the network, but this may be subject to errors. Questions arise with regards to robustness to errors, stability of the control system, reliability of the predictions and accuracy of the assumptions. A prediction error resulting in an overestimation may be less or more severe than an error resulting in an underestimation (Poelman et al., 2020). At very high demands, the network can become oversaturated. In an oversaturated network, coordination (in the sense of providing a green wave) is no longer possible, because of the large queues that are unable to be resolved in the green phase. In such a case, a coordinated traffic controller can still be very useful (H. Wang & Peng, 2022), as it can account for spill-back and bottlenecks in the network. However, the usefulness depends on the accurate estimation of the traffic state or prediction of the breakdown.

1.1.3. HOW TO COORDINATE?

As briefly mentioned in section 1.1.1, coordination works by letting subsequent TLC's share information with each other. This section will explain what information is shared and how this can be done.

[1] Random in the sense that there is no bias in the number of arrivals based on neighbouring intersections

SHARING INFORMATION

First of all, every system is different and there are various methods that achieve coordination in different ways. The exact information that is shared takes different forms in different systems. Moreover, different systems may share more or less information, more or less frequently about more or less different quantities. However, there are some general architectures, common across all systems and these will be discussed in this section. Differences between systems are described in more detail in chapter 2.

The minimal information that is shared between intersections with coordinated traffic light controllers is the following:

- **Green offsets:** Coordination requires the green on the downstream intersection to start after a certain time delay w.r.t. to the coordinated green at the upstream intersection. Without sharing some info with regards to the green times, no coordination can be created. This is true for any coordinated traffic controller, however the information may be shared in a different form, e.g. in terms of number of vehicles released at specific time moments.

- **Cycle times:** Since all traffic light controllers in the network require the same cycle time, this network cycle time is communicated between all controllers.

- **Clock time:** It is important for coordination that all coordinated traffic light controllers agree on what time it is (TRB and NASEM, 2015). Otherwise the shared information about the green offsets will be useless. Preferably the cycles are synchronized such that the start and end time of the cycle coincide on each controller. Accurately synchronizing the internal clocks of the controllers is a difficult topic (Hortizuela, 2020) that is outside of the scope of this thesis. It is done either by setting the internal clocks to the exact same time of day or by establishing an interconnection.

CONTROLLER ARCHITECTURE

In a fixed-time coordinated controller the traffic engineer acts as the information pipeline that provides the required offset information between controllers. However, in a real-time coordinated controller, the information must also be exchanged in real-time and this can not be done by a traffic engineer. Different (real-time) coordinated traffic controllers have different architectures. The architecture determines the order of operation and also how the information is shared in the system. The following architectures can be distinguished:

- **Central:** Systems with a central architecture are systems where all TLC's in the network communicate with one central entity. The central entity determines the optimal coordination and shares the required info with all TLC's. This architecture is shown in figure 1.2.

- **Local:** Coordination can also be achieved on a local level. In this architecture, each TLC shares its information with its neighbours only. Often in these systems, one TLC is appointed as the master. The master makes sure that the cycle time and clock time are in order in the network by sharing this information with its

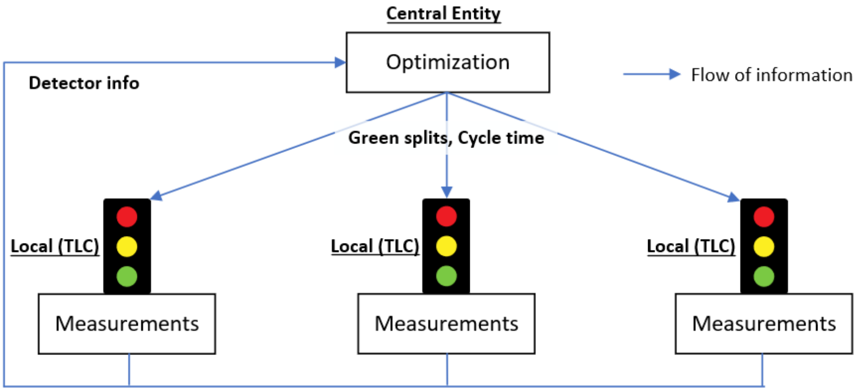

Figure 1.2: Central coordinated traffic control architecture

neighbours, who share with their neighbours, etc., until all TLC's in the network have been reached. Each local TLC determines the optimal coordination based on information from the neighbours. Local coordination often cannot guarantee a green wave across the arterial (Salomons, 2021b). This architecture is shown in figure 1.3.

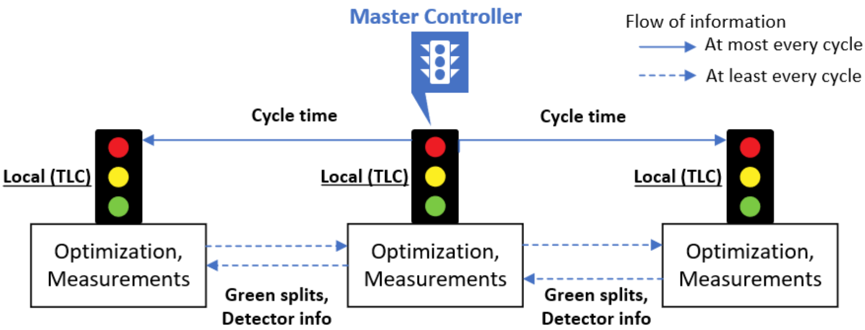

Figure 1.3: Local coordinated traffic control architecture

- **Combination:** There are also combinations possible and these are quite common in practice. In these systems, it is often the case that the long-term decisions (usually longer than 1 cycle time) and strategy is determined centrally and communicated to all TLC's. The short-term decisions (usually shorter than 1 cycle time) and adaptations are determined locally based on neighbours sharing information. In many cases, the central system's findings function as boundaries, wherein the local systems are able to conduct further optimization. This architecture is shown in figure 1.4.

The actual sharing of information takes places via physical cable connections made on the street and internet connections. Dependent on the system, different demands are

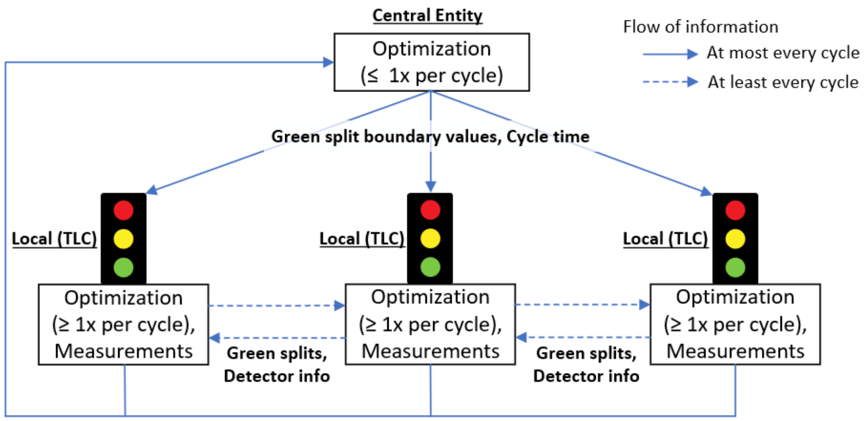

Figure 1.4: Central-Local combination architecture

placed on the connections and their quality. Most modern systems work in real-time, with frequent transfers of information, requiring a fast connection. More importantly, since traffic control systems are important to guarantee safety, the connections have to be very reliable. Glass fibre cables are the current state-of-practice communication cables.

With regards to all things mentioned about why, when, where and how to coordinate: There exist multiple methods and philosophies regarding coordination. A variety of coordinated traffic control systems have been developed over multiple decades of research. Each system comes with its own strong points, trade-offs, models, limitations, assumptions, etc. Chapter 2 will further elaborate on the various coordinated traffic controllers from literature and their differences.

1.2. Problem description

In section 1.1.2 some limitations of coordinated traffic controllers were discussed. One could regard every limitation as part of a problem in coordinated traffic control. Or one could regard the limitations as they are and regard the advantages coordination brings within the limitations as insufficient and thus a problem. In this thesis, not all problems of coordinated traffic control can be addressed. It is desirable to focus on some particular aspect, problem or idea, with more detail, rather than approaching the subject to broad and consequently not being able to go into depth. With this mind, this section will elaborate on the problems, approach and scope this thesis wishes to incorporate.

1.2.1. Main problems

Increasing levels of urbanisation and the subsequent increase in traffic demand have put more pressure on urban coordinated traffic control. Not only have urbanisation levels increased, they are expected to keep increasing in the coming years (Ritchie & Roser, 2018). This puts pressure on urban mobility and leads to a desire to improve the traffic performance in the network controlled by the coordinated traffic controller. At the same time, cities face the challenge of improving their living environment and reducing their climate impact. Adding more lanes to the road is not a solution in this regard, as is discussed in P. M. Jones (2020).

Improving the performance of coordinated traffic controllers, especially in terms of stops and delay in the network, can provide (part of) a solution to these problems in urban environments. This reduction is sought through two different research directions. The first research direction considers an idea called the variable speed, discussed in section 1.2.2. The general idea is that, with a controllable speed per segment (the piece of road between two subsequent intersections) in the network, in the future a reduction of stops and even delay may be achieved. The second research direction investigates potential improvements of a coordinated traffic controller based on demand predictions and is discussed in section 1.2.3. As this thesis is performed at Vialis, their TopTrac (see section 2.1.3) coordinated traffic controller will function as a starting point for the second research direction. TopTrac currently takes the information of the traffic flow from the current optimization period as an input for the optimization for the next period. Consequently, the decisions are reactive to the changing traffic demand and it is believed that by making the decisions proactive, a reduction of stops and delay can be achieved.

Both approaches have the same goal of reducing stops and delay in the network controlled by the coordinated traffic controller. This is the only similarity between the two approaches, and therefore they are treated separately throughout the report. To be able to accurately conclude about the effectiveness of either solution, both are also evaluated separately.

REDUCING STOPS AND DELAY IN THE NETWORK

Reducing the number of stops in the network fits the goals of modern cities. Reducing stops in the network will, besides providing a more comfortable driving experience, greatly benefit a variety of other metrics, like fuel consumption, safety, noise and emissions. With the rapidly changing vehicle fleet, containing increasingly more electric vehicles (Irle, 2022), focusing directly on the benefits of, for example, fuel consumption or noise would depend highly on the current composition of the vehicle fleet.
Focusing only on stops does not tell the whole story. A time-based indicator is also needed to evaluate the network performance, as, for example, the duration of the stops may be much greater. Vehicle delay or total time spent in the network are two of these indicators.

1.2.2. RESEARCH DIRECTION: VARIABLE SPEED

The first research direction considers a variable speed. There exists two ideas that use a variable speed that may lead to a reduction in stops and or delay. Both ideas build on the assumption that vehicle speeds can be influenced, for example through an Intelligent Speed Adaptation (ISA) device (European Road Safety Center, n.d.), allowing a variable speed between intersections.

COORDINATING BETWEEN DIFFERENT CYCLE TIMES

The first idea is to use a variable speed to coordinate between intersections with a different cycle time. This idea is explained using Figure 1.5.

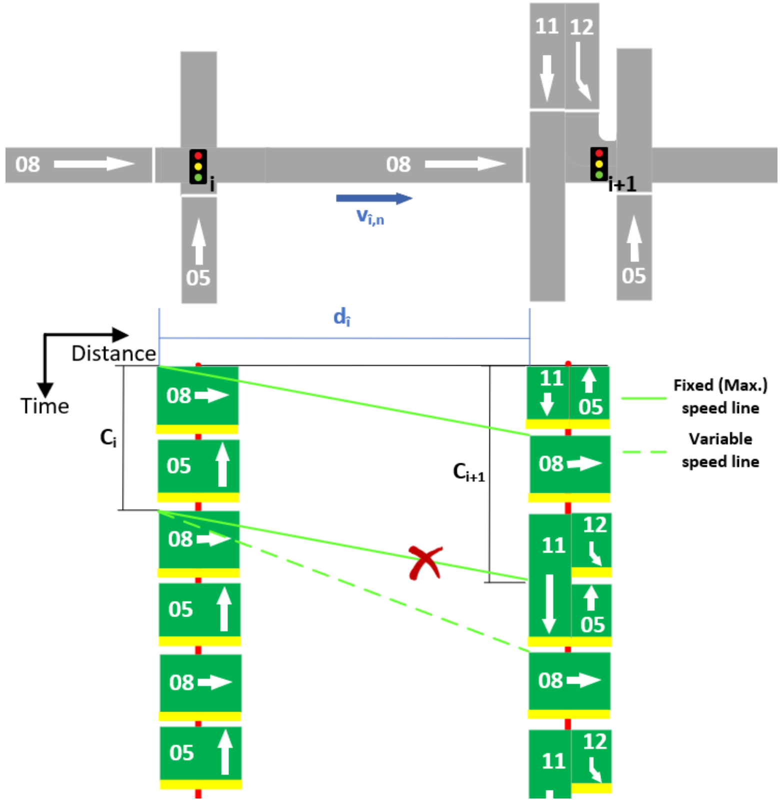

Figure 1.5: Fictitious network and signal timings used to illustrate the first idea for using a variable speed.

In Figure 1.5, a simplified, but general problem of coordinating TLCs is displayed. Namely, the problem of requiring equal cycle times. In Figure 1.5, TLC_{i+1} requires a larger cycle time to fulfill the demand for all its directions than TLC_i. As shown by the red cross in Figure 1.5, in the second cycle of TLC_i, the coordination is no longer possible with the same speed as the first cycle. The first idea investigates to what extent a variable speed can be used to overcome this problem.

COORDINATING BETWEEN THE STARTS OF GREEN IN TWO DIRECTIONS

The second idea is to use the variable speed to coordinate between the starts of green in two directions. This idea is explained using Figure 1.6.

Coordination between the starts of green in two directions is not always possible, such as indicated by the red crosses in Figure 1.6. Note that it is important to mention specifically that the starts of the green times are concerned here. With some imagination, some type of coordination is possible for this network, just not with the starts of green. Figure 1.7 shows some other green timings where some level of coordination is possible.

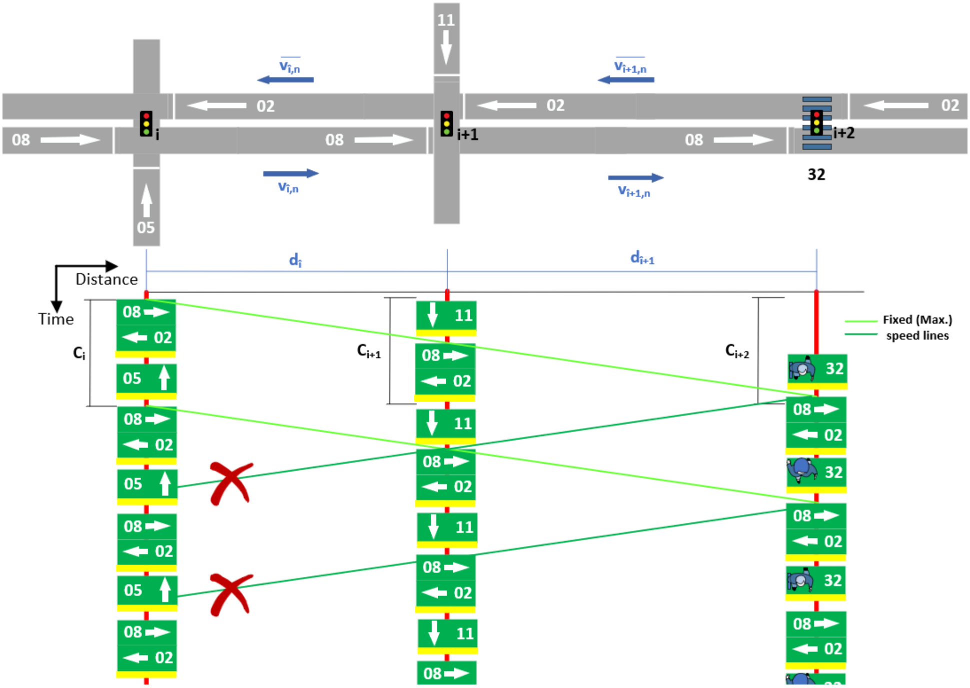

Figure 1.6: Fictitious network and signal timings where coordination between the starts of green is not possible with a fixed speed.

In Figure 1.7, the cycle time was extended, such that the green times of the coordinated directions could be extended. The coordination shown in Figure 1.7 is not discussed further, this Figure is just intended to show that by increasing cycle times and green times, a coordination that does not concern only the starts of green is possible. As mentioned in section 1.1.2, when coordination is applied, the demand for the coordinated directions typically exceeds that of the uncoordinated directions. When the coordinated directions have substantially larger demand than the other directions, or when all other directions conflict with both coordinated directions, the coordinated directions will be given green (nearly) simultaneously in the locally optimal signal timing plans. As Figure 1.6 showed, coordination between these locally optimal signal timing plans is not always possible with a fixed speed. Figure 1.8 shows that, without increasing the cycle time, coordination between the starts of green is possible, by using a variable speed.

The speeds between the first and second intersection in Figure 1.8 are slightly adapted compared to Figure 1.6. A variable speed allows, in this case, to coordinate between the locally optimal timing plans. This second idea for using the variable speed will investigate when the feasibility of the coordination between the starts of green with a fixed speed, compared to a variable speed.

In general, both ideas use the fact that with the variable speed, more offsets are possible, such that the coordination between the starts of green can become feasible, whereas it was otherwise impossible. Not all offsets are possible, as some will result in unrealistic

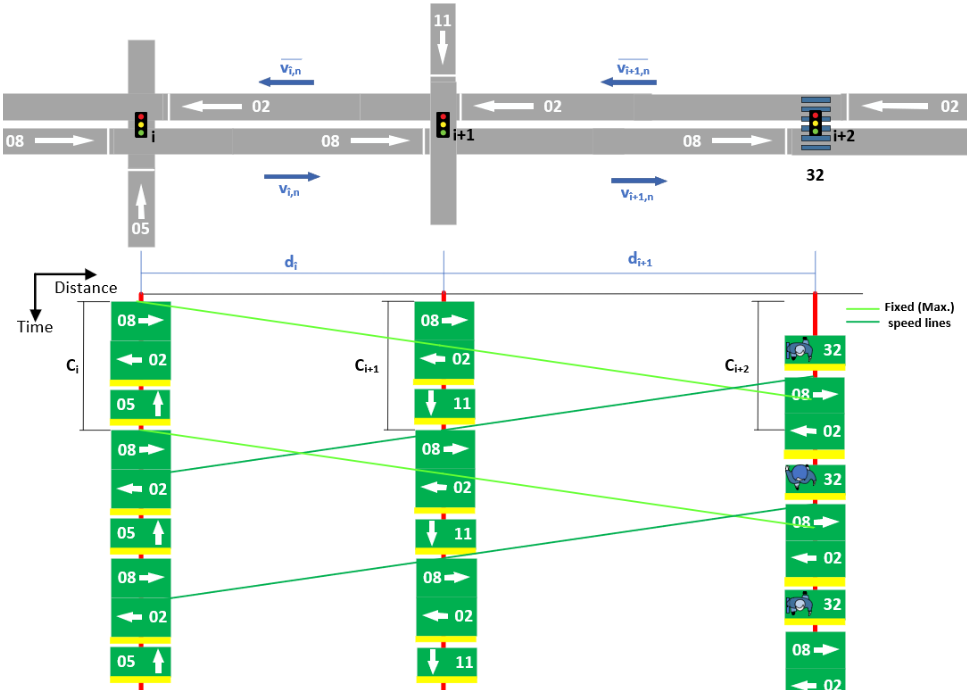

Figure 1.7: Fictitious network and signal timings where some level of coordination is possible with a fixed speed.

speeds. The investigation of this variable speed idea will constrain the speed between some maximum value (the speed limit) and some minimum value. Furthermore, some assumptions will be made, which are stated in section 1.2.4.

1.2.3. RESEARCH DIRECTION: THE POTENTIAL OF DEMAND PREDICTIONS

Real-time coordinated traffic control relies on estimated or measured demand to make an informed control decision. So does the TopTrac coordinated traffic controller by Vialis, which bases this estimate on measured traffic volumes at the stop line. In TopTrac, optimizations are done every two cycles, which are typically 60 to 100 seconds in length. The control decision thus influences the traffic propagation for a couple of minutes in the future, while at that point the traffic volume measured during the previous optimization, may be outdated. Therefore, it is thought that a prediction of the demand, can change the control decision from a reactive nature, to a proactive nature.

If the information about demand, supplied to TopTrac, can be made proactive, benefits in terms of reduced stops and delay are expected. That is, provided the prediction is accurate. In TopTac, the control decisions are based on the effects they have on stops and delay in the network. Since these control decisions are partly based on the demand per direction, it seems logical to investigate whether supplying more accurate information about the demand may lead to reduced stops and delay. Dealing with erroneous

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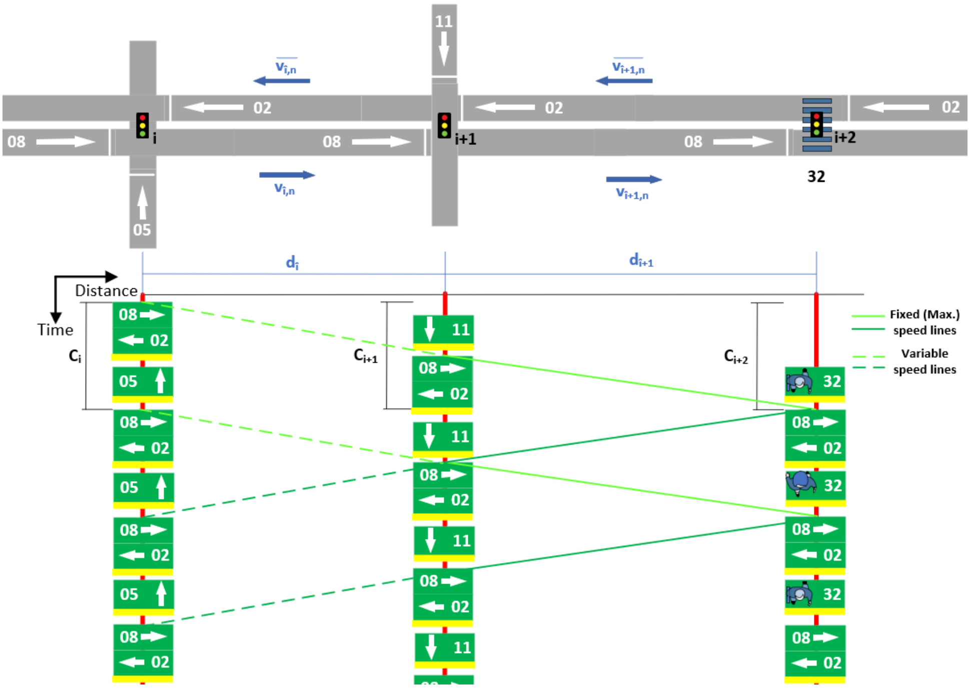

Figure 1.8: Fictitious network and signal timings used to illustrate the second idea for using a variable speed.

predictions is not straightforward, as shown in Poelman et al. (2020) and discussed in section 2.4. Therefore, this thesis focuses on potential improvements obtained via non-erroneous predictions. The goal is to show the maximum potential that predictions in TopTrac could have, such that Vialis can weigh further investment decisions based on these results. General insights from this research into demand predictions for TopTrac can be used in other controllers, that consider using demand predictions, as well.

1.2.4. SCOPE

This research is subject to limited resources, the main one being time. Furthermore, the researcher does not possess unlimited capabilities. These are reasons for clearly specifying the scope of the research. Other limitations in scope, besides the aforementioned limitations regarding non-erroneous predictions, are as follows:

Assumptions: The research of the variable speed for coordination between locally optimal signal timing plans in two directions in this thesis relies on an assumption that outbound and inbound green windows are given green (nearly) simultaneously. It is known from traffic engineers at Vialis that in these timing plans, the green windows preferably are part of the same stage, as the demand for these directions often far exceeds the demand for other directions. When some conflicts are present that conflict with only one of the two coordinated directions, the coordinated directions may be situated in consecutive stages. In practice, the green

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times largely overlap, even in this second case, according to the traffic engineers at Vialis. The structure of locally optimal signal timing plans on arterials with a desire to be coordinated is not studied in this thesis, so it remains an assumption of this thesis that the starts of green of outbound and inbound green windows are (nearly) simultaneous in locally optimal signal timing plans of coordinated TLC's.

During the research it is simply assumed that I2V communication of the speed is possible and effective. Already, various systems currently in development can provide an in-car speed advice, such as ODYSA and GLOSA. These are further discussed in section 2.1.2 and section 2.3.2. The European Union has made it mandatory for new vehicles manufactured from 2022 onward to include an Intelligent Speed Assistance (ISA) device that is able to impose a speed limit, based on GPS coordinates (European Road Safety Center, n.d.). These developments show that controllable variable speeds on different road segments is not a far-fetched technology.

A large amount of research in (coordinated) traffic control is focused at reducing the computational complexity of the traffic control problem (Jeschke, 2020). Advanced coordinated traffic controllers, like the Model Predictive Controllers proposed in Jansen (2016), Lin (2011), Lin et al. (2012) and Jamshidnejad et al. (2017) are not only discussed in the context of predictions, in section 2.4, and not in the context of a variable speed. These controllers are still in testing, have never been tested in practice or already indicate in literature to be infeasible (computationally) for practical applications. An additional decision variable, such as the variable speed discussed in this research, is expected to add to the complexity of the problem. When looking to test the variable speed, this research will investigate whether the variable speed can improve traffic performance in currently computationally feasible models and conditions. Investigating the usage of a variable speed in these types of controllers is too complex for the scope of this thesis.

For testing the potential of demand predictions, it is assumed that traffic volume measurements at the stop line can be used to predict the demand to near-perfection in a follow-up simulation run. Section 6.2 explains this assumption further.

Constraints: The variable speed concept will be constrained during this research. Changing the speed can have many effects on drivers. Firstly, it is not known exactly how this variable speed would be implemented, whether it would be via an in-car advice, a variable message sign or for example a compulsory limit via ISA. Dependent on the implementation, the response of drivers will be different and behavioural adaptation to the variable speed will be different as well. This research will not consider these effects. The research is constrained to exploring the possibilities of a variable speed, when this speed is adhered to. Moreover, to compare the variable speed to the fixed speed, the maximum speed, which is chosen as the speed limit, is used as the fixed speed.

In this research, ideas that allow coordination of multiple downstream directions with one upstream direction are not considered. The variable speed could be used for such an application, where traffic turning onto the arterial can be coordinated, as well as the main direction. This application is outside the scope of this thesis, because there is not enough time to properly explore this during the research.

Both the variable speed research and the tests of the potential of demand predictions consider only undersaturated demand. When oversaturation occurs, coordination (as in creating a green wave) can be heavily distorted. It is outside the scope of the research in this thesis to also consider effects of oversaturated demand conditions.

Research directions: The variable speed and the potential of demand predictions research directions are treated separately. Ideally, if useful, both are used simultaneously, but this will complicate the control problem in ways that take away from the analysis of the effectiveness of each solution, individually.

Problem size: This thesis considers only urban arterials of limited length, without multiple routes per origin-destination pair. This simplification allows to investigate the effects of changes in the coordinated traffic controller clearer. Networks with multiple routes per origin-destination pair often include route choice problems as well, which this thesis wants to stay away from. Some advanced coordinated traffic controllers not only include the coordination of traffic signals on the urban arterial, but also include other traffic management tools, like: Ramp metering, dynamic route advice panels, variable speed limits, etc. In many practical situations this makes sense: coordinating the signals on an arterial to produce large, compact platoons is quite an opposite objective of a ramp metering measure. This thesis however only concerns the coordination of traffic signals along an arterial and neglects the inclusion of other traffic control instruments, because otherwise the problem will become too complex.

Evaluation: The evaluation will be done via simulation only. The simulations done in this research are limited to 1 case study, considering an urban arterial in Almere, The Netherlands. The two research directions are evaluated separately. The variable speeds are implemented in semi-fixed control applications. The developed variable speed model is not directly suitable for real-time applications. The tests of the potential of demand predictions are evaluated using one coordinated traffic controller: TopTrac by Vialis. Conclusions about the potential effectiveness of the predictions remain limited to the one investigated controller.

The variable speed research direction is only evaluated in a fixed-time setting. This holds for both the theoretical part and the evaluation part. Developing a model capable of using and optimising the variable speed in real-time is outside the scope.

1.3. DOCUMENT STRUCTURE

This report is structured as follows. In chapter 2, a literature survey is performed to gain as much knowledge as needed about coordinated traffic control. The survey also tries to identify research gaps and find related literature to the proposed research directions of the variable speed and the demand predictions (tests). The findings of the literature survey and the description of the problem and research directions from this chapter, are used in chapter 3. Chapter 3 states the research questions and discusses the methodology used to answer them. The following two chapters, chapter 4 and chapter 5 are specifically aimed at the variable speed research direction. Chapter 4 discusses theory for coordinating with a variable speed compared to a fixed speed. Coordination with different cycle times and the feasibility of coordination in two directions are considered in great detail. In chapter 5 a model is formulated that can optimize signal timings plans for coordination, using the variable speed. Chapter 6 is aimed at the potential of demand predictions, this chapter discussed how the tests of the potential of the predictions are included in TopTrac and how the near-perfect predictions are made. The simulation setup is discussed in chapter 7. The same setup, in terms of the simulated network and the simulation tool, is used by both simulations with the variable speed and simulations that test the potential of demand predictions in TopTrac. Meaning that in chapter 7, no distinction is made between the two research directions. The results of the simulations are presented and analyzed in chapter 8 and further discussed in chapter 9. Finally, chapter 10 draws the conclusions of this research. This chapter also presents recommendations for future work and investments regarding the variable speed and the potential of demand predictions.

2

LITERATURE SURVEY

This chapter presents a literature survey, the main goals of this literature survey can be summarized as follows:

- Gain insight into various coordinated traffic controllers and their differences, with the aim of developing a deeper understanding of the systems. Insights can be obtained about whether the ideas discussed in section 1.2.2 can be used or are used in coordinated traffic controllers. This is done in the first section of this literature survey, where various coordinated traffic models and controllers are surveyed.

- Gather knowledge about the application of a variable speed in various traffic environments, to learn from these applications and how they relate to the proposed variable speed. This is done in sections 2.2 and 2.3, where Variable Speed Limit (VSL) and Connected Vehicle (CV) research are surveyed, respectively. Any literature that was found that closely resembles the ideas of the proposed variable speed is surveyed in section 2.5.

- Gather knowledge about the application of demand predictions in coordinated traffic control. This is done in section 2.4.

2.1. COORDINATED TRAFFIC CONTROLLERS

2.1.1. THE CORE OF COORDINATION METHODS

There exists two general methods for determining the coordination in fixed time:

1. **Progression-based methods.** These methods are characterized by having an objective that is focused on maximizing progression over the coordinated directions.

2. **Disutility-based methods.** These methods are characterized by having an objective that is focused on minimizing a Performance Index (PI) and thereby minimizing the delay and stops (or any other metric included in the PI) in the network.

The following subsections will explain more about two key models, developed in the seventies. The progression-based method: MAXBAND and the disutility-based method: TRANSYT. These methods were originally developed to aid in the design of fixed-time signal timings for coordinated traffic controllers (Papageorgiou et al., 2003). These are the only fixed time approaches discussed in this literature survey, as state of the art control operates in real-time. Understanding MAXBAND and TRANSYT, however, is important for understanding the real-time systems as well.

MAXBAND

The MAXBAND model was developed in 1964 by J.T. Morgan and J.D.C. Little (Morgan & Little, 1964). In their paper, they define the bandwidth of a street as the proportion of the cycle during which a car could start at one end of a street and travel to the other end of the street, without stopping for a red light. In MAXBAND, timings on the conflicting directions (conflicting to the out- and inbound main direction) are considered given. MAXBAND was later extended in Little et al. (1981) to include variable cycle time, speeds and left-turn lead or lag choice per intersection. This version of the model is mathematically described in section 5.2. Most interestingly, MAXBAND is able to include a variable speed in the optimization model. In MAXBAND, a variable speed is allowed and used to provide a larger progression bandwidth. Traffic engineers using MAXBAND had to weigh the gains in additional bandwidth to the reduction in the true representation of the travel speed. Usually, only small variations in speed are used.

As duration (as fraction of the cycle time) of conflicting reds is considered given in MAXBAND, the problem boils down to placing these reds such that the bandwidth along the main corridor (in one or both directions) is maximized. This problem can be represented as a Mixed-Integer-Linear-Program (MILP), thereby allowing efficient MILP solvers to find the optimal solution fast.

Even further developments resulted in allowing variable bandwidths in the MULTIBAND program (Gartner et al., 1991). Both MAXBAND and MULTIBAND were extended from applications along an arterial to application in a network of arterials in Chang et al. (1988) and Stamatiadis and Gartner (1996), respectively.

A real-time version of MAXBAND and MULTIBAND is presented in Andrzej (1997) as REALBAND. REALBAND uses real-time data obtained by a video detector. Furthermore, some additional logic is presented to able to make real-time decisions about extending or terminating the green phase. One assumption made in the MAXBAND-like programs, is that unsplit bandwidths provide better progression than split bandwidths. Therefore, ever since Morgan and Little (1964), MAXBAND has only been able to calculate unsplit bandwidths. This assumption was explored in Papola (1992), where it was mathematically proven that optimal split bandwidths nearly always coincide with unsplit bandwidths. Especially on longer arterials, the differences between split and unsplit where found to be negligible. Logically, when the additional lost time is ignored, the best solution of split bandwidths is never worse the best solution of unsplit bandwidths (the solution space of the latter is encapsulated in the former). The negligible differences, additional computational effort and additional lost time make allowing split bandwidths not attractive.

More recent developments of the model have led to a version called PM-BAND, which is able to include multiple modes of transport. This version is presented in Ma et al. (2019) and also includes an extended formulation that can produce partitioned bandwidths. By partitioning the bandwidths, not only can PT disruptions be dealt with, an overall better progression can be achieved on longer arterials and feasible solutions (positive bandwidth) are obtained in cases where regular MAXBAND or MULTIBAND could not produce any.

To conclude this section, a model closely related to MAXBAND is discussed. This model, called PASSER (Progression Analysis and Signal System Evaluation Routine), was developed by the Texas Transportation Institute. The PASSER program has been updated multiple times ever since it's initial release. This literature survey only includes the core functionality of the latest version Chaudhary et al. (2002). The way PASSER optimizes progression is as follows. First, from either end of the arterial, a starting cycle is selected. Then, coordination in one direction is optimized, such that the bandwidth in this direction is equal to the maximum green time of the normative intersection for this direction. Next, given the timings for coordination in one direction, coordination in the opposite direction is optimized by optimizing the selection of phase sequences and offsets. With one direction perfectly optimized and the other direction optimized as best as possible, the coordination in both directions is slightly adjusted with the goal of finding the system optimum. PASSER tries to do this by minimizing interference of the bands of the two directions. The objective of minimizing interference can be adjusted by the user to give preference to a certain direction. Minimizing interference is, as explained in Messer et al. (1973) and mentioned in van Katwijk (2008), analogous to maximizing bandwidth in the form presented by Little (Morgan & Little, 1964). The differences between PASSER and MAXBAND are the following: In PASSER the optimization/analysis procedure is split into multiple steps, which allows to define the green time and bandwidths in units of seconds. In MAXBAND, these quantities are depicted as fractions (of the cycle time). The step-wise procedure of PASSER may be easier to understand and it's input values more understandable, however it sacrifices the simultaneous optimization of both directions and multi-cycle coordination included in MAXBAND. Furthermore, since MAXBAND is formulated in one single step (including all constraints), it is easier to modify, add or remove constraints to the problem, according to the users liking. Additional differences are the capabilities of PASSER to include permitted (left-turn) conflicts and the search algorithm used to find the optimal solution. PASSER, contrary to MAXBAND, exhaustively evaluates solutions. Besides a hill-climbing procedure, the latest version can also use a genetic algorithm (Chaudhary et al., 2011). Heuristics are used to limit the search space. These methods cannot guarantee a global optimal solution, in contrast to MAXBANDS MILP formulation, which can.

TRANSYT

TRANSYT (TRAffic Network StudY Tool) is a tool used to optimize the timing of traffic signals for a network and was developed by Robertson (1969). TRANSYT uses a traffic model that calculates performance indicators like stops and delay resulting from certain control decisions. These performance indicators are grouped together in a Performance

Index (PI). In TRANSYT, a set of initial timing plans (one per intersection in the network), are adjusted in terms of green timings and offsets with the goal of minimizing the PI. It is required that these timing plans have an equal cycle time (or half of that cycle time) and that the green timings are connected to so called pulses that can be adjusted during the optimization. Minimization of the PI happens via a hill-climbing optimization method (and in later versions also via genetic algorithms).

The more advanced features of TRANSYT include a platoon dispersion model and the ability of later versions to include public transport, uncontrolled intersections and indicators like fuel consumption in the PI as well (Wilson, 1993). Everything in TRANSYT is modelled in discrete time steps of at least one second. The traffic model is macroscopic and link-based. The only way to optimize the cycle time using TRANSYT is to use the CYOP add-on module (Wilson, 1993), which makes an estimate based on the input demands, or to run the program many times for different input cycle times.

A real time version of TRANSYT is basically the SCOOT system, which is surveyed in section 2.1.4. TopTrac, mentioned in section 2.1.3, also uses TRANSYT as a part of the system.

OVERVIEW

According to Robertson and Bretherton (1991), a weakness of the progression-based methods is that their performance decreases in complex situations, where the 'bands' are distorted in many ways. These are situations where many complex movements intersect and substantial demand from side directions causes complex queues. However, the progression-based methods are able to optimize the speed, which is a very useful property for the research of the variable speed.

Table 2.1 shows an overview of the differences between progression-based (MAXBAND) and disutility-based (TRANSYT) models. Differences in Table 2.1 are true for MAXBAND and TRANSYT and but not in general for all progression- and disutility-based models. Most progression- and disutility-based models share the same characteristics, however there are of course exceptions. For example, the progression-based model PASSER V does not employ a constructive search algorithm, but rather an exhaustive (move-based) search algorithm.

The importance of MAXBAND and TRANSYT on coordinated traffic control should not be understated. Many of the systems applied in practice today are evolved and enhanced versions off either of these programs. The programs are often used as benchmark in coordinated traffic control research.

Research by Cohen (1982) tried to combine the strong suits of both MAXBAND and TRANSYT together, which seems logical, as their strenghts seem to complement each other. In Cohen (1982), this is done by using MAXBAND to generate the initial timing plans that were then further optimized by TRANSYT. This combined approach showed improvements over running each of the programs separately. The research did not consider to use the optimal speeds found by MAXBAND in the TRANSYT model as well (this wouldn't have made sense anyway, as speed is not controlled in the simulation). One thing that is not clear from the paper, is how the MAXBAND output is exactly converted to a TRANSYT input, as MAXBAND doesn't provide green times for side directions, while this is required for the TRANSYT input. More recently, research by Cho et al. (2019)

Table 2.1: Overview of differences between progression- and disutility-based models.

Method	**Progression-based: MAXBAND**	**Disutility-based: TRANSYT**
Objective	Maximize progression (bandwidth)	Minimize disutility (PI)
Inputs	Minimum and maximum bounds for speed, cycle time and speed differences. Red time fraction of the cycle for main directions, queue clearance time fraction on main directions. Link lengths.	Flows per link, speeds, link lengths, initial timing plans (including location of pulses and the cycle time), platoon diffusion coefficient, capacity, and some simulation inputs.
Variables	Offsets, green and band timings, cycle time, bandwidth, speeds	Offsets, green timings.
Search algorithm	Constructive: Branch and bound	Move-Based: Hill-climbing or genetic algorithm
Strengths	Easy interpretation due to geometric origin, option to include optimization of left-turn lead/lag choice.	Considers all directions, considers platoon dispersion.
Weaknesses	Focused on main direction, doesn't include platoon dispersion. Maximization of bandwidth does not generally lead to minimization of delay and stops.	Does not guarantee global optimum solution. Doesn't simultaneously optimize the cycle time.

showed benefits by including the TRANSYT platoon dispersion model into MAXBAND. This approach also seems promising, but doesn't overcome many of the other weaknesses of MAXBAND like lacking optimization for the side directions.

2.1.2. ODYSA

Optimalisatie van de Doorstroming door dYnamische SnelheidsAdvisering (ODYSA) is a system developed by DTV Consultants in the Netherlands (DTV, n.d.-b). ODYSA stands for optimization of the throughput by providing dynamic speed advice to drivers. The initial system gave a speed advice to drivers via road-side message boards. The goal of the system was to provide better coordination and coordination over longer distances by reducing platoon dispersion. Through the speed advice, ODYSA is able to make the front of the platoon slow down a bit and make the tail of the platoon speed up.

The ODYSA system proved to be able to benefit especially the coordinated directions in a large way. Like with nearly all coordinated controllers, the side directions suffered. Furthermore, the speed advice led to frustrations and disappointments in some cases. When others do not follow the advice or when a slow speed is advised, drivers got frustrated. When following the advice didn't result in a green light, drivers got disappointed and the system lost credibility. In some cases, drivers tried to drive extremely fast when they saw a slow or no speed advise, to be able to reach the green light from a cycle sooner than the intended cycle.

The system was further developed to be able to provide the speed advice in-car, eliminating the need for the road-side signs. ODYSA is not capable of real-time optimizations. The coordinated semi-fixed timing plans are not updated in real-time. This allows for more accurate predictions of the time to green and time to red, upon which the speed advice is based.

The research presented in S. Chen et al. (2011) is similar to the idea behind ODYSA. In S. Chen et al. (2011), variable message signs are used to influence the speed of traffic so as to create compact platoons with speeds that allow for coordination.

2.1.3. TopTrac(Flex)

TopTrac is a coordinated traffic controller owned by Vialis. This controller uses TRANSYT to minimize stops and delay in the network in real time. By running TRANSYT multiple times, it can provide the cycle time, start of fixed green time (GF) and end of extension green time (GX) for optimal control of the semi-fixed controllers in the network. Within these timings the offsets found by TRANSYT are included. The timings found by TRANSYT are applied to all directions of all the TLC's in the network. The model uses, amongst other things, traffic volumes per direction as input. The traffic volumes per direction are the main real-time input used by TopTrac. Besides traffic volumes, some pre-specified base timing plans, used by TRANSYT, are supplied which may be selected based on a specific time of day.

In the TopTracFlex system, the local controllers are not semi-fixed, but adaptive FLEX controllers. Adaptive meaning that there is no fixed structure for the uncoordinated directions, their order and duration is optimized by FLEX on the go. The TopTrac output is only used for the coordinated directions. The final signal realization is done by the local FLEX controllers, where the main directions are the only directions not controlled adaptively.

By imposing the cycle time, green times and offsets found by the central TopTrac optimization, coordination can be guaranteed every cycle. The way it is implemented, as semi-fixed GF and GX, allows the green splits to be fine-tuned somewhat. Theoretically, TRANSYT does not guarantee coordination. The model may coordinate a different direction or skip coordination if it sees that this results in a lower performance index. However, through the base timing plans and by applying weights to the desired coordinated directions, in practice a consistent coordination is achieved.

2.1.4. SCOOT

Split, Cycle and Offset Optimization Technique (SCOOT) is a central coordinated traffic controller developed by the British governments' Transport and Road Research Laboratory (TRRL) (Robertson & Bretherton, 1991). Initially, the system was essentially an online, real-time version of TRANSYT. Using real-time data measurements to update the information input to TRANSYT. The workings of SCOOT are therefore nearly identical to TRANSYT. SCOOT requires measurements of traffic, for example via inductive loop detectors. According to Robertson and Bretherton (1991), these sensors preferably are placed well upstream off the stop line, just downstream of the previous junction.

SCOOT optimizes incrementally on three different levels with different frequency:

1. **Green splits** Every few seconds, before a phase change, the green splits are optimized. In SCOOT this comes down to a decision whether to truncate or extend the green time by 4 seconds (Robertson & Bretherton, 1991).

2. **Offsets** Once every cycle, the offsets are optimized. This also comes down to increasing or decreasing the current offset values by 4 seconds.

3. **Cycle time** Every few minutes, thus once every few cycles, the cycle time is optimized. The cycle time may be incremented up or down by a few seconds during this procedure.

The incremental optimization makes changes in timings gradual and less prone to sudden drastic alterations that might disrupt traffic flow (Bretherton, 1990).

2.1.5. SCATS

Sydney Coordinated Adaptive Traffic System (SCATS) is a coordinated traffic controller developed by the Roads and Traffic Authority of New South Wales, Australia. SCATS was designed to control a generally larger network of signals than merely one arterial. SCATS does this by employing a hierarchical system of a central, several regional and many local intelligent computers.

- The local intelligent microcomputers, installed at each intersection make tactical decisions on signal operation and process detector data (Sims & Dobinson, 1980).

- The regional computers control up to 200 local microcomputers and analyze the detector data from the microcomputers, to implement a real-time operation of the traffic signals.

- The central, supervisory computer does not automatically influence traffic signal operation. This computer is merely used for monitoring and analysis purposes.

In essence, the system tries, via the regional computer, to adjust cycle time, offsets and splits such that the local computers can further operate under vehicle-actuated conditions. The optimization with regards to coordination is therefore mostly done via a central system.

2.1.6. UTOPIA-SPOT

The Urban Traffic OPtimization by Integrated Automation (UTOPIA) model was developed by Mauro and Di Taranto (1990). SPOT is the name of the local controllers, integrated into the UTOPIA system, resulting in a combined system: UTOPIA-SPOT. UTOPIA is typically the name of the central entity, though it is sometimes used to reference the whole system.

UTOPIA-SPOT has the capability to include Public Transport (PT) priority throughout the network. However, the coordination for private vehicles may be disrupted by the PT priority, resulting in a loss of performance (Wahlstedt, 2011). The UTOPIA-SPOT systems allows for a variety of control strategies and weights for performance indicators to be employed, giving policy makers and traffic engineers some control over the objectives.

Locally, the traffic-adaptive SPOT controllers are made up of an observer and a controller. The observer performs an optimization roughly every 3 seconds, thereby updating information on traffic counts and signal states. The controller optimizes signal timings every 6 seconds, such that the optimal timings are only in effect for 6 seconds. After each optimization, the controller shares the information with its neighbours. Because the controller considers a 120 second time horizon during the optimization, it is able to consider the effects on future cycles and downstream intersections. In small networks, the local SPOT system suffices, but for networks upwards of 6 intersections, the central UTOPIA-module should be added (van Katwijk, 2008).

Centrally, the same observer and controller structure is present. The central observer optimizes (recalculates) every 3 minutes the information on flows and route choice. At the same time interval, the central controller optimizes weights to be distributed to local intersections based on network-wide indicators like average speed or saturation flow. Special plans of action are in place when over-saturated conditions arise, such as increasing throughput or decreasing demand for certain directions.

During operation, the system tries to gather as much data as possible, for example turning fractions, traffic volumes, travel times and saturation flows. This data is processed by the observers and used in future optimization cycles to account for slow time-varying parameters. This way of 'learning' from the traffic conditions improved the performance of UTOPIA-SPOT substantially, when performance from 1985 was compared to performance from 1986 (Mauro & Di Taranto, 1990).

2.1.7. TUC

Traffic-responsive Urban Control (TUC) is a coordinated traffic controller that models the urban network traffic based on a store-and-forward modelling (Diakaki et al., 2002). The modelling technique uses a simplification to model the outflow of an approach, which sets the maximum optimization frequency of real-time decisions to once every cycle. However, the simplification allows the model to avoid the use of binary variables, which opens the way to application of highly efficient optimisation methods, such as linear programming, with polynomial complexity (Diakaki, 1999). This efficiency, in turn, enables TUC to operate on large networks in real-time. TUC has been successfully applied in practice in multiple networks in Europe (Condie et al., 2004). More recent studies have incorporated actuation in TUC (Manolis et al., 2018), which allow the traffic controllers to be able to respond to intra-cycle variations of the local traffic situation. This system combines the central architecture with some local fine-tuning.

2.1.8. VAN KATWIJK

In van Katwijk (2008) a hierarchical, distributed, adaptive coordinated controller was developed.

This approach sees the intersection controllers or even other traffic controllers, like ramp metering installations, as agents. The network is divided into sub-networks, each with their own set of agents. The optimization problem is formulated as a dynamic programming problem, on the premise that the optimal solution to the overall problem can be found by combining the optimal solutions to the various (smaller) sub-problems. The search algorithm is further optimized by employing a notion from Branch-and-Bound algorithms: It remembers current upper and lower bounds such that any solution directions that do not improve on these bounds are not evaluated. In van Katwijk (2008), greens per direction (movements, not stages) are the smallest controllable unit. This allows the controller to effectively optimize the phase order by skipping green for a direction, if there is no demand. To limit computational complexity, blocks are used, instead of stages, to ensure that conflicting directions are not given green simultaneously. Coordination is not forced: The agents communicate and can coordinate their controls, when they see fit. Coordination between the various agents is achieved on a microscopic level only if agents are close enough according to a rule of thumb. Coordination is always considered on a macroscopic level, mainly in the form of 'gating': letting agents communicate about their in- and outflow. Communication about flows is combined with capacities and a cost is associated based hereon that is then included in the optimization of each agents' control decisions.

2.1.9. VARIOUS OTHER SYSTEMS

There are various other coordinated traffic controllers, besides TopTrac, that are used or have been used in the Netherlands. Often, since they are commercial products, little detail is known about the workings of these controllers. However, it is still important to be aware of alternatives used in practice.

One of these other coordinated traffic controllers used in the Netherlands is the imFlow controller by Dynniq (Dynniq, n.d.). imFlow, as reported by Dynniq, does not require a central server. This server is optional, to allow for performance evaluations. The coordination works on a local level, where optimization is carried out every second. The adaptive controller includes a queue algorithm, to estimate current capacity and a real-time estimation of the saturation flow, to make the controller robust against disruptions (like incidents or weather conditions). Using a variety of weights in the model of the controller, imFlow is able to optimize the flow according to specific policies of the road authority.

Another coordinated traffic controller is FlowTack by Royal Haskoning DHV (DHV, 2022). Flowtack is a central coordinated traffic controller, which is able to take into account various other data sources, besides loop detectors. Data from navigation apps and connected vehicles is used as well.

Another controller previously used in the Netherlands is Sitraffic Motion (previously

MOTION) by Siemens. Sitraffic Motion can be installed with the Sitraffic Scala online traffic management platform of Siemens. Sitraffic Motion is an adaptive coordinated traffic controller. The original MOTION controller used traffic flow counts to switch between predefined signal timing plans. These plans were optimized for different conditions, for example different turning rates. This switching between predefined plans is also an option in Sitraffic Motion. Sitraffic Motion works on two different levels:

1. On the tactical level, where once every 3 to 30 minutes the cycle time, average green-time split, basic phase sequence and network coordination are determined

2. On the operational level, where once every 60 to 140 seconds the cycle time and current phase sequences may be further adjusted (slightly). On a per-second basis, the current green may be extended or terminated.

2.1.10. SUMMARY

In Table 2.2 an overview is given of the discussed coordinated traffic controller with regards to their architecture. The types of architectures are explained in section 1.1.3.

Table 2.2: Overview of different architectures of real-time coordinated traffic controllers in literature

Architecture		
Central	*Local*	*Combination*
SCOOT, SCATS, TopTrac, Flowtack, TUC, MOTION	PRODYN, SPOT, Imflow, ALLONS-D, Back-Pressure based controllers (Salomons, 2021a)	TopTracFlex, UTOPIA-SPOT, RHODES, van Katwijk (2008), VFC-OPAC

This table is relevant to the proposed variable speed as, dependent on the architecture, the variable speed can only be used for certain applications. In the discussed systems with a local architecture, the idea of a variable speed to coordinate with different cycle times may be possible. In contrary, it is unlikely that a real-time version of the variable speed can be used in the discussed systems with a local architecture to allow coordination between (locally) more efficient signal timing plans in two directions. This is because this application of the proposed variable speed requires a complete overview of all signal timings of the coordinated directions in the arterial. Changing the speed has effects on downstream green timings of the whole arterial and on top of that, local TLC's need to agree which speed is applied on the segment. Regardless of the architecture, improvements of the base signal timing plans can be applied to all systems. None of the literature surveyed regarding the various coordinated traffic controllers report any usage of variable speeds. All controllers use equal cycle times, none of the surveyed systems state that research was done into coordination between unequal cycle times. The same can be said about the coordination of the starts of green in two directions.

2.2. VARIABLE SPEED LIMITS

There is a vast body of research concerning variable speed limits (VSLs) on freeways. Usually, this speed limit is communicated via road-side variable message signs, but lately research has also included in-car communicated speed limits in a connected vehicle environment. Research regarding VSLs on freeways has shown both in theory and practice that these VSLs as a controllable unit, are able to control traffic to mitigate the negative side-effects of congestion (D. Frejo & de Schutter, 2018). One thing to take away from VSL research on freeways is that a controllable, lower speed may provide benefits over an uncontrollable, higher speed, however mind that the VSL control problem and the urban traffic signal control problem are dissimilar in many other ways. For example, differences are the presence of conflicts, turning directions, signals and different (spatial) scale.

The application of VSLs on freeways further differs from the idea of applying a variable speed to gain more design freedom in coordinated signal timing plans on urban arterials. The former is mainly aimed at reducing negative consequences of traffic jams when they have occurred or are about to occur, while the latter is aimed at improving design of the timing plans in coordinated controller prior to any congestion happening. Because of the large differences, this literature survey will not include detailed research of VSLs. The concept of VSLs can be brought to the urban arterial and this is done in Othman et al. (2020). The method developed in Othman et al. (2020) is able to use control of speed limits in an urban network, to improve overall travel time, in cases where congestion occurs. Main advantages are reported in cases where the VSLs were able to prevent a gridlock from happening.

In Levin et al. (2019), variable speed limit (VSL) research is combined with connected vehicle (CV) research in an attempt to improve safety on and near urban intersections. This research also uses the idea that vehicle speeds can be controlled for certain benefits. In Levin et al. (2019), this control was used to improve safety, by reducing the deviance of shockwave occurrence. Improvements regarding energy savings were also shown, despite increased average travel times. Potential of the variable speed for traffic signal coordination was not investigated in this paper.

2.3. CONNECTED VEHICLES AND COORDINATION

Connected vehicles (CVs) offer many opportunities, also for coordinated traffic control. Research into CVs is relevant for the proposed variable speed research as both consider exploiting the potential of communication between vehicles and TLCs. In this literature survey, the opportunities for traffic control are split into two categories:

1. **Improved information supply to TLCs.** CVs send data to infrastructure elements, for example a TLC. This data can be used on top of current loop detector data to improve the information supply to the TLC, thereby allowing the TLC to make a more informed control decision.

2. **Improved information supply to traffic.** CVs can receive data from infrastructure elements, for example a TLC. This can be used to provide the CV with for example data about the expected Time To Green (TTG) or Time To Red (TTR). Furthermore,

this data may include a new speed limit or speed advice, which is of course very relevant for potential applications of the research in this thesis.

CV's can also exchange information with each other, but this particular aspect is not considered in this survey, as the focus lies on the relation with the traffic signal controller.

2.3.1. Improved information supply to TLCs

Improved information supply (typically in the form of Floating Car Data), can improve the traffic state estimation on urban signalized links. This is done in Rostami Shahrbabaki et al. (2018) and Rostami-Shahrbabaki et al. (2020). Their estimation, which produces good results even for a penetration rate of 10%, splits the link into two, where the split point is defined by the tail of the queue. This approach allows for accurately keeping track of the tail of the queue, which could be used in a coordinated traffic controller to provide better coordination for cases with a standing queue. The standing queue namely should be cleared just as the platoon arrives, which is hard to time when the size of the queue is unclear. For the proposed variable speed to be used in a practical coordination, the speed should lead to coordination and not lead vehicles into a queue, which makes accurate estimation of the queue length valuable.

In a recent literature review (J. Wang et al., 2021), the potential of improved signal control under a CV environment was highlighted. The review states the current issues in research and identified a research gap regarding the application for coordinated traffic control. Issues are mainly related to different penetration rates and unknown development of these rates, as well as the expectation that traffic flow characteristics will change. The review focused on the improved information supply to the traffic signal controller, which should be able to have a better perception of the traffic. These improvements should lead to better coordination, but this is insufficiently researched.

2.3.2. Improved information supply to traffic: Speed advises

In terms of the improved information supply to traffic, only speed advises are considered in this survey. These most closely relate to coordination with a variable speed, in the sense that the speed of drivers is influenced for some sort of performance gain.

Green Light Optimal Speed Advisory (GLOSA) is defined as a vehicle system that receives upcoming traffic signal cycle information over V2X communication channels and uses relative vehicle position to compute and display a speed recommendation that, if adopted by the driver, would allow the vehicle to pass the upcoming traffic lights during a green interval, thereby reducing stops at red lights (The British Standards Institution, 2020).

GLOSA is broadly researched in modern literature (Asadi et al., 2021). From it's definition and various research papers like (Typaldos et al., 2020) and (Sharara et al., 2019) it becomes clear that GLOSA refers to something quite different than the research of this thesis. Namely, the GLOSA problem takes the known or somewhat known signal timing plans as input and tries to provide the best speed advice, given this information. This differs from the thesis in the sense that the speed is not used to create more effective signal timing plans for coordination.

GLOSA works on a smaller time scale and spatial scale (microscopic), where TTG and TTR predictions (or information) are used to provide individually optimized speed advises. The GLOSA literature that is included in this survey will only be literature that is in some way more related to the coordinated traffic control problem. For example literature that studies GLOSA in a network setting or includes a feedback from the advice (vehicle) back to the controller and it's decisions regarding the design of timing plans.

A lot of GLOSA research has focused on isolated intersections. However, also a multitude of papers focused on multiple intersections. For example in Seredynski et al. (2013), a multi-segment version of GLOSA was formulated. Their formulation required fixed signal timing plans as input, however these plans didn't need to be coordinated according to a fixed speed. Considering multiple segments at once, instead of each one separately, allowed for minimization of the speed differences and thereby reducing fuel consumption. Still, in this research, the coordination of the signal timing plans is not improved based on the speed, rather a variable speed (advice) is used to improve the progression of vehicles through predetermined (fixed speed) plans.

Some papers do not consider GLOSA specifically, but a more general speed advice. In Tang et al. (2018), a speed advice, based on a car-following model, is developed that can be used to determine optimal fuel-efficient trajectories across multiple intersections. This type of research, where (models for) optimal trajectories are sought, is quite common in research that considers urban arterials in a CV-environment. Other studies are for example: (W. Chen et al., 2015), (Sun et al., 2013), (Ye et al., 2019) and (Feng et al., 2019). Noticeably, a common theme in these papers is the goal of fuel consumption minimization. Again, the models assume some info on timing plans to be able to plan the trajectories in time. Any changes to coordinated traffic controllers will affect these types of models. The rebound effects that these models have on the controller is not considered. This would be something more closely related to the research in this thesis, where the effects of a controllable speed (though not individually controllable) on the traffic controller are considered.

An approach that, unlike GLOSA, does consider the signal controller is presented by Wu et al. (2019). In their optimization model, not only are vehicle speeds treated as variables, so is the green time window in the signal controller. The model is compared against a case with a fixed signal timing plan that only optimizes the trajectories and against an actuated signal timing plan that doesn't optimize trajectories. Significant benefits in terms of reduced delay, over both cases, are obtained by the proposed optimization model. However, this research didn't consider multiple intersections and thus didn't consider using the the variable speed to improve the coordination among signal timings plans.

The vast amount of research, experiments and already a few currently functioning systems regarding GLOSA, indicate that GLOSA may be not too far from becoming a more wide-spread reality. This is a relevant fact considering the research of this thesis. When communication infrastructure for GLOSA is fully functional, it may only be one more step until the speed advice could become a controlled speed. Then, adapting signal controllers to be able to use this controlled speed when constructing their timing plans is a small step. Even when a fully controlled speed is not available, adaptations to the signal controllers may be relevant, assuming a certain rate of compliance.

2

2.4. TRAFFIC DEMAND PREDICTION

Research into (short-term) traffic demand predictions based on forecasts can be dated back to the 1980s. A brief historical overview is given in Vlahogianni et al. (2014). Over the years, parametric, Time-Series forecasting methods like exponential smoothing, the Auto-Regressive Moving Average (ARIMA) and the Holt-Winters method have been used for forecasting traffic demand. More recent studies research non-parametric methods like Neural Networks, fuzzy logic methods (van Hinsbergen et al., 2007; Vlahogianni et al., 2014).

While both research into predictions and research into traffic control have become quite advanced, they are rarely combined according to Jiang et al. (2021). Real-time signal control uses demand as an input for making the control decision. However it can not be assumed that by providing predicted demands, even if done quite accurately, the traffic signal controller performance will increase. Different saturation levels, the role of errors in the prediction and especially the behaviour of the coordinated traffic controller play a role regarding the usefulness of working with demand predictions.

Poelman et al. (2020) researched the sensitivity of predictive (cyclic vs. structure-free) controllers to over- and underestimations regarding different variables (queue, arrival and departure) under varying saturation levels. Their research, conducted on an isolated intersection, showed that the structure-free controller outperformed the cyclic controller in all scenario's, showing that the flexibility in the control scheme is sufficient to overcome the higher dependency on (erroneous) input data. Most importantly, the research also showed that using predictions improves performance (in most cases), despite errors in input data. Generally, the saturation level has a big influence on the importance of errors. In under-saturation, arrival and departure errors cause bad performance, to the point that it would be better if no estimation is used, but queue errors have little effect (there are barely any queues). In over-saturation, errors in any variable have a (relatively) low effect on performance. Underestimations of (the number of) departures leads to great performance loss of the controllers. Errors in queue length and arrivals can compensate each other, making them less negative on overall performance. Predictions of departures should therefore preferably be overestimated than underestimated.

In traffic control literature there exist very advanced controllers, like the Model Predictive Controllers (MPCs) discussed in Jeschke (2020), Jansen (2016), Lin (2011), Lin et al. (2012) and Jamshidnejad et al. (2017). These controllers make demand predictions, though slightly different than those that are tested in this thesis. MPCs make predictions about the evolution of demand throughout the network, by using models. Given a certain initial state, MPCs model the propagation of traffic (including the effect of control decisions) to allow the planning of future control decisions based on (predicted) demand figures. The literature states that the initial estimates and later corrections of the demand in these MPCs can be based on measurements, prediction models and/or historic figures. MPCs generally show benefits in terms of traffic performance (like reduced stops or delay), but are not applied in practice or even state to be computationally infeasible for applications in practice. The (promising) results of MPCs indicate that more accurate information of demand can improve controller performance. The survey of MPCs has given no reason to assume that similar performance improvements cannot be realized in TopTrac.

2.5. CONTROLLABLE VARIABLE SPEED

Only a few papers were found in literature that more closely resemble the variable speed research direction proposed by this thesis. This section will explore these papers in detail and explain the differences with the proposed research.

Firstly, the research done by van Leersum (1985) is discussed. In their research, it was shown that a modified version of TRANSYT, that could include a variable speed, was able to produce benefits in terms of reduced fuel consumption and stops. Benefits increased monotonically with increasing compliance.

The way the variable speed is implemented in (van Leersum, 1985) is as follows. TRANSYT shifts the offsets in it's optimization procedure, then the speed is determined by simply checking if the current offset, combined with the link length, leads to a speed that is acceptable according to the limits (i.e. between vmin and vmax). If the speed corresponding to the offsets is acceptable, then the progression of flow is calculated in a different way. Namely not the typical dispersion and travel time model of TRANSYT is used, but the flow pattern of complying vehicles is simply shifted with the offset value. This way, more sharp-edged flow patterns are obtained that finally lead to improved values in the performance index. This way of modifying the TRANSYT program is a smart way to trick the program into finding offsets that correspond to an acceptable speed, but does not guarantee that an acceptable speed is found/used. What this method does not consider, is the possibilities for a variable speed to allow different phase orders in the signal timing plans nor the interactions with the cycle time (only a fixed cycle time is considered).

Regardless, the benefits shown in terms of fuel consumption indicate once more the potential such a variable speed may bring.

Another paper worth discussing in detail is the paper by De Nunzio et al. (2015) who formulate a progression-based MILP (inspired by the LP problem formulated in (Gomes, 2015), which itself takes some inspiration from MAXBAND) model that maximizes bandwidth and minimizes fuel consumption by optimizing offsets and speeds. Their paper does assume full compliance to the variable speed limit and shows that reductions in fuel consumption are obtained, while travel times are not affected in a negative way.

While the formulated problem has similarities with MAXBAND, there are some differences. For one, the cycle time is not treated as a variable, instead it is an input. Furthermore, a different objective function is used, that, besides maximizing bandwidth, strives to minimize travel time and speed difference on consecutive segments. The latter two are given weights that allow to give more preference to either one.

The model with variable speed limits is compared to one without. A theoretical analysis, 13000 model runs with varying input parameters, shows that the model with variable speed limits always produces more bandwidth. The final part of the paper motivates why additional bandwidth is beneficial for the traffic in terms of characteristics like fuel consumption. The paper does not explore varying cycle times nor does it look at varying phase order, meaning that it does not state how the variable speed relates to those.

2.6. SUMMARY

The survey of various coordinated traffic controllers has provided more insight in the workings of these controllers and their characteristics. Controllers with a local architecture are not suitable to include a real-time version of the variable speed for the goal of improving coordination between the starts of green in two directions. This application requires knowledge of the timing plans of all controllers in the network, which is only available in coordinated traffic controllers with a central architectural component. Most importantly, the survey found that, of the two most commonly used (and available) models for creating coordinated signal timing plans, MAXBAND and TRANSYT, MAXBAND is the most suitable for optimizing with a variable speed.

Various variable speed technologies, like freeway variable speed limits and speed advises in a connected vehicle environment (among which GLOSA) were surveyed, to understand how they relate to the proposed variable speed solution. Differences mainly occur when looking at the underlying goal of the variable speed: none of the technologies investigate the potential benefits for improving coordination by constructing different signal timing plans. Freeway variable speed limits share some resemblances regarding the macroscopic character, but are applied in different environments and for different reasons. GLOSA-like techniques are similar in that they are applied in signal control, but they are of a microscopic nature and often take the signal timing plan as (given) input.

A survey of demand prediction research indicated that this research is rarely combined with coordinated traffic signal control research. Understanding how a coordinated traffic signal controller reacts to being supplied with a prediction is a very relevant research topic. Advanced controllers in literature show performance improvements when models are used to provide the controller with a more accurate view of (future) demand.

Some papers, like De Nunzio et al. (2015), are more similar to the variable speed as proposed in this research. This paper has shown that, in the model used in the paper, a variable speed consistently leads to more bandwidth of the coordination. The specific uses of the variable speed, as discussed in section 1.2.2, are not treated in this research.

Overall, coordinating with equal cycle times is presented in every research as a given. While it is logical, there are no studies that present any figures that motivate this assumption. This thesis could fill this gap by looking at coordination with unequal cycle times in a variable speed environment, which will give insight into a fixed speed environment as well. The feasibility of coordination in two directions is another topic, about which no theoretical analysis was found in literature. It can be concluded that a literature gap exists regarding the variable speed ideas discussed in section 1.2.2.

3

Research questions and methodology

On the basis of the introduction and the literature survey, this chapter presents the research questions. Next, the methodology for answering these questions is laid out.

3.1. Research questions

Both research directions share the same goal of reducing stops and delay in the network controlled by the coordinated traffic controller. This goal is motivated in section 1.2.1. Other than this goal, the research directions are too dissimilar to capture in one research question. Therefore, the choice is made for two main research questions, subdivided according to the two research directions. Firstly, the proposed variable speed is discussed. Secondly, the demand predictions in TopTrac are discussed. This order is maintained throughout the report.

Variable speed

The literature survey concludes a clear research gap regarding the usage of variable speed as proposed in this research in section 1.2.2. The main goal of this thesis is to contribute to filling this gap. As a reminder, the variable speed that is considered in this research will not exceed the maximum speed (speed limit), meaning that a variable speed also means a lower permitted speed than normal. The reason for this decision is that applying a higher speed than the speed limit is seen as unsafe. Furthermore, the assumption is made that there is full compliance to the proposed variable speed. Under these assumptions, this thesis will investigate the potential advantages in achieving coordination under certain under-saturated demands. These advantages are sought in the creation of coordinated signal timing plans. The variable speed will be considered to vary both in space as well as in time. Variations in space are variations per road segment between intersections as well as per direction.

The first main research question is:

> *What are the benefits, for the coordination of traffic controllers, in terms of reduced stops and delay, when optimizing signal timing plans with variable speeds compared to fixed speeds in an undersaturated demand scenario?*

To answer the first main research question, the following sub-questions should be answered:

1.1 To what extent does applying a variable speed theoretically allow different cycle times per intersection for the coordination of signal timing plans in one direction?

1.2 In which cases does applying a variable speed theoretically allow the coordination of locally optimal signal timing plans in two direction, compared to a fixed speed?

1.3 How can the variable speed be included in a model that finds the optimal variable speeds to optimize the coordination?

1.4 What are the benefits, in terms of reduced stops and delay, of applying variable speeds to coordinated pre-timed semi-fixed signal timing plans?

TESTING THE POTENTIAL OF DEMAND PREDICTIONS IN TOPTRAC

Based on the desires of Vialis and literature found in the literature survey, it is interesting to investigate the potential benefits that demand predictions may bring in TopTrac. It is unknown how the performance of TopTrac will be affected in response to implementing demand predictions. And to what extent this leads to a reduction in stops and/or delay. Moreover, it is unclear how tests of the potential of demand predictions can be included in TopTrac, in a technical sense. In the literature survey it was found that the behaviour of coordinated traffic controllers using demand predictions is not an extensively researched topic.

The second main research question is:

> *For TopTrac, what are the maximum benefits, in terms of reduced stops and delay, obtainable by using near-perfect demand predictions compared to normal operations in undersaturated conditions?*

To answer the second main research question, the following sub-question should be answered:

2.1 How can tests of the potential of demand predictions of undersaturated demand be included in TopTrac in real-time?

2.2 Supposing that the (undersaturated) demand is known and TopTrac gets a 'perfect' prediction, what are the gains in terms of reduced stops and delay for a typical morning rush hour?

3.2. METHODOLOGY

This section will explain the methodology that will be applied in this research to answer the research questions.

STEP 1A

In this step, it is investigated whether, by using the variable speed, coordination in one direction under different cycle times may be feasible. To this end, an equation is formulated that describes the evolution of the offset between the starts of green of a coordinated direction between two intersections with different cycle times. This equation is then solved and evaluated for both the variable speed and the fixed speed, in terms of relative and absolute differences. This step delivers the answer to research question **1.1**.

STEP 1B

In this step, it is investigated when the variable speed allows for a feasible coordination between the starts of green in two directions, compared to a fixed speed. To this end, a set of equations is formulated that describe the conditions for a coordination between the starts of green in two directions. Three different cases, related to different signal timing plan characteristics, are considered first. The cases are formulated as the simultaneous start of coordinated green windows, the nearly simultaneous start of coordinated green windows and finally no constraint on the start of green windows. And these are assumed to apply to all intersections of the arterial. After considering these cases, a generalisation is made that allows an arterial with any combination of the aforementioned cases. This step delivers the answer to research question **1.2**.

STEP 1C

In this step, the inclusion of a variable speed in a coordinated traffic control model is explored. Literature does provide some useful base here: The progression based methods like MAXBAND and MULTIBAND are capable of including a variable speed. However, the methods were not designed specifically to produce the best coordination under the assumption that a variable speed is available. Rather, the methods are designed to produce a maximum progression and can thereby use a variety of different speeds to find different offsets. Therefore, this step in the research will take the MAXBAND model as starting point and will further explore ways to optimize with the variable speed using this model. This step produces the answer to research question **1.3**.

STEP 1D

In this step, a simulation study is performed to analyze the benefits of a variable speed compared to a fixed speed for a specific case. The case will consider an urban arterial in Almere, namely the Havendreef-Stedendreef arterial. Simulations will be performed with VISSIM. The network and the reasons for using VISSIM are explained in chapter 7. Variable speed simulations will include several simulation with coordinated pre-timed semi-fixed signal timing plans generated by the model. These simulations are benchmarked against simulations that use timing plans created through the same methodology, but with a fixed speed. The only difference between the two simulations will be a different lower speed limit. This step answers research question **1.4**.

STEP 2A

The TopTrac coordinated traffic controller is analyzed and a suitable way to supply the tests of the potential of demand predictions is found. A test-environment is created where the inputs of TopTrac can be influenced, such that the desired tests can be performed. This analysis provides the answer to research question **2.1**.

STEP 2B

A reference simulation with TopTrac is conducted and the inputs to TopTrac and the traffic flow measurements from the simulation are saved. A follow-up simulation is conducted with the same random seed, such that the demand is exactly the same. In this follow-up simulation, the inputs supplied to TopTrac in the reference simulation are shifted forward in time by one optimization cycle. This test should supply the 'perfect' prediction in the sense that exactly the amount of demand is seen by controller as was affected by the control decision from the controller in the reference simulation. The idea of this prediction test is to change the control decisions from reactive to proactive. Data from the simulations is saved and compared on stop and delay performance indicators. This step provides the answer to research question **2.2**.

4

THE VARIABLE SPEED CONCEPT

This chapter will explore the concept of a variable speed in coordinated traffic control, on a theoretical basis. In section 4.1 a practical limitation regarding coordinated traffic control is put to the test. Namely, the assumption that coordination can only occur with equal cycle times. This is done because it should not be assumed that this limitation holds for the variable speed as well. To show the differences with a fixed speed, the same in depth look at coordination with different cycle times is also given for the fixed speed. Section 4.1 gives the answer to research question **1.1**: *To what extent does applying a variable speed theoretically allow different cycle times per intersection for the coordination of signal timing plans in one direction?*

Theoretically, the variable should be able to assist in the challenge of coordinating in two directions. For coordinating in two directions, a geometric challenge arises when the start of green of the coordinated directions overlaps. This challenge is investigated in section 4.2, to show if this theory holds. Section 4.2 gives the answer to research question **1.2**: *In which cases does applying a variable speed theoretically allow the coordination of signal timing plans in two direction, compared to a fixed speed?*

Both of these ideas can be summarized as follows: The variable speed may be used to coordinate between more efficient local signal timing plans. With efficient, in this thesis, it is meant that the cycle time and green times are as low as possible to serve the demand. Increasing the cycle time on some intersection to be equal to a network cycle time is not ideal on a local level. Hence the expectation of benefits, in terms of stops and delay, when coordination between unequal cycle times may be possible using a variable speed. Similarly, in two directions, assuming efficient timing plans have (nearly) simultaneous starts of outbound and inbound coordinated green windows, coordination is not always possible between efficient timing plans. Hence the expectation of benefits, in terms of stops and delay, when the coordination in two directions between efficient timings may be possible using a variable speed.

Firstly, a brief reminder of the technical details of the proposed variable speed, as these will be used throughout this chapter. The variable speed is allowed to vary per direction (inbound versus outbound), per segment and through time (on a per cycle basis), but is always constrained in both directions according to Equations 4.1 and 4.2.

$$v_{min} \leq v_{\hat{i},n} \leq v_{max} \tag{4.1}$$

$$\text{with} \quad v_{min} = l * v_{max}, \quad 0 < l \leq 1 \tag{4.2}$$

Depending on the chosen lower bound, the maximum speed and the length of the segment, the variable speed can be used to supply a certain range in offsets. The lowest offset is provided when the segment is traversed with the highest speed. The highest offset is provided when the segment is traversed with the lowest speed. The difference between the lowest and highest offset represents the maximum range in time the variable speed can provide for the given segment. This maximum time difference is given by Equation 4.3.

$$\Delta T_{\hat{i}} = \frac{d_{\hat{i}}}{v_{min}} - \frac{d_{\hat{i}}}{v_{max}} \tag{4.3}$$

Where:

$v_{\hat{i},n}(\overline{v_{\hat{i},n}})$ = The outbound (inbound) variable speed between intersection i and i+1 in cycle n [m/s]. Equations 4.1, 4.2 and 4.3 hold for the inbound variable speed as well.

l = A factor of the maximum speed used to denote the lower bound of the variable speed $0 < l \leq 1$ [-].

$d_{\hat{i}}$ = The distance between intersection i and i+1 [m].

v_{min}, v_{max} = The minimum, maximum speed allowed on any segment, in any cycle [m/s].

$\Delta T_{\hat{i}}$ = The maximum time difference allowed by the minimum and maximum speed and distance on segment $\hat{i}$ [s].

Very low speeds, for example of 1 kilometres per hour, are not realistic driving speeds in practice. This implies that for practical applications of the equations some lower bound l_{min} exists. The formulas presented in this thesis hold for any l_{min} greater than 0. However, throughout the chapter, a value for l_{min} of 0.6 is used, just to keep conclusions regarding the feasibility of coordination with the variable speed realistic. This choice is somewhat arbitrary, but anyone can plug in any value in the equations and draw their own conclusions. The value of 0.6 leads to a lower bound of 30 kilometres per hour for the upper bound of 50 kilometres per hour, which are seen as practical values.

4.1. COORDINATION IN ONE DIRECTION

Coordination of traffic signals in one direction is a lot less restrictive than in two directions. Typically, coordination is performed under equal cycle times. When coordinating in one direction under equal cycle times, there is no restriction with regards to the position of the coordinated green in the signal timing plan. The inbound green can be situated at any place in the signal timing plan that is optimal for the local demand, because there is no coordination for the inbound direction. The proposed variable speed will, in these cases, not be able to improve the signal timing plan design for coordination. One idea, that multiple directions downstream may be coordinated with one upstream direction, is outside of the scope for this thesis, as mentioned in section 1.2.4. Another idea is that the variable speed may be used to coordination with unequal cycle times. When the restriction of equal cycle times across the network is not needed, all the disadvantages, mentioned in CROW (2014), of a too large cycle time to serve the local demand can be avoided.

4.1.1. COORDINATION WITH DIFFERENT CYCLE TIMES

In this section we consider that different intersections are controlled with different cycle times, not that one intersection will over time be controlled with different cycle times. In Donaldson (2022) time-varying cycle times are used to increase the flexibility of transit signal priorities in coordinated systems. While time-varying cycle times certainly are interesting, they still require (on average) an equal network cycle time. No literature could be found regarding coordination under different cycle times. The only exception being that half the network cycle time is allowed in some models, like TRANSYT (Wilson, 1993). This section investigates whether the variable speed will be better suited for coordination between the starts of green under different cycle times than the fixed speed.

To coordinate the start green times of a direction, when assuming the green time downstream is given some time after the green time upstream, Equation 4.4 applies. Assuming time-invariant cycle times on intersections i and i+1, the start green times per intersection are determined according to Equations 4.5 and 4.6.

$$T_{off,\hat{i},n} = SG_{i+1,n} - SG_{i,n} \tag{4.4}$$

$$SG_{i,n} = SG_{i,0} + n \cdot C_i \tag{4.5}$$

$$SG_{i+1,n} = SG_{i+1,0} + n \cdot C_{i+1} \tag{4.6}$$

Where:
n = An integer number of cycles, with $n \in \mathbb{N}^0$ [-].
$T_{off,\hat{i},n}$ = The offset on segment $\hat{i}$ and in cycle n required for coordinating in the outbound (inbound) direction [s].
$SG_{i,n}$ = The start green time of the coordinated direction at TLC i in cycle n [s].
C_i = The upstream cycle time, as an integer, $C_i \in \mathbb{N}^+$ [s].
C_{i+1} = The downstream cycle time, as an integer, $C_{i+1} \in \mathbb{N}^+$ [s].

Note that in the definition, it is assumed that cycle times are integers. The assumption that the cycle times are integers is sensible. From Vialis it is known that typically, traffic light controller software includes integer cycle times. In section 4.1.2 a reflection is made upon this assumption. From the definition of the cycle times, any positive integer greater than zero is allowed. Very short or very long cycle times, of for example 2 seconds and 2000 seconds, are not realistic in practice. This implies that there exist some lower and upper bounds $C_{\min}$ and $C_{\max}$ on a practical cycle time. The equations presented in this thesis make no assumption regarding these bounds. A value for the lower bound of 30 seconds is used throughout this chapter, just to keep conclusions regarding the feasibility of coordination realistic. Cycle times below 30 seconds are not practical, at least in the Netherlands (M. Barto, 2022, personal statement).

Consider the first cycle ($n = 0$). Setting $SG_{i,0} = 0$ in Equation 4.4, we obtain $T_{off,\hat{i},0} = SG_{i+1,0}$. The start of green of the coordinated direction of the first intersection is given at the start of the cycle. Combining this fact with Equations 4.4, 4.5 and 4.6, we can express the offset for coordination in terms of the cycle times of each intersection as in Equation 4.7.

$$T_{off,\hat{i},n} = (C_{i+1} - C_i) \cdot n + T_{off,\hat{i},0} \tag{4.7}$$

From Equation 4.7, it can be seen that when cycle times C_i and C_{i+1} are not equal, the offset will be subject to change over different cycles. A fixed speed thus immediately implies that cycle times must be equal to provide coordination in every cycle, namely only one speed can correspond with $T_{off,\hat{i},n}$. From the same equation it follows that the cyclic time-varying offset, could be allowed by the cyclic time-varying speed. Namely, this may be the case when the variable speed can correspond with all $T_{off,\hat{i},n}$ for all n. However, since the cycle times themselves are time-invariant, the offset will drift, ever increasing or ever decreasing. This means that the required speed for coordination will be ever decreasing or increasing, respectively.

Now consider the case where $C_{i+1} > C_i$, then the offset will continually increase according to Equation 4.7. Coordination under different cycle times, with $C_{i+1} > C_i$, is depicted in Figure 4.1. The choice to consider the case that $C_{i+1} > C_i$, that the downstream cycle time is larger than the upstream cycle time, instead of the other way around, is made arbitrarily. In section 4.1.2 a reflection is made on this choice and the differences with the $C_i > C_{i+1}$ case.

It is assumed that we provide the first coordination (at $n = 0$) with maximum speed ($v_{\hat{i},0} = v_{max}$). This assumption is also discussed in section 4.1.2. We can use Equation 4.7 and write it as Equation 4.8, to give the variable speed required for coordination in any cycle. Equation 4.8 reflects the logic that, for the considered case, with an increasing number of cycles, the speed required for coordination will continually decrease.

$$v_{\hat{i},n} = \frac{1}{\frac{(C_{i+1}-C_i)\cdot n}{d_{\hat{i}}} + \frac{1}{v_{max}}} \tag{4.8}$$

The constraint in Equation 4.1 can be translated, using Equation 4.3 and Equation 4.8, to Equation 4.9, showing for how many consecutive cycles n, the constrained variable speed can provide coordination for different cycle times. Equation 4.9 indicates for how many cycles n, the speed required for coordination is below the lower bound of the variable speed. Equations 4.1.1 and 4.9 only give sensible answers regarding the first few cycles.

$$n \leq \frac{\Delta T_{\hat{i}}}{(C_{i+1} - C_i)} \tag{4.9}$$

When time goes on, because the two cycles have different length, after some number of cycles, their start moments will coincide again. For example the 50th cycle of C_i, the

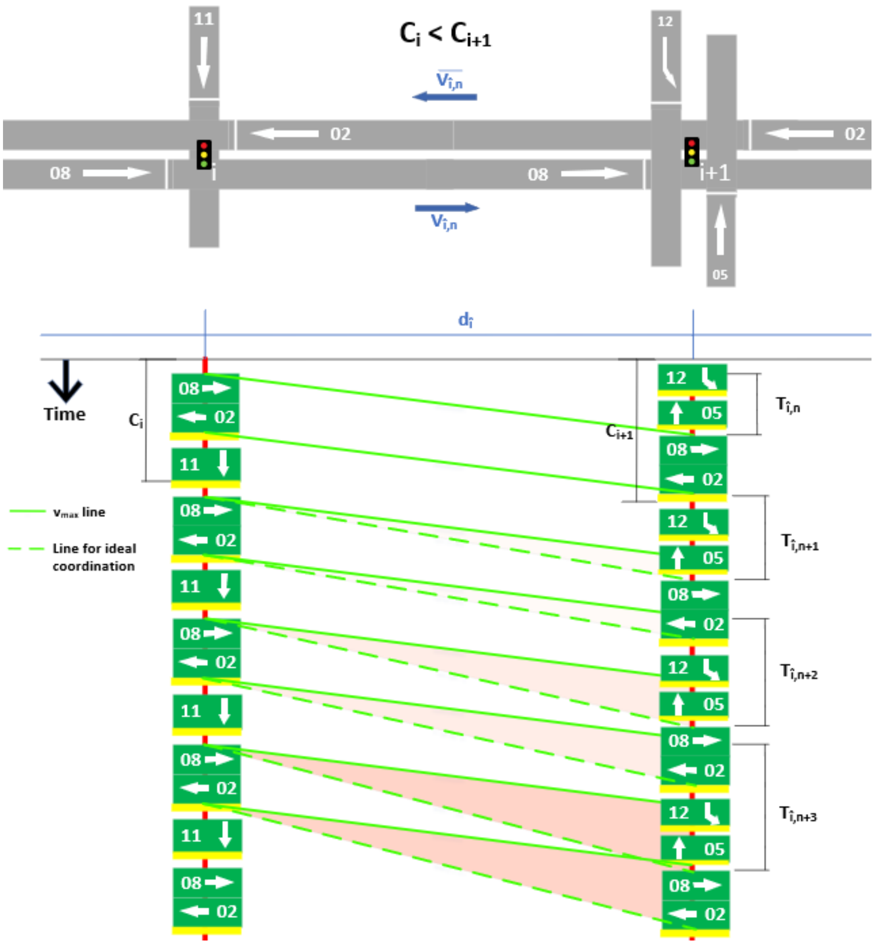

Figure 4.1: Fictitious network and signal timings showing coordination under different cycle times

upstream intersection with smaller cycle time, could start at the same time as the 49th cycle of C_{i+1}. When this happens, it is not logical to try to coordinate the 50th upstream cycle with the 50th downstream cycle. When the 50th upstream cycle is coordinated with the 49th downstream cycle, 49th upstream cycle surely can't be coordinated. The coordination of this cycle at this intersection is skipped. In essence, we are skipping an upstream cycle to lower the offset. By skipping an upstream cycle, the offset is lowered by C_i (for the upstream cycle time) and $(C_{i+1} - C_i) \cdot 1$ (for skipping 1 n). Skipping a cycle thus results in a reduction of $-C_i - (C_{i+1} - C_i) \cdot 1 = -C_{i+1}$ on the offset. As the first cycle is coordinated with the maximum speed, and the speed in subsequent cycles only decreases, lowering the offset should not result in an offset that requires a speed higher than the maximum speed. At the same time, any time the offset can be lowered, such that a speed below or at the maximum speed can be obtained, we should do so. This way, we always provide coordination with the highest speed allowed. Based on Equation

4.7, we see that when Condition 4.10 is true, we can lower the offset by C_{i+1} and obtain an offset for coordination $T_{off,\hat{\imath},n}$ that is equal to or higher than $T_{off,\hat{\imath},0}$, which is the offset corresponding to the maximum speed. Lowering the offset any time sooner than when Condition 4.10 is true, for example when the coordination is no longer possible based on the minimum speed, is not useful. Namely, lowering before Condition 4.10 will always result in an infeasible coordination, based on the maximum speed.

$$(C_{i+1} - C_i) \cdot n \geq C_{i+1} \tag{4.10}$$

So, we want to lower the offset (skip an upstream cycle) when Condition 4.10 is true, and the offset is then lowered by C_{i+1}. Skipping the coordination of an upstream cycle cannot happen without consequences. Vehicles making use of this upstream cycle will not experience coordination with a downstream cycle. When the downstream green time is designed to only accommodate one upstream green cycle, a queue will form. So, to be able to skip an upstream cycle, the downstream intersection needs to make sure to have enough green time to resolve the queue. The consequences of skipping a cycle are further discussed in section 4.1.2. Skipping an upstream cycle could also be done by simply not giving green to the upstream coordinated direction in a cycle, but this methodology is not considered as it would lead to unacceptable waiting times (CROW, 2014). It should be noted that, in general, coordination may not be possible every cycle (based on the minimum speed). Still, a general formula can give insight into the amount of times coordination can be achieved. The idea behind this step is that when coordination could be provided in 99 out of 100 cycles, or 23 out of 24 or in every other cycle, this could be acceptable. Thus, by systematically decreasing the offset whenever possible, skipping a cycle and realigning the coordination, it can be investigated how many cycles will receive coordination in this way.

Equation 4.7 can be modified to reflect the logic of reducing the increase in offset by C_{i+1} when Equation 4.10 is true, producing Equation 4.11. This process is also visually depicted in Figure 4.2.

$$T_{off,\hat{\imath},n} = T_{off,\hat{\imath},0} + ((C_{i+1} - C_i) \cdot n) \bmod C_{i+1} \tag{4.11}$$

To draw conclusions regarding the usefulness of a variable speed for coordinating under different cycle times, Equation 4.11 is analyzed. The analysis, given in Appendix A, involved proving that Equation 4.12 is true. The proof explains the appearance of the greatest common divisor (gcd[1]). Equation 4.13 gives the equation for the repetition number $R_{\hat{\imath}}$ in Equation 4.12. This number denotes the integer for which the values resembling the increase in offset (added to $T_{off,\hat{\imath},0}$) repeat. For each n within the repetition cycle, a unique offset and m exists, but the respective n and m values are not necessarily equal.

$$T_{off,\hat{\imath},n} \equiv T_{off,\hat{\imath},0} + m \cdot \mathrm{gcd}(C_i, C_{i+1}) \quad \exists m \in \{0, R_{\hat{\imath}} - 1\} \tag{4.12}$$

$$R_{\hat{\imath}} = \frac{C_{i+1}}{\mathrm{gcd}(C_i, C_{i+1})} \tag{4.13}$$

[1] $\mathrm{gcd}(C_i, C_{i+1})$ is the greatest common divisor of C_i and C_{i+1}, the largest integer that divides both C_i and C_{i+1}. This function is only defined for integers.

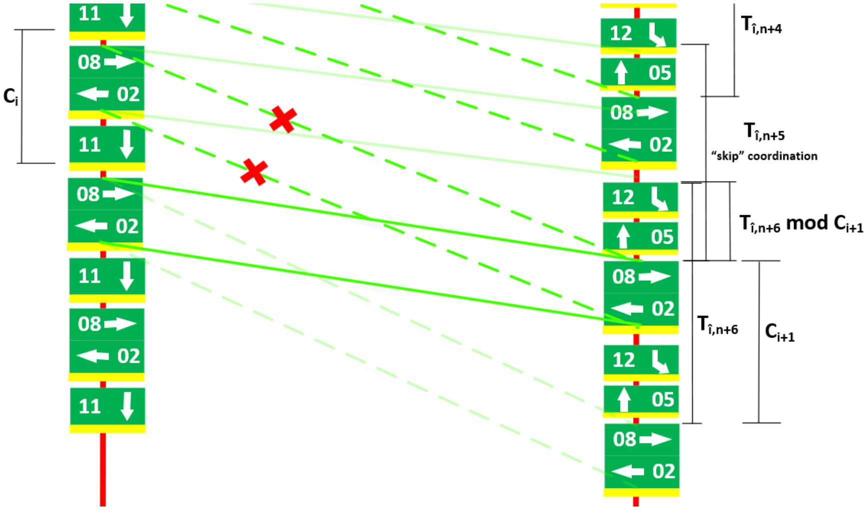

Figure 4.2: Fictitious network and signal timings showing the process of 'realigning' the coordination

Using the findings it becomes clear that under a fixed speed, coordination can be provided whenever $m = 0$. This happens every $R_{\hat{i}}$th upstream cycle, forgoing (any or 'ideal') coordination in the other cycles. This is true because whenever $m = 0$ Equation 4.12 shows that the offset is equal to $T_{off,\hat{i},0}$.

The only way to have coordination with a variable speed in every cycle of both intersections is to allow multiple upstream green windows to be coordinated with one downstream green window. Assuming this is possible, we can investigate cases where coordination is possible in every cycle. Namely, based on upper and lower bounds of the variable speed (see Equations 4.1 and 4.3), we need $T_{off,\hat{i},n}$ to satisfy the constraint in Equation 4.14.

$$T_{off,\hat{i},0} \leq T_{off,\hat{i},n} \leq T_{off,\hat{i},0} + \Delta T_{\hat{i}} \tag{4.14}$$

Whenever $m = R_{\hat{i}} - 1$, the value for $T_{off,\hat{i},n}$ is highest. We can see that, when combining constraint 4.14 with Equation 4.12 we obtain Equation 4.15, which simplifies to Equation 4.16. The lower bound is always satisfied, as can be seen from equation 4.12. Remember that per our definition $C_{i+1} > C_i > 0$, which, using Equation 4.13, shows that $R_{\hat{i}}$ can only take on integer values in the interval $\{2, C_{i+1}\}$. This implies that m will only take on positive integer values and, since the greatest common divisor is always a positive integer as well, that the varying offset only takes on values larger than the initial offset. This allows us to safely ignore the lower bound of Equation 4.14.

$$(R_{\hat{i}} - 1) \cdot \gcd(C_i, C_{i+1}) \leq \Delta T_{\hat{i}} \tag{4.15}$$

$$C_{i+1} - \gcd(C_i, C_{i+1}) \leq \Delta T_{\hat{i}} \tag{4.16}$$

4

From Equation 4.16 it follows that the range in time allowed by the variable speed ($\Delta T_{\hat{i}}$) determines the possible cycle times. Namely, if coordination in every cycle up- and downstream cycle is desired, the upstream cycle time must never be larger than this range ($\Delta T_{\hat{i}}$). Furthermore, when $R_{\hat{i}} = 2$, the constraint is easiest to satisfy. When evaluating 4.3 with some typical values of a minimum speed of 8.3 m/s, segment length of 500 meters and maximum speed of 13.9 m/s a $\Delta T_{\hat{i}}$ of 24 seconds is obtained. From Equation 4.16 we see that, at most, a downstream cycle time of 48 seconds and therefore, (when $R_{\hat{i}} = 2$) at most, an upstream cycle time of 24 seconds is allowed. It shows that coordination in every (up- and downstream) cycle, is only achievable by the variable speed if the cycle times are very low. In this case, the cycles alternate between the minimum and maximum speed and both of these upstream cycles have to be coordinated with one downstream cycle. Essentially, this creates the effect that every other upstream cycle, the vehicles drive slow enough (with the minimum speed) such that they can get coordination together with vehicles from the next upstream cycle (instructed to drive at maximum speed). The vehicles have to bunch together and a large downstream green time is required (enough for two upstream green windows). This is unrealistic behaviour and together with the low bound on the maximum cycle time for practical variable speeds, it is concluded that coordination in every cycle of both intersections is not possible.

In general, when $\Delta T_{\hat{i}} \geq C_i$, multiple upstream green windows can be coordinated with a downstream green window. This results in unrealistic cases, only cases where $C_i > \Delta T_{\hat{i}}$ are considered practical. More extreme cases are possible (especially with longer segment lengths), for example with segment lengths upwards of 800 meters, minimum and maximum speeds of 30 and 50 kilometers per hour. In this case $\Delta T_{\hat{i}}$ is equal to 38 seconds, which is still a quite low cycle time. Based on these examples, it is argued that cycle times lower than $\Delta T_{\hat{i}}$ can be excluded.

Coordination in every cycle of both intersections leads to unpractical cases. Still it is interesting to know how often coordination can be achieved, when it is allowed to skip some cycles. More specifically, it is interesting to investigate if the variable speed can provide any real benefits over the fixed speed in this regard. Forgoing the coordination sometimes, while giving it in other times is typically confusing and annoying for drivers. Still, it is desirable to find a formula to describe the percentage (or fraction) of cycles that can be coordinated. Perhaps, when 95% of the time coordination of the start green times can be achieved under different cycle times, a road authority may conclude that forgoing the coordination sometimes, may overall lead to better performance and comfort for all drivers. While such a decision is up to authority, the fraction (or percentage) of cycles that are coordinated will be useful when weighing the pros and cons of coordinating under equal cycle times or coordinating under different cycle times.

With a fixed speed, the percentage of cycles that will have coordination can be calculated according to Equations 4.17 and 4.18. These equations are easily found by knowing that upstream, every $R_{\hat{i}}$th cycle, $m = 0$ and thus coordination is achieved. To translate this fraction to the percentage of downstream cycles, Equation 4.17 is multiplied by $\frac{C_{i+1}}{C_i}$. These equations show that, when applying coordination with a fixed speed between unequal cycle times, at most 50% of the upstream cycles can be coordinated. Remember that this now assumes that only one upstream green window is allowed to be coordinated with a downstream green window. Low percentages are obtained when C_{i+1} and C_i are relatively prime (e.g. the greatest common divisor is 1).

$$p_{\text{fixed},i} = \frac{\gcd(C_i, C_{i+1})}{C_{i+1}} \cdot 100\% \tag{4.17}$$

$$p_{\text{fixed},i+1} = \frac{\gcd(C_i, C_{i+1})}{C_i} \cdot 100\% \tag{4.18}$$

Where:

$p_{\text{fixed},i}$ ($p_{\text{fixed},i+1}$) = The percentage of upstream (downstream) cycles that can be coordinated with a fixed speed between C_i and C_{i+1} [%].

To find the percentage of up- and downstream cycles that will have coordination using the variable speed we are interested in finding the number of values m for which the $\gcd(C_i, C_{i+1})$ fits inside $\Delta T_{\hat{i}}$. This number can be found by dividing and rounding down to the nearest integer, where we add 1 to account for the case where $m = 0$, or the cycle repeats. Then, by dividing this number by $R_{\hat{i}}$, we obtain the fraction of upstream cycles for which coordination can be achieved with the variable speed. The formula for this percentage is shown in Equation 4.19. Again we can find the percentage of downstream cycles by multiplying by $\frac{C_{i+1}}{C_i}$. To properly discount cases where $\Delta T_{\hat{i}} \geq C_i$, the equation must take into account that coordination can never be achieved more often than the number of downstream green windows per repetition cycle. Thus, at most $\frac{C_i}{\gcd(C_i, C_{i+1})}$ times per $R_{\hat{i}}$ cycles. For this reason a min function is included in the equations.

$$p_{\text{variable},i} = \frac{\gcd(C_i, C_{i+1})}{C_{i+1}} \cdot \min\left\{ \left\lfloor \frac{\Delta T_{\hat{i}}}{\gcd(C_i, C_{i+1})} \right\rfloor + 1; \frac{C_i}{\gcd(C_i, C_{i+1})} \right\} \cdot 100\% \tag{4.19}$$

$$p_{\text{variable},i+1} = \frac{\gcd(C_i, C_{i+1})}{C_i} \cdot \min\left\{ \left\lfloor \frac{\Delta T_{\hat{i}}}{\gcd(C_i, C_{i+1})} \right\rfloor + 1; \frac{C_i}{\gcd(C_i, C_{i+1})} \right\} \cdot 100\% \tag{4.20}$$

Where:

$p_{\text{variable},i}$ ($p_{\text{variable},i+1}$) = The percentage of upstream (downstream) cycles that can be coordinated with a variable speed, based on the maximum time difference allowed by the minimum and maximum speed limits and distance on segment $\hat{i}$ ($C_i < C_{i+1}$) [%].

Note that we skip the coordination upstream every time the modulo operation in Equation 4.11 is applied. This happens a fraction: $(C_{i+1} - C_i)/C_{i+1}$ of the cycles. To find the amount of times we forgo coordination in a repetition cycle, we multiply the fraction by the number of cycles per repetition $R_{\hat{i}}$ to get: $R_{\hat{i}}(C_{i+1} - C_i)/C_{i+1} = (C_{i+1} - C_i)/\gcd(C_i, C_{i+1})$ times. This number needs to be subtracted from $R_{\hat{i}}$ to get the maximum fraction of cycles that can be coordinated (accounting for the cycles where coordination is skipped). Equation 4.21 shows that including this in the min brackets in

Equation 4.19, has no effect at all, and was therefore left out. Note that if we recalculate the percentage of cycles that can be coordinated with the same figures as before: $C_i = 24, C_{i+1} = 48$ and $\Delta T_{\hat{i}} = 24$, we find that we now have coordination 50% of the upstream cycles. Before, when we allowed multiple upstream green windows to be coordinated with a single downstream green window, this percentage would have been a 100%. Now the skipped cycles are assumed to not coordinate, resulting in the lower percentage.

$$R_{\hat{i}} - \frac{(C_{i+1} - C_i)}{\gcd(C_i, C_{i+1})} = \frac{C_i}{\gcd(C_i, C_{i+1})} \tag{4.21}$$

Dividing Equations 4.17 and 4.18 by Equations 4.19 and 4.20, we see that with the variable speed, coordination can be achieved $\min\left\{\left\lfloor \frac{\Delta T_{\hat{i}}}{\gcd(C_i, C_{i+1})} \right\rfloor + 1; \frac{C_i}{\gcd(C_i, C_{i+1})}\right\}$ times more than with the fixed speed. In section 4.1.2, we use this fraction to draw some conclusions about the usefulness of a variable speed for coordination with different cycle times.

4

4.1.2. CONCLUSION

Regarding the application of a variable speed for coordination with different cycle times, given that the upstream cycle time is greater than $\Delta T_{\hat{i}}$, the following three lines of reasoning can be applied:

- The different cycle times are quite close to each other (e.g. within 15% or 10 seconds range). The best choice will likely be to set the times equal and coordinate with equal cycle times. In this case, fixed and variable speed achieve the same amount of coordination, namely in every cycle. Reason being that when the cycle times are close, there is not much time 'lost' by having both intersections coordinated under the highest cycle time. The drawback of coordinating under different cycle times in this case, namely that the coordination is skipped in some cycles, is likely not worth it to bridge a small difference in cycle times.

- The different cycle times have a very large difference (e.g. the shorter cycle time is approximately half or a third of the longer cycle time). In this case, it is possible to design for cycle times that are a fraction of each other, without having to increase either of the cycle times substantially. This leads to a high $\gcd(C_i, C_{i+1})$, such that $\frac{\Delta T_{\hat{i}}}{\gcd(C_i, C_{i+1})} < 1$ and variable speeds are not able to make a difference over fixed speeds. For example a when $C_i = 40$ and $C_{i+1} = 80$ seconds.

- Now consider an extreme case, where the percentage of cycles with coordination provided with a variable speed significantly exceeds that of the fixed speed. Using $\Delta T_{\hat{i}} = 24$ seconds from before, such a case may arise when $C_i = 62$ and $C_{i+1} = 80$ seconds. The variable speed then provides coordination (32.5% of upstream cycles) thirteen times as often, over the fixed speed (2.5% of upstream cycle). While relatively, this is a good result for the variable speed, only coordinating in 32.5% of the cycles is not good in absolute terms. Furthermore, just increasing the upstream cycle time by 2 seconds ($C_i = 64$ seconds), the variable speed (40%) coordinates only twice as often, over the fixed speed (20%). Here, a traffic engineer could also choose to apply an 80 second cycle time to both intersections, for coordination in all cycles. Whether a 16 to 18 seconds lower cycle time upstream outweighs the disadvantage of skipping the coordination in about 60 to 68% of the upstream cycles is doubtful. More research would be needed to assess the advantages and disadvantages, such that an engineer or municipality may make an informed decision.

Evaluating all lines of reasoning, the (constrained) variable speed may only rarely provide benefits over a fixed speed for coordination under different cycle times. When the variable speed, relatively speaking, does provide coordination in significantly more cycles over the fixed speed, the absolute number of cycles is quite low such that it's difficult to draw further conclusions. These rest upon the question whether a lower cycle time or more coordination is more beneficial.

This conclusion rests on some assumptions made throughout section 4.1.1. Firstly, it was assumed that cycle times are integers. All quantities in the derived equations are in seconds. The formulas are also applicable in for example milliseconds, as long as the cycle times are integer. If it is desired to know the percentage of cycles with coordination between cycle times of 62,3 seconds and 81,7 seconds, with $\Delta T_{\hat{i}} = 20$ seconds the formulas can be applied with $C_i = 623$, $C_{i+1} = 817$ and $T_{\hat{i}} = 200$.

Secondly, the choice was made to look at the case $C_i < C_{i+1}$. The same formulas are not directly applicable to the case $C_i > C_{i+1}$. Furthermore, this case requires a different interpretation. When the upstream cycle time is larger than the downstream cycle time, downstream the cycle skipping takes place. Skipping the coordination of a downstream cycle does not affect vehicles on the segment, as these need to pass the upstream cycle first. For the case $C_i > C_{i+1}$ the derivation of the formulas is nearly identical, and the final results, besides the aforementioned difference in interpretation, are expected to be similar as well. The expectation of highly similar results is the reason for thesis to not consider cases where $C_i > C_{i+1}$. A difference in the derivation occurs at the assumption of supplying the first coordination with the maximum speed. When $C_i > C_{i+1}$, the required offset for coordination is decreasing every cycle, instead of increasing. For the derivation, it is possible to start with the minimum speed in the first cycle, to be able to apply the same (mirrored) condition regarding the skipping of cycles. However, this does have further consequences for cases where $\Delta T_{\hat{i}} \geq C_{i+1}$, namely where multiple (in this case downstream) green windows can be coordinated with one upstream green window. When the upstream cycle is coordinated, it is not logical to have vehicles from one upstream green window drive with two different speeds to arrive at two different downstream green windows. Coordination should in this case always happen with only one downstream green window, and when two speeds are possible, it should be the highest speed, to avoid unnecessary delays and incredibility. This will be a problem when condition 4.10 is applied in the same way, but with the minimum speed as starting point. Overall, it is advised to simply disregard cases where multiple downstream green windows can be coordinated from one upstream green window, as these require quite large link lengths and low cycle times to occur anyway.

Another assumption is that coordination is based on the alignment of the start of the green window. The equations are equally valid for the coordination of any other single point in the green window, like for example the middle of green. So in truth, the equation gives the fraction of cycles that the start of green (or another point) can be aligned. When the start of green is aligned, there is coordination (when the green time is sufficient, for the complete platoon). However, when the start of green is not aligned, there still may be some amount of coordination for some part of the platoon. The equations do not include this partial coordination. Evaluating the alignment of the full green window (instead of just the start of green) is outside the scope of this thesis. It is expected that, when including the alignment of complete windows, the amount of coordination will be higher, and the increase for the variable speed will be higher than for the fixed speed.

In this (theoretical) analysis no demands, queue lengths, platoon dispersion, etc. is included. This doesn't change the conclusion, but should be noted when trying to apply this theory to practice.

A final note about the analysis is that it was done between two intersections, disregarding the rest of the arterial. However, since segments are not independent, the percentage can only reflect an arterial when all upstream coordination (i.e. from C_{i-1} to C_i) and further downstream coordination (i.e. from C_{i+1} to C_{i+2}) happens in 100% of the cycles. With more effort, formulas could be derived to find the percentage of cycles that coordinate when viewing an entire arterial. However, this percentage can never be larger than the percentage for any individual segment. Since absolute percentages for a single segment are already low, research into percentages for the entire arterial, when multiple intersections of the arterial have different cycle times, is discouraged.

4.2. COORDINATION IN TWO DIRECTIONS

This section is aimed at analyzing when and where, in theory, the proposed variable speed provides more opportunities for coordination in two directions, over a fixed speed. This analysis is focused on a coordination that is based on the alignment of the start of green in cases where the starts of green of the coordinated directions are (nearly) simultaneous.

Based on the findings for coordination under different cycle times in one direction (see section 4.1.1), cycle times are assumed equal for the analysis in two directions.

To provide coordination between the start of green in two directions, another constraint is put on the cycle time. This happens because two neighbouring intersections (with equal cycle times) require that a certain offset is applied along the same segment. When the start of green is assumed to start at the same point in time every cycle (in other words: cycle times are time-invariant) the two directions that are coordinated have a fixed internal time-difference. In other words, the green for the coordinated outbound direction will come before or after in time with respect to the green for the coordinated inbound direction. The difference at a certain intersection will be equal in every cycle. Figure 4.3 shows a schematic for a random (fictitious) network, with coordination in two directions.

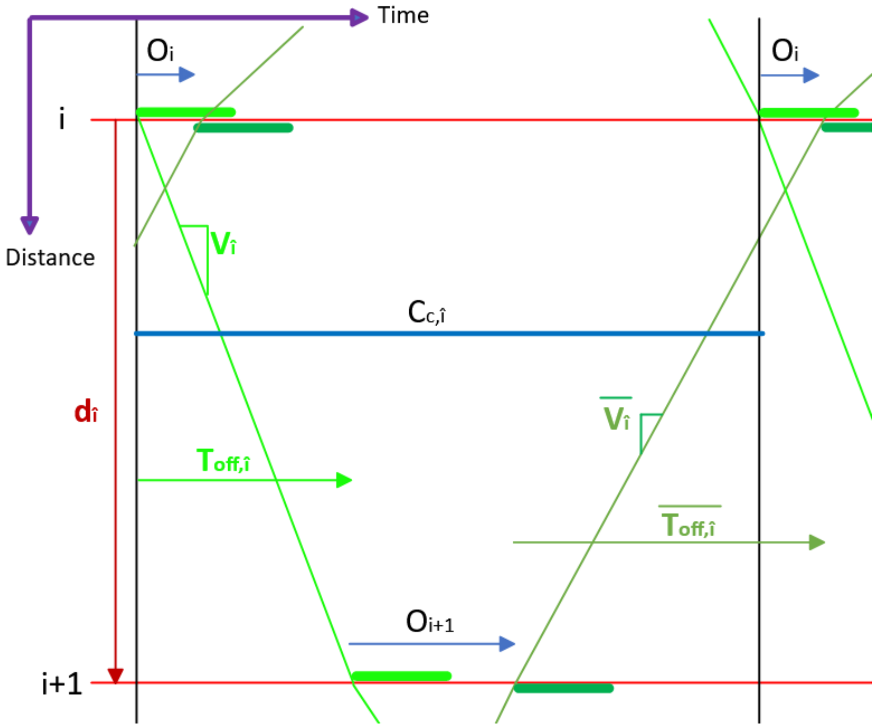

Figure 4.3: Fictitious network and signal timings showing coordination in two directions and the variables at play

Equation 4.22 formulates the constraint, which assumes that there is coordination when the starts of green for the coordinated directions are aligned according to the offset. Notation of certain variables is simplified in comparison to the previous section, as the variables are now time-invariant. In section 4.3, a reflection is made on the assumption that coordination takes place between the start of green. Another assumption made by this formulation is that the outbound and inbound segments between the same intersections are equal in length. When measuring segment lengths between the stop lines of neighbouring intersection's coordinated directions, slight differences in the length can be caused by different intersection topologies or for example the presence of a curve. The length of segment that takes the outside of the curve will be slightly longer than the length of the segment on the inside of the curve. These differences are assumed to be negligible.

$$n_{\hat{i}} \cdot C_{c,\hat{i}} = T_{off,\hat{i}} + O_i + \overline{T_{off,\hat{i}}} - O_{i+1} \tag{4.22}$$

$$T_{off,\hat{i}} = \frac{d_{\hat{i}}}{v_{\hat{i}}} \tag{4.23}$$

$$\overline{T_{off,\hat{i}}} = \frac{d_{\hat{i}}}{\overline{v_{\hat{i}}}} \tag{4.24}$$

$$O_i = SG_i - \overline{SG_i} \tag{4.25}$$

Where:

$n_{\hat{i}}$ = An integer number of cycles on segment $\hat{i}$ with $n \in \mathbb{N}^+$ [-]. The coordination does not necessarily have to happen between subsequent cycles. Coordination is also achieved when any (positive, whole) number of cycles is skipped.

$C_{c,\hat{i}}$ = The minimum cycle time according to segment $\hat{i}$ to provide coordination in two directions (between intersection i and i+1) in every cycle, meaning that this cycle time should be applied to both intersection i and i+1 [s].

$T_{off,\hat{i}}(\overline{T_{off,\hat{i}}})$ = The offset on segment $\hat{i}$ required for coordinating in the outbound (inbound) direction [s]. The offset is determined by the distance and outbound (inbound) speed according to Equation 4.23 (4.24). Variables for the inbound direction are always denoted with an overline above the variable.

O_i = The (fixed) internal time difference between the start of green of the two coordinated directions at intersection i as defined by Equation 4.25 [s]. The time difference should be taken within one cycle, such that $O_i < C_i$.

$SG_i(\overline{SG_i})$ = The start green time of the outbound (inbound) coordinated direction [s].

The problem is divided into three sub-problems, characterized by the following three Equality's:

1. No space is allowed between outbound and inbound green, both have to start at exactly the same time. Reflected by Equality 4.26. In practice, such a situation may occur when all conflicts on an intersection conflict with both main directions. When the main directions are of similar demand, it is very inefficient to not have them have green at the same time in the timing plan. When the main directions do not start at the same time, the difference in start times is added to the cycle time, consequently creating a larger cycle time than needed to serve the demand.

$$O_i = 0 \quad \forall i \in \{1, I\} \tag{4.26}$$

2. Some space is allowed between outbound and inbound green, but this space is constrained. Reflected by Equality 4.27. In practice, this situation may occur, when for example the main directions are accompanied by separate left-turns. These left-turns conflict only with one of the two main directions, but have relatively low demand to the main directions. This allows for the main directions to start nearly simultaneous (not exactly), without losing a lot of efficiency with regards to the locally optimal timing plan or without creating a larger cycle time than needed to serve the demand.

$$|O_i| \leq G \quad \forall i \in \{1, I\} \tag{4.27}$$

3. Outbound and inbound green do not have to start at the same time, the space in between is unconstrained. Reflected by Equality 4.28. In practice this may

occur when, besides left-turns for the main directions, also separate right-turns are present on the side directions. And/or that (all of) these directions have substantial demand, relative to the main directions. Consequently, a lot of different control structures are possible, also ones where the main directions are not given green at the same time.

$$|O_i| \leq C_{c,\hat{i}} \quad \forall i \in \{1, I\} \tag{4.28}$$

In each three sub-problems, all intersections in the network have a similar layout. This is of course not realistic for networks in practice. The solution methods found in the sub-problems will be generalized in section 4.2.3 to allow for a network consisting of intersections with different layouts. As mentioned in section 1.2.4, this thesis does not investigate under which conditions coordinated green windows start (nearly) simultaneous. That this is a requirement for locally optimal signal timing plans in coordinated networks is part of the assumptions. As follows from the problem setup in this section, this assumption is vital for the application and usefulness of the analysis.

4.2.1. COORDINATION IN TWO DIRECTIONS: SUB-PROBLEM 1

Sub-problem 1 is characterized by the simultaneous start of coordinated outbound and inbound green windows. This can occur in practice in a network of intersections that do not have any separate directions in the control structure that have a conflict with only one of the two coordinated directions. For example, assume directions 02 and 08 (according to the Dutch naming convention (CROW, 2014)) are coordinated, then this section assumes that there is no separate 03, 04, 09 or 10 present. When these directions are not present, the coordinated directions (e.g. 02 and 08), assuming that both have substantial demand, should be given green at the same time in an efficient timing plan. No internal offset is present between the coordinated directions, the value is 0, thus by applying constraint 4.26, Equation 4.22 simplifies to Equation 4.29.

$$C_{c,\hat{i}} = \frac{T_{off,\hat{i}} + \overline{T_{off,\hat{i}}}}{n_{\hat{i}}} \tag{4.29}$$

Since equal cycle times on all intersections are required we must have that equality 4.30 is always true.

$$\frac{T_{off,\hat{i}} + \overline{T_{off,\hat{i}}}}{n_{\hat{i}}} = \frac{T_{off,\hat{i}+1} + \overline{T_{off,\hat{i}+1}}}{n_{\hat{i}+1}} \quad \forall \hat{i} \in \{1, I-2\} \tag{4.30}$$

FIXED SPEED

If a fixed speed is applied, outbound offsets are equal to inbound offsets. Furthermore, since all segments have the same (fixed) speed, the offsets per segment given by Equations 4.23 and 4.24, differ only based on the distance. Equation 4.29 can thus be written as Equation 4.31 and equality 4.30 can be written as equality 4.32.

$$C_{c,\hat{i}} = \frac{2 \cdot d_{\hat{i}}}{n_{\hat{i}} \cdot v} \tag{4.31}$$

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$$d_{\hat{i}} = \frac{n_{\hat{i}}}{n_{\hat{i}+1}} \cdot d_{\hat{i}+1} \quad \forall \hat{i} \in \{1, I-2\} \tag{4.32}$$

From equality 4.32 it follows that all segments must have the same distance or have a distance with a multiple of $\frac{n_{\hat{i}}}{n_{\hat{i}+1}}$, otherwise some intersections will not be able to coordinate the starts of green of the outbound and inbound direction and fulfill the cycle time constraint.

All segments being equidistant is unlikely to occur in practice. Though this very much depends on where you live (Boeing, 2017). If any values for $n_{\hat{i}}$ are allowed, the coordination of the starts of outbound and inbound green would be possible with the fixed speed. However, it is important to realize here that Equation 4.31 demands that $n_{\hat{i}}$ not be very large. On a 1000 meter segment, with a fixed speed of 50 km/h, a $n_{\hat{i}}$ of 5 already results in a cycle time of only 29 seconds. Cycle times below 30 seconds are not practical, at least in the Netherlands (M. Barto, 2022, personal statement). Even then if Equality 4.32 is satisfied, it should still be checked that the found cycle time is realistic. Overall, it can be seen that with a fixed speed, compromises will have to be made between coordination of simultaneous starts of green and the ideal cycle time and signal timings for fulfilling demand.

VARIABLE SPEED

For the variable speed, which varies per segment and per outbound/inbound direction, Equation 4.29 can be written as Equation 4.33 and Equality 4.30 can be written as equality 4.34.

$$C_{c,\hat{i}} = \frac{d_{\hat{i}} \cdot (v_{\hat{i}} + \overline{v_{\hat{i}}})}{n_{\hat{i}} \cdot v_{\hat{i}} \cdot \overline{v_{\hat{i}}}} \tag{4.33}$$

$$\frac{d_{\hat{i}} \cdot (v_{\hat{i}} + \overline{v_{\hat{i}}})}{n_{\hat{i}} \cdot v_{\hat{i}} \cdot \overline{v_{\hat{i}}}} = \frac{d_{\hat{i}+1} \cdot (v_{\hat{i}+1} + \overline{v_{\hat{i}+1}})}{n_{\hat{i}+1} \cdot v_{\hat{i}+1} \cdot \overline{v_{\hat{i}+1}}} \quad \forall \hat{i} \in \{1, I-2\} \tag{4.34}$$

Equality 4.34 would be very easy to satisfy if any value of $n_{\hat{i}} \in \mathbb{N}$ is allowed. However, Equation 4.33 shows that, also for the variable speed, large values of $n_{\hat{i}}$ lead to unrealistically low cycle times.

Distances are the only uncontrollable network characteristic in Equation 4.33 and equality 4.34. To investigate the uses of the variable speed further, consider the set of distances, segment lengths, of the entire arterial: $\{d_0, \ldots, d_{\hat{i}}, \ldots, d_{I-1}\}$. We know there must be some maximum distance (d_{max}) and minimum distance (d_{min}) in the set. The lowest and highest possible speed on any segment is achieved when both outbound and inbound speeds are set to v_{min} and v_{max}, respectively. Denote $n_{\hat{i}}$ on the segment with the minimum and maximum distance as n_{min} and n_{max}, respectively. To satisfy equality 4.34 it makes intuitive sense to apply the highest speed on the longest segment. This makes it easiest to satisfy the equality and makes sure that, if all constraints are satisfied, the travel times is minimal. Furthermore, in the most extreme case, the shortest segment in the set is followed immediately by the longest segment or vice versa. Processing these steps in Equality 4.34, we obtain Equality 4.35, which, if it can be satisfied, should indicate that Equality 4.34 can also be satisfied. Namely, if coordination is possible in this extreme case, where the shortest and longest segment of the set are next to each other, it should also be possible in other cases.

$$\frac{n_{max} \cdot d_{min}}{n_{min} \cdot d_{max}} = \frac{v_{min}}{v_{max}} \tag{4.35}$$

Equality 4.35 is satisfied when the applied v_{min} is higher than the minimum allowed v_{min}. We use constraint in Equation 4.1, which, as a reminder, also states that the minimum speed can be written as a fraction l times the maximum speed. l resembles the fraction of the maximum speed chosen as the minimum speed. This can be used to translate equality 4.35 to the more useful representation shown in Inequality 4.36.

$$\frac{n_{max}}{n_{min}} \cdot d_{min} \geq d_{max} \cdot l \tag{4.36}$$

Inequality 4.36 shows that the variable speed can always provide coordination if $d_{min} \geq d_{max} \cdot l$. In this case, $n_{max} = n_{min}$. Imagine other cases, where $d_{min} < d_{max} \cdot l$, then coordination is still possible by increasing n_{max}. Logically, for higher l, coordination is harder to achieve by the variable speed. A value for l lower than 0.5 could result in impractically low speeds (e.g. slower than 25 km/h if the maximum speed is 50 km/h). However, for realistic values of 0.5 or 0.6, it can be seen that the variable speed is able to provide coordination between the simultaneous starts of outbound and inbound greens for a large range of segment distances. It is hard, at least in the Netherlands, to find an arterial where the shortest segment is shorter than 250 meters and the longest is longer than 1000 meters (for segments longer than 1000 meters often coordination is not even applied). In the extreme case, when these segments are next to each other, than still coordination is possible, provided $l = 0.5$, $n_{min} = 1$ and $n_{max} = 2$.

This shows that the (constrained) variable speed allows to satisfy equality 4.34 for many realistic sets of distances. However, thus far, only the geometric part of the coordination is considered. It says nothing about the obtained cycle times and whether these are realistic. As mentioned before, high $n_{\hat{i}}$ lead to unrealistically low cycle times.

Since we require that $C_{c,\hat{i}}$ from 4.33 be equal on all segments, we denote this 'coordination' cycle time as C_c. Furthermore, we have that this cycle time should ideally be equal to the (network) cycle time that is able to deal with the demand, denoted as C_q.

$$C_c = C_q \tag{4.37}$$

$$C_q = \frac{d_{\hat{i}} \cdot (v_{\hat{i}} + \overline{v_{\hat{i}}})}{n_{\hat{i}} \cdot v_{\hat{i}} \cdot \overline{v_{\hat{i}}}} \quad \forall \hat{i} \in \{1, I-1\} \tag{4.38}$$

To satisfy equality 4.38, we can identify the most extreme case. This happens when C_q has a large value, (small C_q can more easily be achieved since we are allowed to divide by $n_{\hat{i}}$). A high C_q is hardest to satisfy on the shortest segment. This segment will be normative in determining whether the network cycle time can be achieved on all segments. It makes sense, to set $n_{\hat{i}}$ to 1 in this case. Furthermore, we again consider constraint 4.1, using which we can conclude that constraint 4.39 must hold to guarantee that the coordination between the simultaneous starts of outbound and inbound green can be provided by the variable speed, for the given network (demand) cycle time of C_q.

$$C_q \leq \frac{2 \cdot d_{min}}{l \cdot v_{max}} \tag{4.39}$$

Typically in a design process, the network cycle time (C_q) would be calculated first. Then the speeds could be designed to provide the best coordination possible (spawning an iterative process of tuning timings and changing speeds).

Checking some extreme values, we see that with a shortest segment length of 250 meters and a maximum speed of 13,89 meters per second, the network cycle time must not be larger than 60 seconds for $l = 0.6$ and 72 seconds for $l = 0.5$. These cycle times are reasonable, but slightly on the short side for networks used in practice. However, in practice, the systems are more complex. In this section, we have considered sub-problem 1, where no separate conflicts exist for the main directions and the coordinated directions are given green at the exact same time. For these simpler networks a network cycle time below 60 seconds can reasonably be expected. This assumes that the intersections are not over-saturated, such that the high demand would require a larger cycle time. Using the famous cycle time formula by Webster (1958), given a total lost time of 9 seconds, a 60 second cycle time is optimal when the total saturation rate of the critical conflict group is 70%. These figures are all fictitious, meant to give an indication. Overall they indicate that the variable speed can realistically provide coordination between the start of green of outbound and inbound green windows that start simultaneously, in many systems. More importantly, the variable speed can do this way more easily than the fixed speed. Equation 4.39 can also be used for a fixed speed, by choosing $l = 1$. For the same values, only low cycle times (below 36 seconds) are possible with the fixed speed.

One important design aspect to mention regards the application of different speeds on subsequent segments. From a driver's point of view, large speed differences between subsequent segments may be undesirable as they would require 'sudden' braking or accelerating. Preferably, the designed plan takes into account that differences between speeds on subsequent segments are minimized. However, this may not always be possible, for example when the segment with d_{min} is followed by a segment just shorter than twice the length of d_{min}. In this case, to provide equal cycle times, it may be inevitable that the speed difference on subsequent segments is roughly equal to $v_{max} - v_{min}$. This topic is further discussed in section 9.1.

4.2.2. COORDINATION IN TWO DIRECTIONS: SUB-PROBLEM 2

Sub-problem 2 is characterized by allowing some time, though constrained, between the starts of green of outbound and inbound green windows. In practice, some direction (with some demand) may conflict with the coordination in one direction, but not with the coordination in the other direction. When constructing the 'optimal' signal timing plan for the demand, these separate conflicts provide an opportunity to apply different phase orders, where the main directions are not given green at the exact same time.

Once again using the Dutch naming convention, suppose that 02 and 08 are the directions designed for coordination, then this section considers cases where a separate lane and signal is present for (for example) a 03, 04, 09 or 10 movement. However, the total demand for these movements should be small relative to the demand for the main directions. This results in cases where, in the locally optimal timing plan, main directions do not have to start at exactly the same time, but still start nearly simultaneous.

Sub-problem 2 is mathematically formulated by constraining the fixed internal time difference by some time G according to Equation 4.40. This G can be interpreted as for example the green time of a 03 direction. However, typically this value would be different per intersection. For this analysis, to come up with some general formulas, G is treated as a constant across all intersections of the arterial. This is similar to how sub-problem 1 was handled and can be interpreted as all intersections of the arterial being identical. In section 4.2.3, when the generalization is made, different values per intersection are allowed. The absolute value marks are needed as per definition of the internal time difference by Equation 4.25, it is known that it can be positive or negative.

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$$|O_i| \le G \quad \forall i \in \{1, I\} \tag{4.40}$$

FIXED SPEED

Starting with the fixed speed and following the same reasoning as section 4.2.1, we obtain Equality 4.41.

$$\frac{\frac{2\cdot d_{\hat{i}}}{v} + O_i - O_{i+1}}{n_{\hat{i}}} = \frac{\frac{2\cdot d_{\hat{i}+1}}{v} + O_{i+1} - O_{i+2}}{n_{\hat{i}+1}} \quad \forall \hat{i}, i \in \{0, I-2\} \tag{4.41}$$

Equality 4.41 can be rearranged to get Equality 4.42, which can further be rearranged to get Equality 4.43.

$$\frac{2\cdot d_{\hat{i}}}{v} + O_i - O_{i+1} = \frac{n_{\hat{i}}}{n_{\hat{i}+1}} \cdot \left(\frac{2\cdot d_{\hat{i}+1}}{v} + O_{i+1} - O_{i+2}\right) \quad \forall \hat{i}, i \in \{0, I-2\} \tag{4.42}$$

$$d_{\hat{i}} - \frac{n_{\hat{i}}}{n_{\hat{i}+1}} \cdot d_{\hat{i}+1} = \frac{v}{2} \cdot \left(\left(1 + \frac{n_{\hat{i}}}{n_{\hat{i}+1}}\right) \cdot O_{i+1} - O_i - \frac{n_{\hat{i}}}{n_{\hat{i}+1}} \cdot O_{i+2}\right) \quad \forall \hat{i}, i \in \{1, I-2\} \tag{4.43}$$

In the extreme case, the internal time differences are all equal to G. The right hand side of Equality 4.43 is minimized or maximized when O_{i+1} has the opposite sign of O_i and O_{i+2}. Thus, Inequality 4.44 gives a condition which must hold for the fixed speed to be able to coordinate between the nearly simultaneous starts of green of outbound and inbound green windows. Inequality 4.44, when satisfied, does not guarantee coordination with a fixed speed between the nearly simultaneous starts of green is possible. This is because it might not be possible for O_{i+1} to have the opposite sign of O_i and O_{i+2} across the same two segments where the difference in consecutive segment distances is largest.

$$\left| d_{\hat{i}} - \frac{n_{\hat{i}}}{n_{\hat{i}+1}} \cdot d_{\hat{i}+1} \right| \le v \cdot \left(1 + \frac{n_{\hat{i}}}{n_{\hat{i}+1}}\right) \cdot G \quad \forall \hat{i} \in \{1, I-2\} \tag{4.44}$$

Finding the zero's of the left hand side of Equality 4.44 (as if the right hand side was zero) would be equal to solving Equality 4.32. This, as previously mentioned on page 62, is unlikely to produces practical results. We see now, that, because of allowing some time difference between the starts of green of the coordinated directions, the fixed speed can provide coordination in two directions more often (for more sets of distances). The right hand side can be larger than zero. To be able to analyze more properties we consider cases where $n_{\hat{i}} = 1$ for all $\hat{i}$ in $\{1, I-1\}$. This assumption is quite realistic as from the cycle time formula, we know that $n_{\hat{i}} = 1$ results in larger cycle times. As a reminder, this section considers intersections where some conflicts are present that conflict with only one of the two coordinated directions. With the presence of at least one of these conflicts, a moderate (to large) cycle time could be expected, more so than in section 4.2.1.

Now Equality 4.43 becomes Equality 4.45.

4

$$d_{\hat{i}} - d_{\hat{i}+1} = \frac{v}{2} \cdot (2 \cdot O_{i+1} - O_i - O_{i+2}) \quad \forall \hat{i}, i \in \{1, I-2\} \tag{4.45}$$

We see that Equality 4.45 represents a system of linear equations. To analyze this system further, it is written in matrix form in Equation 4.46. Note that the analysis from this point on and this matrix form, do not represent cases where intersections are provided coordination with an unequal $n_{\hat{i}}$.

$$A \cdot \mathbf{O} = \frac{2}{v} \cdot \delta \mathbf{d} \tag{4.46}$$

With:

$$A_{(I-2 \times I)} = \begin{bmatrix} -1 & 2 & -1 & 0 & 0 & \cdots & 0 & 0 & 0 \\ 0 & -1 & 2 & -1 & 0 & \cdots & 0 & 0 & 0 \\ 0 & 0 & -1 & 2 & -1 & \cdots & 0 & 0 & 0 \\ \vdots & \vdots & & \ddots & \ddots & \ddots & & \vdots & \vdots \\ 0 & 0 & 0 & \cdots & -1 & 2 & -1 & 0 & 0 \\ 0 & 0 & 0 & \cdots & 0 & -1 & 2 & -1 & 0 \\ 0 & 0 & 0 & \cdots & 0 & 0 & -1 & 2 & -1 \end{bmatrix}, \tag{4.47}$$

$$\mathbf{O}_{(I \times 1)} = \begin{bmatrix} O_i \\ O_{i+1} \\ O_{i+2} \\ \vdots \\ O_{I-1} \\ O_I \end{bmatrix} \text{ and} \tag{4.48}$$

$$\delta \mathbf{d}_{(I-2 \times 1)} = \begin{bmatrix} d_1 - d_2 \\ d_2 - d_3 \\ d_4 - d_5 \\ \vdots \\ d_{I-3} - d_{I-2} \\ d_{I-2} - d_{I-1} \end{bmatrix} \tag{4.49}$$

From Matrix A, shown in Equation 4.47 a few things can be concluded. Firstly, it's dimensions are (I-2 × I), meaning that there are more columns than rows. For our problem this means that there are either no solutions or infinitely many solutions. It is easy to see that the rank of matrix A is I-2 (multiply all rows of A by -1 and, starting from the bottom, adding each row twice to the row above). In other words, all I-2 rows are nonzero rows in the echelon from of A. Thus, A is full rank. The linear algebra definitions are taken from Lay et al. (2015). This means that we can conclude that the problem always has infinitely many solutions. Intuitively, this makes sense, since we know from section 4.2.1 that a fully equidistant network combined with all zero internal offsets is at least one solution. Furthermore, it follows that the Null space of A contains two dimensions, such that in any solution, $\mathbf{O}$ has two free parameters. If any value for the entries in $\mathbf{O}$ is allowed, coordination between the starts of green is always possible with the fixed speed. To solve the system, construct the augmented matrix $(A|\delta\mathbf{d})$ and find the echelon form as described before. The solutions found by this procedure (all of the solutions) are described by Equation 4.50.

4

$$\mathbf{O} = \mathbf{O}_p + u \tag{4.50}$$

$$\mathcal{N}(A)_{(I\times 2)} = \text{span}\left(\begin{bmatrix} I-1 \\ I-2 \\ I-3 \\ \vdots \\ 2 \\ 1 \\ 0 \end{bmatrix}, \begin{bmatrix} 2-I \\ 3-I \\ 4-I \\ \vdots \\ -1 \\ 0 \\ 1 \end{bmatrix}\right) \tag{4.51}$$

Where:

$\mathbf{O}_p$ = Any particular solution to the system described by Equation 4.46.

u = Any vector in the Null-Space of A: $u \in \mathcal{N}(A)$.

$\mathcal{N}(A)$ = The Null-Space of A, obtained via the aforementioned echelon form of A and choosing the two free variables as 1, is given by Equation 4.51.

As a side note, from Equation 4.51 it can be observed that for any I, the Null-Space contains a vector of all ones. This vector is obtained adding the first base to the second base of the Null-Space. Furthermore, any scalar multiple of this vector is also in the Null-Space of A. This means that any vector with all equal entries is a valid vector for u, which means that by adding any constant to all entries of any particular solution $\mathbf{O}_p$ also produces a valid solution, according to Equation 4.50. Intuitively, this finding makes sense. If a solution is found, translating this solution in time should have no effect at all on satisfying the geometric requirements for coordinating the starts of green.

Regarding this section and constraint 4.40, we are interested in the solution $\mathbf{O}$ that has the minimum maximum absolute value for each entry in $\mathbf{O}$. Namely, we need to check if these values are less than G, according to constraint 4.40. Such a solution is known as a minimum L_∞ norm, minimum uniform norm and minimum Chebyshev norm solution. Mathematically, we want to find the solution $\mathbf{O}$ that minimizes Equation 4.52, because we need to satisfy Inequality 4.53. Finding this solution in a general way, for the given system, is no trivial task. As far as the author is aware there is no general (analytical) solution for the minimum uniform norm solution for an under-determined system of equations. There do exist (computationally efficient) algorithms for computing this solution, for example (Earle et al., 2017). Using such an algorithm for this problem would be quite overkill and take away from the intuition and understandability of the current formulation. Instead, a useful way to approach the problem is from the reverse direction. A linear program can then be formulated, which instead of minimizing Equation 4.52, minimizes a temporary bound, denoted as G^*. The minimized bound G^* can be compared to G to conclude if coordination between the starts of green can be provided (geometrically) by the fixed speed, when the starts of inbound and outbound green are not simultaneous, but are allowed to differ in time by at most G seconds.

$$||\mathbf{O}||_\infty = \max(|O_1|, |O_2|, |O_3|, \ldots, |O_{I-1}|, |O_I|) \tag{4.52}$$

$$||\mathbf{O}||_\infty \leq G \tag{4.53}$$

LP1: Coordination with a fixed speed in Sub-problem 1

Objective function:

$$\min_{G^*} \quad z = G^*$$

Subject to:

$$\begin{aligned} A \cdot \mathbf{O} &= \frac{2}{v} \cdot \delta\mathbf{d} \\ -G^* - O_i &\leq 0 \quad \forall i \in \{1, I\} \\ -G^* + O_i &\leq 0 \quad \forall i \in \{1, I\} \end{aligned}$$

Variables:

$$\begin{aligned} O_i &\in \mathbb{R} \quad \forall i \in \{1, I\} \\ G^* &\geq 0 \end{aligned}$$

Input parameters:

$$v, \ \delta\mathbf{d}$$

Two heuristics can be identified, which, when they are not satisfied, allow us to conclude that the formulated coordination is not possible, before running the linear program. The first heuristic follows from 4.45 and is given by Inequality 4.54.

$$\left|\delta\mathbf{d}_{\hat{i}}\right| \leq 2 \cdot v \cdot G \quad \forall \hat{i} \in \{1, I-2\} \tag{4.54}$$

The second heuristic is found by summing all equations. Using matrix A, we see that summing all equation (analogous to summing all rows) leaves Equation 4.55, from which we conclude the second heuristic in Inequality 4.56.

$$\sum_{\hat{i}}^{I-2} \delta \mathbf{d}_{\hat{i}} = -O_i + O_{i+1} + O_{I-1} - O_I \tag{4.55}$$

$$\left| \sum_{\hat{i}}^{I-2} \delta \mathbf{d}_{\hat{i}} \right| \leq 2 \cdot v \cdot G \tag{4.56}$$

The first heuristic tells us that large differences in consecutive segment lengths can be a problem for achieving coordination (as formulated in this thesis). The second heuristic tell us that arterials where the segments continually increase (or decrease) in one direction can be a problem for achieving coordination (as formulated in this thesis).

So far in this section, we have only focused on the geometric requirement that arises when coordinating the (nearly simultaneous) starts of green in two directions. Satisfying this requirement says nothing about the final obtained cycle time. For this reason LP1 is extended to LP2 as follows.

4

LP2: Coordination and cycle time for coordination with a fixed speed in Subproblem 1

Objective function:

$$\min_{G^*} \quad z = G^*$$

Subject to:

$$\begin{aligned} A \cdot \mathbf{O} &= \frac{2}{v} \cdot \delta \mathbf{d} \\ -G^* - O_i &\leq 0 \quad \forall i \in \{1, I\} \\ -G^* + O_i &\leq 0 \quad \forall i \in \{1, I\} \\ -O_1 + O_2 &\leq \frac{2 \cdot d_1}{v} - C_q \end{aligned}$$

Variables:

$$\begin{aligned} O_i &\in \mathbb{R} \quad \forall i \in \{1, I\} \\ G^* &\geq 0 \end{aligned}$$

Input parameters:

$$v,\ \delta \mathbf{d},\ C_q$$

The outcome of LP2 will show whether coordination is possible with a fixed speed for the given network cycle time C_q. In LP2, only one segment (here the first, but it could be any) is included as a constraint for the cycle time (the final constraint in LP2). The matrix equation (the first constraint in LP2) will make sure that all segments have equal cycle times. Equation 4.57, in the next subsection, gives a formula for loose upper bound of the cycle time. The network cycle time to fulfill the demand should be feasible to achieve on the shortest segment.

VARIABLE SPEED

For the variable speed it was already concluded in section 4.2.1 that, geometrically, coordination between the simultaneous starts of inbound and outbound greens can be provided for many sets of distances. This will also be true when the starts of outbound and inbound green are not simultaneous, but are allowed to start with a time difference of G seconds as well. This only allows more possibilities for providing coordination between the starts of green (not constraining it in any way). The geometric effects of nearly simultaneous starts of outbound and inbound green are therefore not further investigated.

Regarding the cycle time, larger cycle times may be possible, dependent on the value of G. Using the same logic as section 4.2.1, Inequality 4.39 can now be transformed. Including the bounds on O_i from Inequality 4.40, it can be written as Inequality 4.57.

$$C_q \leq \frac{2 \cdot d_{min}}{l \cdot v_{max}} + 2 \cdot G \tag{4.57}$$

However, unlike Inequality 4.39 in section 4.2.1, which provides a tight upper bound, Inequality 4.57 only gives a loose upper bound for the cycle time. This means that when Inequality 4.57 is satisfied, it doesn't mean that the associated C_q is actually feasible. The loose upper bound indicates that when 4.57 is not satisfied, we know that the respective cycle time is definitely not feasible by the variable speed. The reason for this loose upper bound is that it may not be possible to have maximum internal time differences of G on the two intersections that neighbour the shortest segment in the network. The time difference allowed by G only increases the maximum allowed cycle time that satisfies the coordination between starts of green. This increase can not be larger than at most $2 \cdot G$.

In section 4.2.1 we found cycle times of 60, 72 seconds for $l = 0.6, 0.5$, respectively. Using the same values ($d_{min} = 250$ m and $v_{max} = 13.89$ m/s), dependent on the size of G (or, more generally O_i and O_{i+1}), for example when G is 6 seconds, cycle times as large as 72 and 84 seconds may be allowed. 6 seconds is a common minimum fixed green time in the Netherlands (CROW, 2014). When two directions are present that conflict with one of the coordinated directions, and these directions have a green time of 6 seconds, the starts of the coordinated green windows can differ by 6 seconds, without losing much efficiency in the locally optimal signal timing plan. Equation 4.57 can also be used for a fixed speed, by choosing $l = 1$. For the same values, only low cycle times (below 48 seconds) are possible with the fixed speed.

4.2.3. COORDINATION IN TWO DIRECTIONS: SUB-PROBLEM 3 AND GENERALIZATION

This section considers cases where it is realistic that the green times for the coordinated directions need not start simultaneously at all in a locally optimal timing plan. For these cases, the internal offset O_i can take any value and this means that it is geometrically possible to provide coordination between the starts of green with the fixed speed. Only the cycle time constraint potentially makes the coordination of the starts of green infeasible (when the cycle time required for coordination is smaller than the cycle time required for the demand).

Sub-problem 3 may occur in practice when multiple conflicts are present that conflict with only one of the main directions and when these conflicts have a substantial demand compared to the coordinated directions. Solving this type of system can be done in the same manner as in section 4.2.2, since the problem was approached and solved for any G.

Now, to reflect the additional room a larger upper bound for G can be applied. Since the green windows of the coordinated directions are not required to start (nearly) simultaneously, but do need to be given green in the same cycle, the internal offset can take any value smaller than the cycle time. This is reflected through equation 4.58, where the bound is set as the cycle time (i.e. $G = C_q$). C_q is the network cycle time needed to fulfill the demand, the actual cycle may be higher. However, ideally, the actual cycle time is not higher, and this makes C_q a suitable upper bound to use for solving sub-problem 3. Reasons for wanting as a low a cycle time as needed to serve the demand are given in section 1.1.2.

$$|O_i| \leq C_q \quad \forall i \in \{1, I\} \tag{4.58}$$

To solve for the coordination of the starts of green in combination with the cycle time, the LP2 program from section 4.2.2 can be used. The found G^* can now be compared to C_q (instead of to G) to check validity of coordination.

GENERALISATION

All the knowledge obtained from the previous sub-problems can be combined in a simple linear program that allows to examine the feasibility of coordination of the starts of green in two directions, with a fixed speed, for any combination of requirements regarding the simultaneous start of outbound and inbound green windows. Namely, this is achieved by allowing G in the previous equations to vary per intersection, denoting it as G_i. Using G_i, the available space for the internal offset at each intersection can take a different value.

4

LP3: Coordination and cycle time for a combination of various intersection layouts with a fixed speed.

Objective function

$$\max_{r} \quad z = r$$

Subject to:

$$\begin{aligned} A \cdot \mathbf{O} &= \frac{2}{v} \cdot \delta \mathbf{d} \\ r - O_i &\leq G_i \quad \forall i \in \{1, I\} \\ r + O_i &\leq G_i \quad \forall i \in \{1, I\} \\ -O_i + O_{i+1} &\leq \frac{2 \cdot d_i}{v} - C_q \quad \forall i \in \{1, I-1\} \end{aligned}$$

Variables:

$$\begin{aligned} O_i &\in \mathbb{R} \quad \forall i \in \{1, I\} \\ r &\geq 0 \end{aligned}$$

Input parameters:

$$\begin{aligned} v,\ \delta \mathbf{d},\ C_q & \\ G_i & \quad \forall i \in \{1, I\} \end{aligned}$$

It is not possible to draw further generic conclusions based on LP3. Feasibility of the problem highly depends on the input parameters. However, this program can quickly indicate if a feasible solution exists for the given cycle time, segment distances and internal offset bounds. The objective of LP3 is to maximize r, a variable which can be interpreted as the freedom in internal offset used for the coordination on the normative intersection. Maximizing r will make sure that, if a feasible solution is obtained, it is the solution where the least amount of freedom in internal offset is used on the normative intersection. In section 7.7, LP3 is put to the test on a network from practice.

4.3. CONCLUSIONS

This section presents an overview of the findings of the theoretical analysis of the variable speed. Section 4.1 looked at coordination with a variable speed under different cycle times to answer research question **1.1**: *To what extent does applying a variable speed theoretically allow different cycle times per intersection for the coordination of signal timing plans in one direction?*

The findings of section 4.1.1, summarized in section 4.1.2, conclude that there are only few cases where the variable speed provides significant relative improvement regarding the coordination of the starts of green, with different cycle times. When there is a large relative improvement, in absolute terms, the fraction of cycles that are coordinated is not high enough to make coordination under different cycle times very attractive. Further conclusions depend on the question whether a lower cycle time is worth skipping the coordination in a significant fraction of the cycles. Equations 4.17, 4.18, 4.19 and 4.20, that can be used to compute the percentage of cycles in which the start of green can be coordinated, are applicable in general, when it is allowed to skip coordination in the other cycles and when the assumptions mentioned in section 4.1.2 hold. One assumption is particularly relevant for the conclusion. Namely the requirement that the downstream cycle time is larger than the upstream cycle time. The formulas and conclusions only hold for the cases where the downstream cycle time is larger. Similar conclusions are expected for the mirrored case, but not investigated.

Section 4.2 looked at the feasibility of coordination in two directions to answer research question **1.2**: *In which cases does applying a variable speed theoretically allow the coordination of locally optimal signal timing plans in two direction, compared to a fixed speed?*

Coordinating the start of green in two directions is not always feasible with locally efficient timing plans, since the coordinated directions and common cycle time put constraints on the start of green times for coordination. It was found that the variable speed provides more opportunities in terms of the feasibility of the coordination of the starts of green in cases where the outbound and inbound green windows start (nearly) simultaneous, over the fixed speed. The provided opportunities by the variable speed over the fixed speed are largest and most impactful in constrained systems, where the two coordinated directions are part of the same stage and start at exactly the same time. This was shown in section 4.2.1. Namely, it was found that the variable speed can coordinate the starts of green for many sets of realistic segment lengths, even when the outbound and inbound green must start simultaneously. In these cases the fixed speed can only provide coordination between the starts of green when segments are equidistant. The conclusions drawn from the analysis are subject to a major (simplifying) assumption, the main one being that coordination happens between the start of green of the green windows. Upstream, this assumption has no large implications, normally the start of green upstream should be part of the coordination. Downstream, this assumption has large implication and makes it so that, for example, the presence of queues is not considered. In general, it must be kept in mind that all findings of this chapter only regard coordination between the starts of green.

When an efficient local timing plan does not require that the starts of outbound and inbound greens are simultaneous, the coordination is more often feasible under a fixed speed. Thus, when more directions that conflict only with one of the two coordinated directions are present and/or there is more demand for these directions relative to the coordinated directions, the coordination of the starts of green is more often feasible with the fixed speed. This was shown in sections 4.2.2 and 4.2.3, by developing a linear program. This program can be used to solve for the feasibility of coordination of the starts of green with a fixed speed, when the outbound and inbound green are to some degree simultaneous.

Showing when coordination is feasible or not feasible with an efficient timing plan does not conclude the extent of the usefulness of the variable speed. Namely, the theory shows that the variable speed provides more options and flexibility. Even when coordination with both the variable speed and fixed speed is feasible in an efficient timing plan, the variable speed may still be used for a coordination that allows for the inclusion of other traffic phenomena. Throughout this chapter, alignment of the start of green was assumed to provide coordination. In practice, more complex phenomena like standing queues, platoon dispersion and the position of left-turn greens can affect the quality of the coordination. The flexibility of the variable speed could be used to account for these effects, while still providing coordination with efficient signal timing plans.

Besides assuming coordination between the start of green, the conclusions of section 4.2 are based on some other assumptions as well. Firstly, the (fixed) internal time difference O was defined as a continuous variable. In reality, this is not a continuous variable, not any value would lead to efficient timing plans. Dependent on the control structure, length of the other green phases and clearance times, only some combinations of specific values for O lead to efficient timing plans. A more constrained O is only a further constraint on the feasibility of coordination of the starts of green with a fixed speed.

Secondly it was assumed, from section 4.2.2 onward, that all $n_{\hat{i}}$ where equal to 1. $n_{\hat{i}}$ represent an integer, greater than or equal to one, that is normally included to allow coordination with half, or a third, or a fourth, etc. of the network cycle time. This assumption is only relevant when either very long segments or a very short cycle time is at play at a certain intersection. When this is the case, it must be remembered that the linear programs in sections 4.2.2 and 4.2.3 cannot be used for conclusions about the feasibility in cases with unequal $n_{\hat{i}}$.

Overall, the theoretical findings about the usefulness of a variable speed are promising regarding coordination in two directions. Even in cases where a feasible solution can be obtained under a fixed speed, the additional possibilities from the variable speed may be used for a more practical coordination that is better able to account for things like standing queues, positions of left-turn greens, coordination considering the full green window and platoon dispersion. Including these affects in a theoretical analysis similar to the one performed in this chapter is too broad for the scope of this thesis. However, traffic models are able to derive a more practical coordination which include these effects. To continue research regarding the uses of the variable speed for a coordination in two directions, the next chapter will focus on a traffic model suitable for including the proposed variable speed. This model will be used to be able to create practical timing plans for a simulation.

5

MODEL FORMULATION

In chapter 4 it was found that the proposed variable speed allows to satisfy the geometric constraints of coordinating the starts of green between locally optimal timing plans in two directions, more easily. Some cases are quite clear: when the main directions are required to have green at the exact same time, with a fixed speed often large compromises have to be made regarding the coordination. In section 4.2.1, it was found that the variable speed, which introduces a compromise in the sense of reducing the speed on some segments, often does allow for coordination with both directions having green at the same time.

When a lot of conflicts (with relatively high demand compared to the main directions) are present that conflict with only one of the two main directions, the fixed speed can more often provide a feasible coordination, with fewer compromises. For these cases, the additional opportunities provided by the variable speed are not expected to be very impactful.

Within these two extremes, there are systems where it is unknown if the variable speed is worthwhile. Namely, coordination in two directions between locally optimal timing plans may be possible with a fixed speed, but perhaps only in limited configurations and with small compromises. For the same case, a variable speed may allow more configurations, with fewer compromises. This could result in a more effective coordination, bringing benefits in terms of improved flow or reduced stops. To realize such benefits, the inclusion of more traffic phenomena, such as left-turn green positions, queue clearance times or platoon dispersion is required. Most importantly, coordination with some part of the green window (not only the start) should be considered as well. To include all of these effects in optimization of a large network, where the number of factors and possibilities is extremely large, traffic models can be used. The goal of this chapter is to have a model ready that is capable of optimizing for the variable speed, so that the application of the variable speed can be tested in a simulation. The development of the model gives the answers to research questions **1.3**: *How can the variable speed be included in a model, such that the variable speed is used to optimize the coordination?*

5.1. VARIABLE SPEED

Two of the most well-documented models are TRANSYT and MAXBAND. The workings of both models were previously discussed in Section 2.1.1. From there workings, it can be considered which of the two is most suitable for including the proposed variable speed, which will be done in more detail, in this section.

In TRANSYT, technically, the most logical place for including it would be within the platoon dispersion model. There, the link travel time in number of time steps could be adapted to reflect a different speed. However, the only way to optimize for this variable speed would be to perform multiple optimizations where multiple different settings of the link travel times are tested. Dependent on the allowed range in the variable speed and the size of the network, this results in a lot of optimizations. Then there is still the issue that TRANSYT requires timing plans with a predetermined phase order. The model will not be capable of optimizing the timing plans (phase orders) and speeds simultaneously, thus it is likely that TRANSYT will not be able to effectively use the variable speed in the model.

A different approach is taken in (van Leersum, 1985) who include the speed more in hindsight. In their formulation, TRANSYT is run unmodified. After optimization, the offset values are compared against possible offset values allowed by a variable speed. Based hereon, adaptations are made to the flow profiles. Only the flows on time steps that can be positively affected by a change of speed, are changed. However, this approach is not in line with the proposed variable speed, which is of a macroscopic nature. A change in segment speed should affect all propagation on the link. Even when this approach is modified to reflect the proposed variable speed, still the issue of predetermined phases persists.

MAXBAND is a more suitable model for including the variable speed. The model is capable of optimizing for the speed, positions of green and cycle time simultaneously, without the need for any adaptations. The only major downside is that no side directions are included in the optimisation. However, the model can very effectively optimize for the main directions, including effects of left-turn placement and queues. The side directions can be added manually afterwards and when this is done consistently between a reference (fixed speed) simulation and a variable speed simulation, the final results should be reliable.

Overall, despite a TRANSYT program being available at Vialis, the choice is made to use MAXBAND to further deepen the research into the variable speed. A MAXBAND program was created in python, based on the formulation in literature. Some extensions are introduced to the model to increase it's capabilities. Other MAXBAND variations, like MULTIBAND or PASSER could just as well have been chosen. The choice for MAXBAND was made as it has the desired properties and is the simplest model. The goal of the research is not to develop the best or most advanced model for the variable speed, we simply aim to investigate the benefits over a fixed speed. With this in mind, using MAXBAND to determine both the fixed and variable speed coordination should result in a fair comparison.

The python program implements the MAXBAND model as presented in the 1981 paper by J. D. C. Little, M. D. Kelson and N. H. Gartner (Little et al., 1981). Both the LP1 and the more advanced LP2, as formulated in section 5.2 formulations are included in the program.

5.2. MAXBAND

MAXBAND (Little et al., 1981) is formulated as a Mixed Integer Linear Program (MILP). The mathematical formulation is as follows:

Objective function:

$$\max_{b,\overline{b}} \quad z = (b + k\overline{b}) \tag{5.1}$$

Subject to:

$$\begin{cases} b = \overline{b} & \text{if } k = 1 \\ (1-k)\overline{b} \geq (1-k)kb & \text{if } k \neq 1 \end{cases} \tag{5.2}$$

$$c \leq \frac{1}{C_{min}} \tag{5.3}$$

$$-c \leq -\frac{1}{C_{max}} \tag{5.4}$$

$$\left.\begin{aligned} w_i + b &\leq 1 - r_i \\ \overline{w_i} + \overline{b} &\leq 1 - \overline{r_i} \end{aligned}\right\} \forall i \in \{1, \ldots, I\} \tag{5.5}$$

$$(w_i + \overline{w_i}) - (w_{i+1} + \overline{w_{i+1}}) + (t_i + \overline{t_i}) + \delta_i l_i - \overline{\delta_i l_i} - \delta_{i+1} l_{i+1} + \overline{\delta_{i+1} l_{i+1}} - m_i = (r_{i+1} - r_i) + (\overline{\tau_i} + \tau_{i+1}) \ \forall i \in \{1, \ldots, I-1\} \tag{5.6}$$

$$\left.\begin{aligned} \frac{d_i}{v_{max}} c \leq t_i \leq \frac{d_i}{v_{min}} c \\ \frac{\overline{d_i}}{\overline{v_{max}}} c \leq \overline{t_i} \leq \frac{\overline{d_i}}{\overline{v_{min}}} c \end{aligned}\right\} \forall i \in \{1, \ldots, I-1\} \tag{5.7}$$

$$\left.\begin{aligned} \frac{d_i}{h} c \leq \frac{d_i}{d_{i+1}} t_{i+1} - t_i \leq \frac{d_i}{g} c \\ \frac{\overline{d_i}}{\overline{h}} c \leq \frac{\overline{d_i}}{\overline{d_{i+1}}} \overline{t_{i+1}} - \overline{t_i} \leq \frac{\overline{d_i}}{\overline{g}} c \end{aligned}\right\} \forall i \in \{1, \ldots, I-2\} \tag{5.8}$$

Variables:

$$b, \overline{b}, c \geq 0$$
$$w_i, \overline{w_i} \geq 0, \ \forall i \in \{1, \ldots, I\}$$
$$t_i, \overline{t_i} \geq 0, \ \forall i \in \{1, \ldots, I-1\}$$
$$m_i \in \mathbb{Z}, \ \forall i \in \{1, \ldots, I\}$$
$$\delta_i, \overline{\delta_i} \in \{0, 1\}, \ \forall i \in \{1, \ldots, I\}$$

5

With:

$b(\overline{b})$ = the bandwidth on the outbound (inbound) direction [cycles]

c = the signal frequency (1/cycle time) of all TLC's in the network [cycles/s]

$w_i(\overline{w_i})$ = time as fraction of the cycle time from the right (left) side of red at TLC i to the left (right) side of the outbound (inbound) green band [cycles]

$t_i(\overline{t_i})$ = the outbound (inbound) travel time between TLC$_i$ and TLC$_{i+1}$ as fraction of the cycle time [cycles]

$\delta_i(\overline{\delta_i})$ binary variables used to decide whether the outbound (inbound) left-turn should lead (=0) or lag (=1)

m_i = integer offset in number of cycles at TLC$_i$ [#]

A graph showing all the variables, their relation and meaning is included in Figure 5.1.

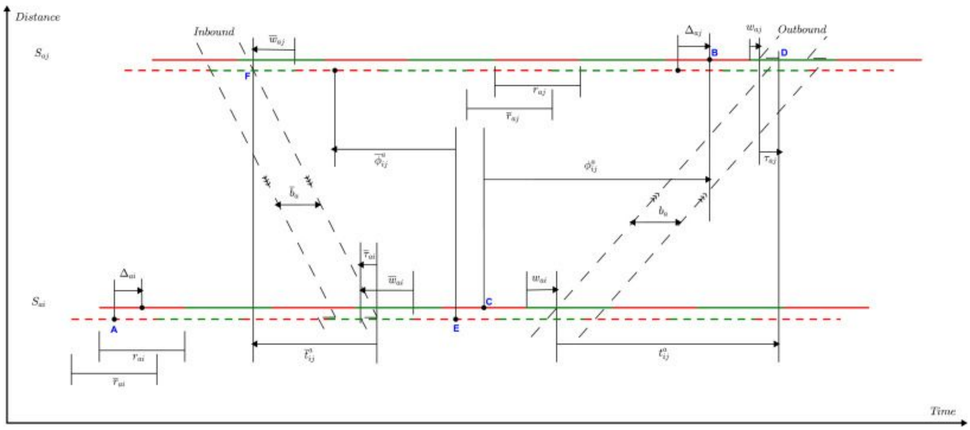

Figure 5.1: MAXBAND variable definition. Image downloaded on September first, 2022, from (Cabezas et al., 2019)

And with input parameters:

I = the number of Traffic Light Controllers (TLC's) on the arterial.

k = a weight used to favor the outbound (if < 1) or the inbound (if > 1) direction.

C_{min}, C_{max} = the minimum, maximum values of the cycle time [s].

$r_i(\overline{r_i})$ = the outbound (inbound) red time at TLC$_i$ as fraction of the cycle time [cycles].

$l_i(\overline{l_i})$ = the green time of the outbound (inbound) left-turn at TLC$_i$ as fraction of the cycle time [cycles].

$\tau_i(\overline{\tau_i})$ = the outbound (inbound) queue clearance time at TLC$_i$ as fraction of the cycle time [cycles]. This time is needed to clear the standing queue, before the platoon arrives.

$d_i(\overline{d_i})$ = the outbound (inbound) distance between TLC$_i$ and TLC$_{i+1}$ [m].

v_{min}, v_{max} = minimum, maximum speed allowed on the outbound and inbound direction [m/s]. Note that, contrary to the paper, the bounds of the speed are invariant with respect to the segment or outbound and inbound direction. The choice for this slightly different formulation is made, because it saves some effort in specifying the inputs, while maintaining the desired capabilities for the research of this thesis. The same holds for the bounds of the reciprocal speed difference between segments.

h, g = lower and upper limits on the reciprocal speed difference allowed on the outbound and inbound direction such that $\frac{1}{h} \leq \frac{1}{v_{i+1}} - \frac{1}{v_i} \leq \frac{1}{g}$ [s/m]. This formulation is needed to keep the constraints in the problem linear w.r.t. time.

The MILP is solved with the Pulp module in python which uses the CBC solver (Forrest & Hafer, 2018). The only major feature from the paper by Little et al. (1981) that is not implemented, is the loop constraint. This constraint is needed to accurately deal with networks that contain a loop. For this thesis, this part of the program was not necessary and therefore left out. A basic program output is shown in Figure 5.2.

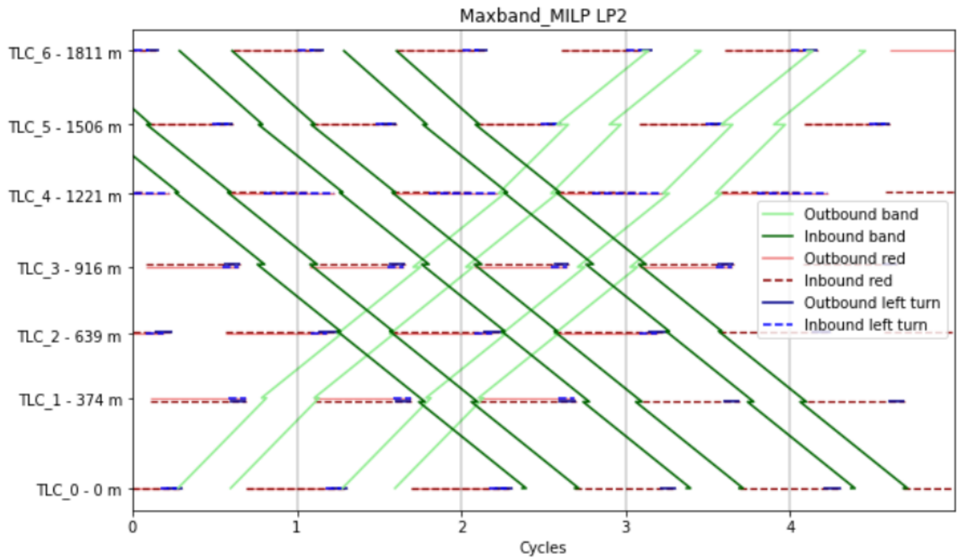

Figure 5.2: Typical output of a MAXBAND LP2 program run.

5.2.1. MAXBAND MODEL EXTENSIONS

Based on the feedback received from various traffic engineers at Vialis, the program was further extended to include more fine-tuning capabilities. Using these extensions, multiple timing plans can be created and compared. A selection can be made both visually and based on their output characteristics. These extensions can be summarized as follows;

- A way to enforce that the start of green at the first intersection of the arterial is always part of the band. This is easily added as a constraint, shown by Equality 5.9 for the outbound direction and Equality 5.10 for the inbound direction.

$$w_0 = 0 \tag{5.9}$$

$$\overline{w_n} = 1 - \overline{r_n} - \overline{b} \tag{5.10}$$

- A way to make sure that the downstream intersection also allows enough green for the tail of the band, when the band is shifted forward to include the queue clearance time. This is added by changing the constraints of Equality 5.5 to Equality 5.11.

$$\left.\begin{aligned} w_i + b \leq 1 - r_i - \tau_i \\ \overline{w_i} + \overline{b} \leq 1 - \overline{r_i} \\ \overline{w_i} \geq \overline{\tau_i} \end{aligned}\right\} \forall i \in \{1, \ldots, n\} \tag{5.11}$$

- Similarly to the previous extension, it could be desired that the downstream intersections allow enough green to account for all the upstream shifts. This way, any cleared queue becomes part of the band for the entire length of the arterial. To realize this the constraints of Equality 5.5 are changed to Equality 5.12.

$$\left.\begin{aligned} w_i + b \leq 1 - r_i - \sum_{1}^{i} \tau_i \\ \overline{w_i} + \overline{b} \leq 1 - \overline{r_i} \\ \overline{w_i} \geq \sum_{i}^{n} \overline{\tau_i} \end{aligned}\right\} \forall i \in \{1, \ldots, n\} \tag{5.12}$$

- The previous extensions are related to the position and or shape of the band. However, it is also useful to have more control over the entire green window (not just the green that is part of the band). Especially to use the available green time more effectively. The green between the end of the red and the start of the band can be allowed to only increase when going downstream the arterial. This will create the effect that, when going downstream, any available green time is put in front of the band, but no more than the intersections further downstream can also put in front of the band. The idea is that, when possible, the downstream green time is added in front of the band and becomes part of coordinated green time for the rest of the arterial. This is especially useful when the size of the platoon increases along the arterial, for example by turning traffic onto the arterial. Upstream, any additional available green time is more likely located after the band. The constraint is formulated in Equality 5.13.

$$\left.\begin{aligned} w_i - w_{i+1} \leq 0 \\ \overline{w_{i+1}} - \overline{w_i} \geq \overline{r_i} - \overline{r_{i+1}} \end{aligned}\right\} \forall i \in \{1, \ldots, n-1\} \tag{5.13}$$

All model extensions are realized in the form of additional or more restrictive constraints. It should be noted that these will result in smaller bandwidths and that these will more often lead to the program not being able to find a feasible solution.

In the end, the choice for the best suitable timing plan should be based on the interpretation and insights of the user of the program.

The program is made publicly available on my personal GitHub page at: MAXBAND

5

5.3. USING MAXBAND

To prepare for a set of simulations in which the variable speed is tested, the MAXBAND model, including the extensions discussed in section 5.2.1, is put to use. The model input consists of data from practice, from an arterial in the Netherlands. The specific arterial and more details about the simulation are discussed in chapter 7. The steps taken in specifying the input parameters are discussed in detail in Appendix B.

In the specific network considered during the simulation, it was found that the model outputs often didn't result in realistic signal timing plans. This, despite setting the minimum cycle time high enough and providing sufficient red time and left-turn time. The limitation of MAXBAND, namely that it cannot include side-directions, means that the produced output is not always fully realistic. Furthermore, the program doesn't account for clearance times, lost times or yellow times. When converting the output to an actual signal timing plan, this often resulted in infeasible plans, where there was not enough room available for the side directions.

It was found that especially the left-turn patterns were of a big influence. In the MAXBAND model, the program is free to choose from four left turn patterns, doing so with the idea to maximize the bandwidth. However, the left-turn on the main directions conflict with both side directions. When the patterns lead-lag or lag-lead were selected, this often resulted in an infeasible situation at the specific intersection.

For this reason, more extensions (optional constraint) were added to customize the left-turn patterns. By constraining the binary decision variables, any desired combination of patterns can be obtained. Namely a zero represents that a direction 'leads' and a one represents that a direction 'lags'. When, for example, requiring that the sum of both left turn variables at an intersection is two, it essentially says that only the lag-lag pattern is allowed. For a more detailed look, please refer to the explanation presented in Little et al. (1981). Only the following three options were included in the program, as these were desired to use for the considered case study:

- Constraint 5.14 allows only the lead-lag or lag-lead patterns.

$$\delta_i + \overline{\delta_i} = 1 \tag{5.14}$$

- Constraint 5.15 allows only the lead-lead or lag-lag patterns.

$$\delta_i - \overline{\delta_i} = 0 \tag{5.15}$$

- Constraint 5.16 allows only the lag-lag pattern.

$$\delta_i + \overline{\delta_i} = 2 \tag{5.16}$$

Using 5.15 resulted in more feasible timing plans. However, after some more simulation tests, it was found that (for this specific case), the lag-lag pattern performed better than the lead-lead pattern. The demand for the left-turns in the considered network came from traffic of the main directions. So by choosing the lag-lag pattern, the waiting time for these left-turns is minimized, as just after the main directions have terminated their green (and supplied the demand to the left-turns), the left-turns are resolved. Lagging left-turns could result in an issue, when there is a lot of demand, a queue and not enough storage space in the left-turn lane. This potentially causes blocking back of the main direction and disruption of the coordination when the lag pattern is selected. In the considered network, these issues are not present. However, the example shows that such decisions are left to the traffic engineer using MAXBAND, as the program itself does not account for such effects.

From some more simulation tests, it was observed that although the demand on the main directions is very high, the demand for traffic that traverses the entire arterial is substantially lower in the considered case. A lot of traffic enters and leaves the 'band'. For this reason, the choice was made to not include the optional constraints that account for the queue clearance time of upstream intersections. Furthermore, when including the left-turn lag-lag constraints, also including the constraints formulated in Equality 5.13, resulted in infeasible problem formulations.

So finally, the base program, with the additional Constraints 5.9, 5.10 and 5.16 were used to obtain the MAXBAND output that was used to test the variable speed. The output plot is shown in Figure 5.3.

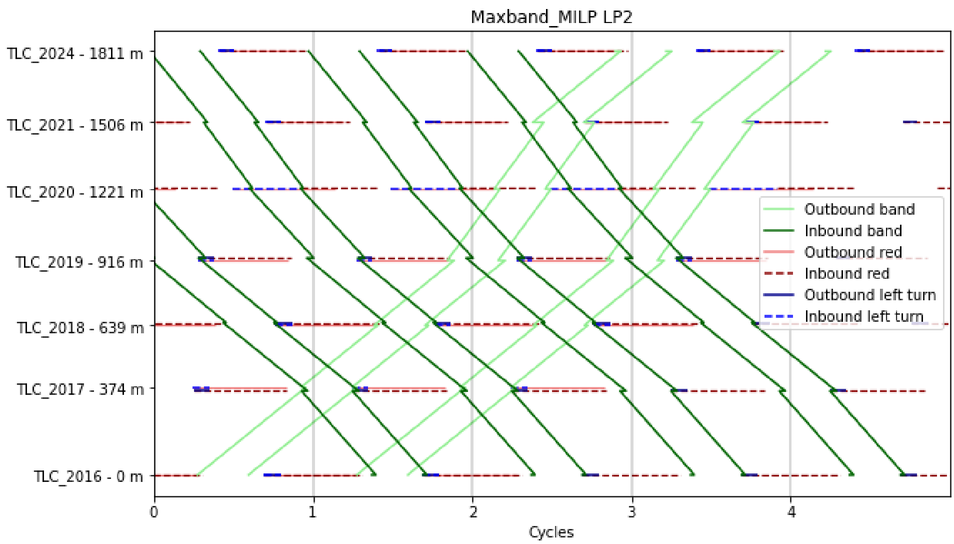

Figure 5.3: MAXBAND optimization output with a variable speed . The obtained cycle time is 66 seconds and bandwidth is 21.1 seconds.

To have as objective a comparison as possible, the fixed speed timing plans were obtained by running the program again while only changing a single input parameter: the minimum speed is now set equal to the maximum speed (50 kilometres per hour). This gave the output as shown in Figure 5.4.

The output values of MAXBAND for the outbound variable speeds per segment are [30, 30, 30, 50, 50, 30] [km/h] and for the inbound variable speeds per segment are [41.8,

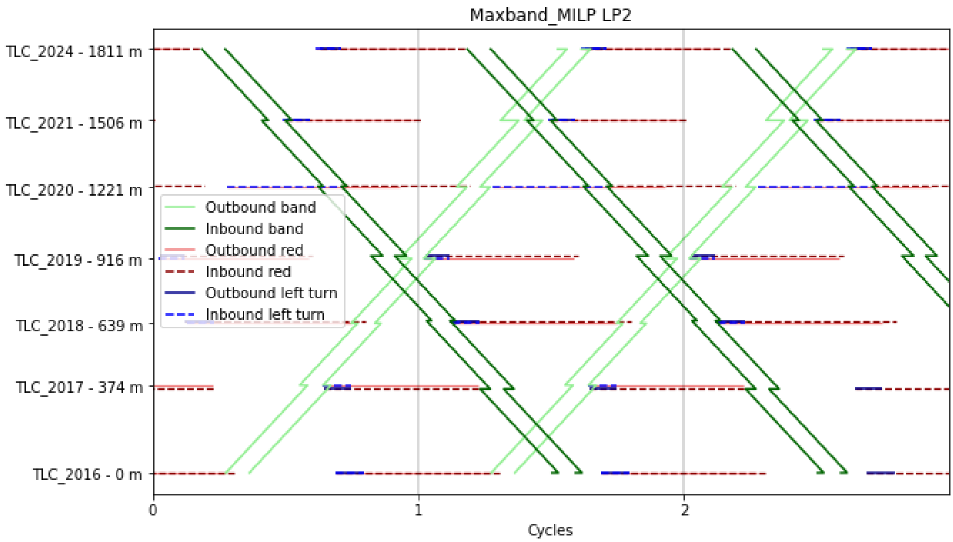

Figure 5.4: MAXBAND optimization output with a fixed speed. The obtained cycle time is 87 seconds and bandwidth is 7.8 seconds.

30, 30.6, 40.1, 50, 45.3] [km/h]. The output values for the fixed speed are, of course, 50 kilometers per hour on every outbound and inbound segment.

From Figures 5.3 and 5.4, it is evident that the variable speed allows for a lot more bandwidth. The additional flexibility provided by the variable speed, allows to include the same constraints in the program as the fixed speed, but for a much lower cycle time and more bandwidth. The cycle time with variable speeds is 66 seconds and the bandwidth is 21.1 seconds. For the fixed speed, the optimal cycle time found by MAXBAND is 87 seconds with corresponding bandwidth of 7.8 seconds. Of course, the performance of the timings plans and coordination cannot be judged from simply the space-time diagram. Therefore, both outputs are converted (using exactly the same methodology for both conversions) to semi-fixed signal timing plans and these are evaluated in a simulation. The steps taken in this conversion are discussed in more detail in Appendix B. The results of this simulation can be found in chapter 8.

In the investigated case, indicated by Figures 5.3 and 5.4, the flexibility provided by the variable speed resulted in more efficient timing plans for coordination (having more bandwidth and a lower cycle time), compared to the fixed speed. In the following two subsections two other (fictional) cases will be discussed, where the variable speed is relatively less impactful compared to the fixed speed regarding efficient timing plans for coordination. These cases are intended to highlight the factors that influence the relative usefulness of the variable speed (compared to the fixed speed) the most: Segment lengths and allowed internal time difference between outbound and inbound coordinated directions.

MAXBAND OUTPUT IN A FICTIONAL NETWORK WITH EQUAL SEGMENT LENGTHS

Figures 5.5 and 5.6 show the MAXBAND output given the exact same input as Figures 5.3 and 5.4, but with just two exceptions:

1. The length of all segments was set to 500 meters. As discussed in section 4.2.1, in a network with equidistant segments, coordination between locally efficient timing plans is also feasible with a fixed speed. The minimum cycle time to fulfill the demand is 66 seconds, Equation 4.39 was used in reverse to find that segment lengths should ideally be larger than about 460 meters, hence the choice for 500 meters for this fictional network.

2. The required left-turn lag-lag pattern was disabled. This constraint is relatively a lot more constraining for the fixed speed than for the variable speed, because variations in the left-turn pattern can no longer be used to improve the coordination with a fixed speed. Disabling this constraint is comparable to increasing the value of G. As discussed in section 4.2.2, this allows for more room in internal offset between the outbound and inbound coordinated directions. In this case, the effects are only minor, since the left-turn green times are short compared to the main direction green times.

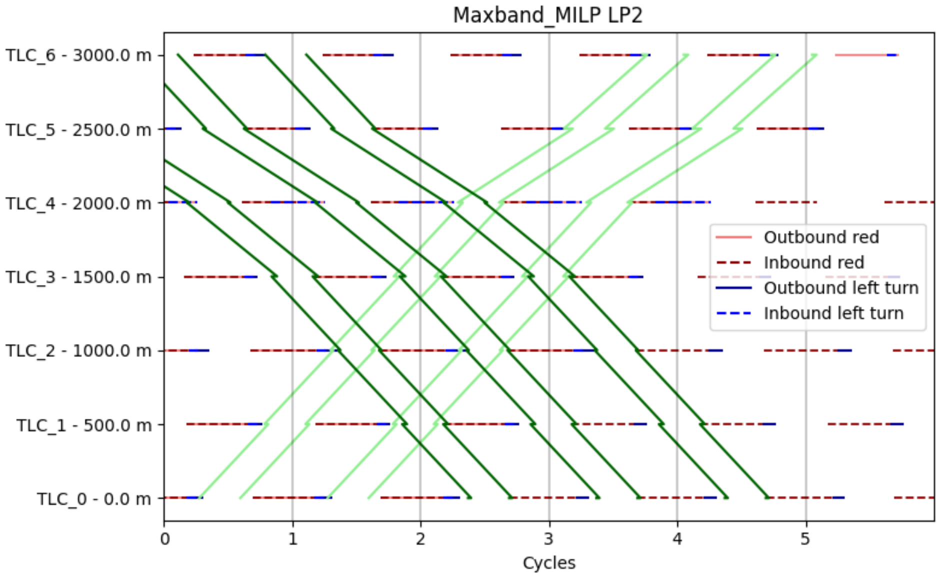

Figure 5.5: MAXBAND optimization output with a variable speed. The obtained cycle time is 67 seconds and bandwidth is 21.4 seconds. In this case, the length of all segments was chosen equal and all four left-turn patterns are allowed.

Comparing the outputs of Figures 5.5 and 5.5, it is evident that the outputs are pretty much identical in terms of the coordination between efficient timing plans. The Figures indicate that the flexibility of the variable speed is not needed and cannot be used to

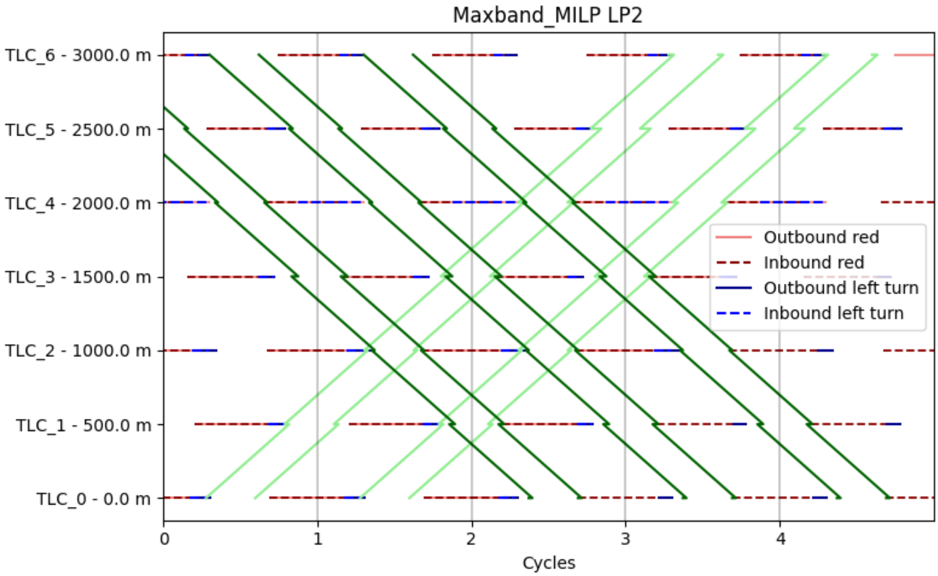

Figure 5.6: MAXBAND optimization output with a fixed speed. The obtained cycle time is 66 seconds and bandwidth is 21.3 seconds. In this case, the length of all segments was chosen equal and all four left-turn patterns are allowed.

improve the output in a significant way in this case. The findings with MAXBAND are in line with the findings of section 4.2. As differences between segment lengths increase, the benefits of a variable speed for coordination between efficient timing plans are expected to increase as well.

MAXBAND OUTPUT IN A FICTIONAL NETWORK WITH LARGE INTERNAL TIME DIFFERENCE BETWEEN COORDINATED DIRECTIONS

Figures 5.7 and 5.8 show the MAXBAND output given the exact same input as Figures 5.3 and 5.4, but with just two exceptions:

1. The duration of the inbound and outbound left-turns was increased by 0.1 [cycles]. On most intersections this roughly doubles the outbound and inbound left-turn green time.

2. Similar to the previous case, the required left-turn lag-lag pattern was disabled. In this case, disabling this constraint has a large effect on the efficiency of the timing plan with the fixed speed.

Comparing the outputs of Figures 5.7 and 5.8, it is evident that the the variable speed has resulted in a more efficient coordination than the fixed speed (indicated by the much lower cycle time, though slightly lower bandwidth). However, comparing Figures 5.4 and 5.7, a large improvement regarding the coordination with the fixed speed is observed (a much lower cycle time and much more bandwidth). This observation is in line with

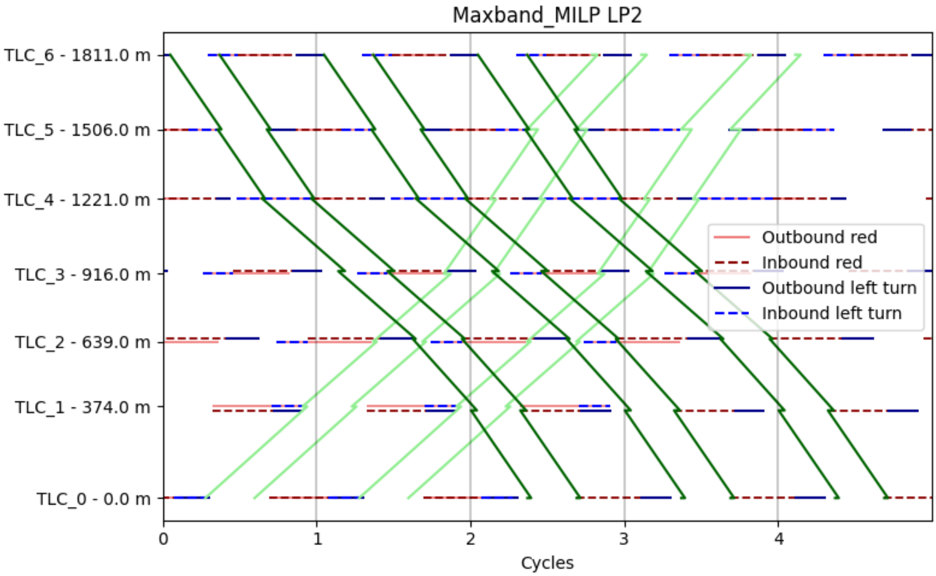

Figure 5.7: MAXBAND optimization output with a variable speed. The obtained cycle time is 66 seconds and bandwidth is 21.1 seconds. In this case, the duration of the outbound and inbound left turn is increased and all four left-turn patterns are allowed.

the findings of section 4.2. Allowing more internal time difference between outbound and inbound coordinated directions, increases the possibilities of coordinating between efficient timing plans with a fixed speed. Figure 5.8 clearly shows that outbound and inbound green windows do not start simultaneous at most intersections (anymore). In line with the findings of section 4.2, it is expected that with increasing flexibility regarding the internal time difference, the benefits of using a variable speed for efficient coordinated signal timings plans will decrease.

Besides these highlighted cases, there are other cases imaginable, where the coordination between efficient timing plans is also possible with the fixed speed, such that the variable speed provides no benefits in this regard. If the desired queue clearance times align just right, these can 'compensate' the differences in segment lengths somewhat. However, this is not true in general. In general, differences in segment lengths and the allowed internal time difference between outbound and inbound coordinated directions are the two most important factors in determining the relative usefulness of the variable speed over the fixed speed for a given network. Green time duration is another factor, though in a slightly different sense. When the duration of the coordinated green time windows on all intersections increases with the same amount, the bandwidth increases as well, in most cases with the same amount for the variable speed as for the fixed speed. This creates the effect that, relatively speaking, the usefulness of the variable speed can decrease with increasing green time duration. However, increasing green

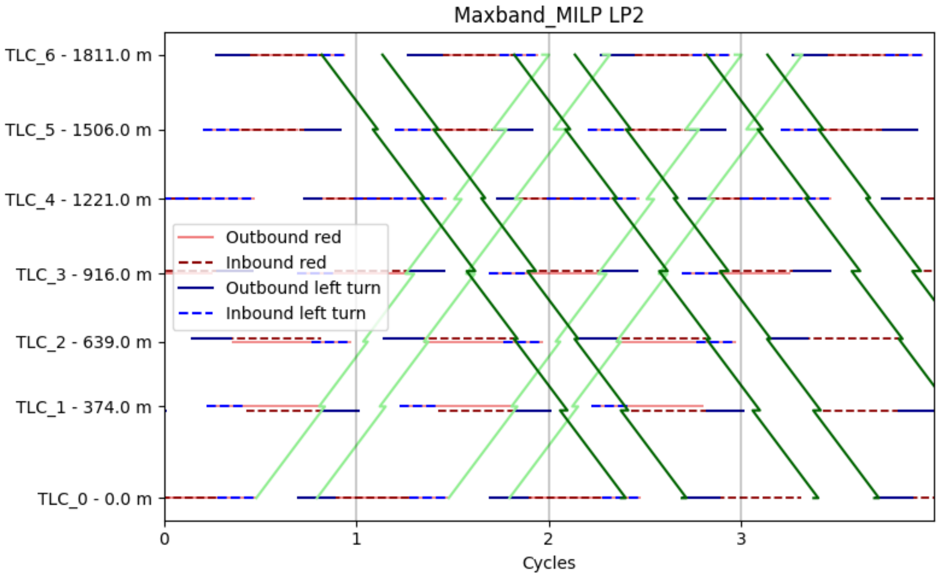

Figure 5.8: MAXBAND optimization output with a fixed speed. The obtained cycle time is 75.6 seconds and bandwidth is 23.8 seconds. In this case, the duration of the outbound and inbound left turn is increased and all four left-turn patterns are allowed.

time duration does not necessarily make it easier to create a coordination between efficient timing plans. Namely, increasing the green time reduces the efficiency (when this green time is not needed to fulfill the demand). Only in some cases, where the green time duration on some specific (normative) intersections is affected, may the efficiency of the coordination be improved. This report does not discuss such a case.

5.4. CONCLUSIONS

This chapter laid out and used the MAXBAND model to evaluate the usefulness of the variable speed, to answer research questions 1.3: *How can the variable speed be included in a model, such that the variable speed is used to optimize the coordination?*

The variable speed can be included in the MAXBAND model, described in section 5.2, as an optimizable variable. There are numerous trade-offs to consider, such as the speed difference between subsequent segments, the inclusion of queue clearance times and the freedom in the positions of the left-turn green times. The variable speed allows for a more efficient use of the available green time for coordination, especially when subject to additional constraints, when compared to the fixed speed.

The MAXBAND model is capable of optimizing with the variable speed and can be used on any arterial without modifications. Though, as section 5.3 discussed, an iterative process with tests and model runs is needed to get the most out of the model. For example, some arterials may benefit more from (dis)allowing certain left-turn patterns.

The model output is only a basis for coordinated signal timing plans. The model only gives the timings for the coordinated directions and the positions of the left-turns on the main arterial. The model accounts for left-turn green time, but doesn't account for clearance times or minimum green times, which are needed for realistic timing plans.

Using the MAXBAND model on some fictional cases has produced findings that are in line with the findings of section 4.2. The relative benefits of the variable speed over the fixed speed for coordination between efficient timing plans largely depends on the differences in segment lengths and the allowed internal time difference between outbound and inbound coordinated green windows.

Chapter 7 will discuss the simulation setup wherein the model is applied. Two simulation, one with a fixed speed and one with a variable speed will be compared. The results are discussed in chapter 8.

6

SETUP FOR TESTING THE POTENTIAL OF DEMAND PREDICTIONS

In this chapter, the methodology regarding the the tests of the potential of traffic demand predictions for TopTrac is further explained. This chapter provides the answer to research question 2.1: *How can tests of the potential of demand predictions of undersaturated demand be included in TopTrac in real-time?*

6.1. TECHNICAL SETUP

To be able to successfully test the potential of predictions in TopTrac, some technical challenges have to be overcome. Firstly, TopTrac itself runs on a server, where only limited access is available to the researcher. Changing any source code in TopTrac itself was deemed infeasible due to various reasons. TopTrac had to remain available (in it's standard form) on the server to other users. Furthermore, it is quite a challenge to actually deploy changes and construct a new build of TopTrac as many systems are interconnected with the TopTrac-service.

So, to be able to test predictions, a solution to these technical challenges had to be found. Since TopTrac receives it's traffic flow measurements from the local TLC's, an approach was adopted where the measurements within each TLC could be modified, without influencing the TLC's performance. The TLC's would read these new parameters every 2 seconds. This timing was found suitable, as it allowed for enough precision, without slowing down the simulation too much. By modifying the counts to reflect the desired predicted (testing) values, TopTrac would receive the desired values, without having to change anything on the server or in the source code. Any other users of TopTrac could simply select different (default) local TLC's and still be able to use an unmodified version of TopTrac on the server.

Supplying the desired flow measurements at the right time to the TLC's and thus to

TopTrac also proved to be a challenge. TopTrac optimizes for the coming two cycles. Since this optimization includes an optimization of the cycle time, the optimization period is of a variable length. Supplying measurements at fixed intervals will therefore not result in the desired behaviour, as during the simulation the timings of optimization cycles, as well as the duration of these cycles will vary.

To overcome this challenge, one of the TLC's was adapted such that it would write it's current cycle time to a text file every 2 seconds. By keeping track of the cycle time, the length of the coming optimization period can be calculated.

Another major challenge was posed by the start-up time of TopTrac. When a simulation is started, TopTrac is not directly active. The system first gathers some information and does some optimizations in the background (on the server) before it fully initializes. During this initialization period, the local TLC's simply use an adaptive or locally stored (uncoordinated) timing plan. In the mean time, TopTrac runs as normal, gathering data from the local TLC's and optimizing in the cloud. The challenge arises mainly by the fact that during the initialization period, it is unknown which cycle times are used by TopTrac. The optimizations are performed on the server and the cycle times (and timing plans) are not communicated back to the TLC's. A certain cycle time may be found optimal based on the measurements during the operation of the local adaptive or stored timing plans. This cycle time changes the duration of the following measurement period. This leads to the start timing being unknown and the length of the optimization period being unknown. Furthermore, the duration of the initialization period itself varies between simulations.

To overcome this challenge, the local TLC's were set to adaptive control during the initialization period and the counts that were to be send to TopTrac would be overwritten by the supplied predicted counts every (internal) loop of the TLC. The reasons being firstly, that during adaptive control, no fixed cycle time is used. Thus, when a fixed cycle time is used, we can be sure that it came from TopTrac and that TopTrac has been fully initialized. Secondly, by overwriting the counts, the internal mechanism, supposed to be counting for TopTrac, is overwritten. This means that we only need to supply a prediction, anytime after the previous and before the next optimization period (not necessarily exactly on time). By analyzing the logs of multiple simulations, a good estimate of the start-up time was obtained for each simulation seed, on the basis of the timing of the first noted cycle time. Through this estimate, finally the desired behaviour could be obtained and any desired flow measurement could be given at any desired second of the simulation. With the correct supply of flow measurements, this test setup is sufficient to test the potential of demand predictions in TopTrac.

The final schematic of the technical setup is shown in Figure 6.1. Besides the aforementioned technical challenges there were some other major challenges, which cannot be described due to confidentiality reasons. As these issues were related to detailed inner workings of TopTrac. It is also not necessary to describe these issues as they only cost the researcher more time, but do not influence the outcome of the research.

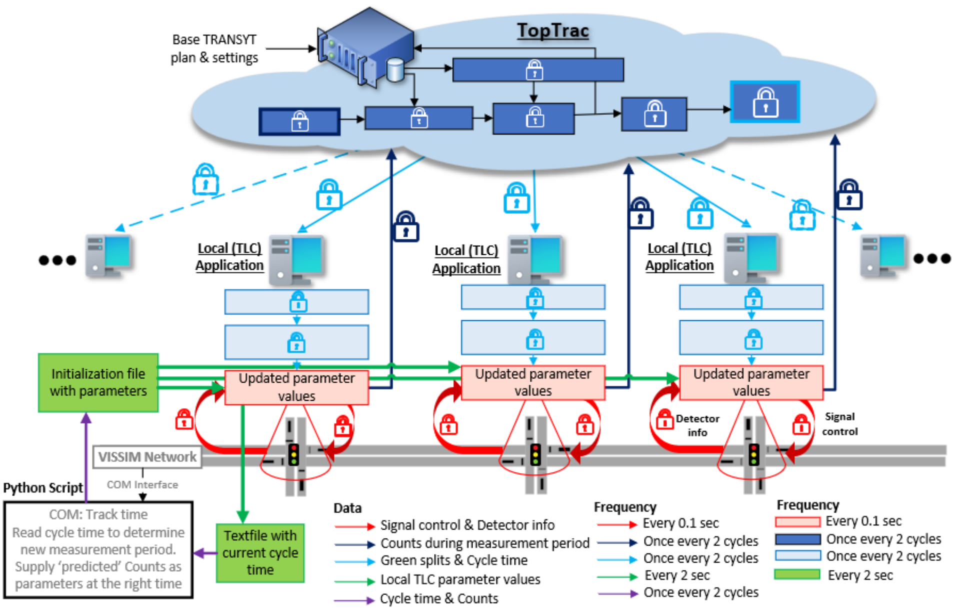

Figure 6.1: Technical setup used to supply predictions to TopTrac.

6.2. METHODOLOGY FOR TESTING THE POTENTIAL OF USING DEMAND PREDICTIONS IN TOPTRAC

To be able to assess whether developing and integrating a prediction model in TopTrac would be worthwhile, it is desired to know the maximal gain predictions can offer. This maximal gain would be expected when the predictions are (near-)'perfect'. At least these tested predictions should be better than any practical (erroneous) method would achieve. Note that 'perfect' is put in quotation marks as perfect could be interpreted in multiple ways: Perfect in the sense of perfectly resembling the traffic pattern or perfect in the the sense of perfectly supplying counts according to the traffic models workings. In an ideal world, perfect input into the model would lead to perfect output. But in a real world this is not the case. An overestimation of the traffic volume could for example lead to better performance than the true traffic volume. During this research, it is assumed that the traffic model is benefited by knowing the (near-)perfect traffic situation. Thus, when mentioning a 'perfect' prediction (test) this is intended to reflect a perfect resemblance of the actual traffic pattern. Making the prediction tests near-perfect means that the model will receive the actual traffic volume, for which it has to optimize control. The methodology assumes that the traffic volume measured at the stop line is a good measurement of the demand. A further assumption is that, by shifting the measured traffic volume at the stop line forward in time, a prediction can be made. In truth, measured flow is not equal to demand. Large discrepancies between measured flow and demand arise when there is over-saturation. Traffic volume measurements at the stop line are at best a lower bound for the actual traffic demand for the respective green cycle. During this research however, the traffic volume measurements at the stop line are translated to tests for demand predictions. TopTrac works with and is built to use traffic volumes, with separate strategies for detecting and dealing with over-saturation. Changing traffic volumes to actual demand may affect the controller in unexpected ways. Furthermore, the intention is to test a network where no over-saturation occurs. Chapter 9 reflects on the assumptions.

To be able to calculate the near-'perfect' prediction, the traffic situation of a reference simulation was used as input for a simulation that tests the potential of predictions. In an initial set of simulations the traffic inputs into TopTrac were simply shifted forwards in time by one optimization period. During this shift a certain vehicle count could result in a different traffic volume, as the vehicles may be counted in a measurement period of different length. This is accounted for by dividing the old count by the old period length and multiplying by the new period length. This conversion introduces non-integer vehicle counts, which are not accepted by the local TLC's. The decision was made to round up to the nearest integer after the conversion.

Figure 6.2 shows how this first near-'perfect' prediction turned out. A random link of the network is shown for a random seed, but other links and seeds show the same pattern. The predicted counts are quite nicely shifted forward in time by one optimization period. Volumes are plotted step wise, as this more accurately reflects that a certain volume is used to optimize the traffic signals for a certain time period. There are slight differences to be seen. These are caused by either the aforementioned rounding or by TopTrac processing the counts slightly different. However, processing the counts is a deterministic module and by effectively shifting the count the result is, as can be seen in Figure 6.2, that the processed counts are nicely shifted by one optimization period as well.

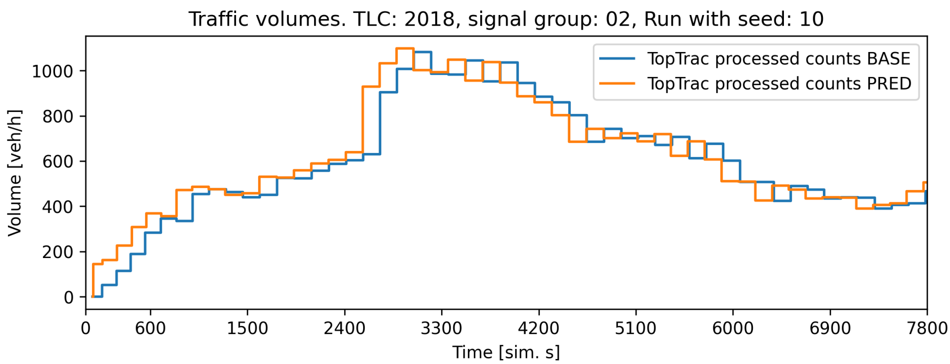

Figure 6.2: Reference (BASE) versus first predicted (PRED) counts, after passing TopTrac processing module. TLC, signal group and seed chosen randomly out of the 10 runs

Besides simply shifting the counts by one optimization period, preferably another method was employed as well. Figure 6.3 illustrates that after processing the counts,

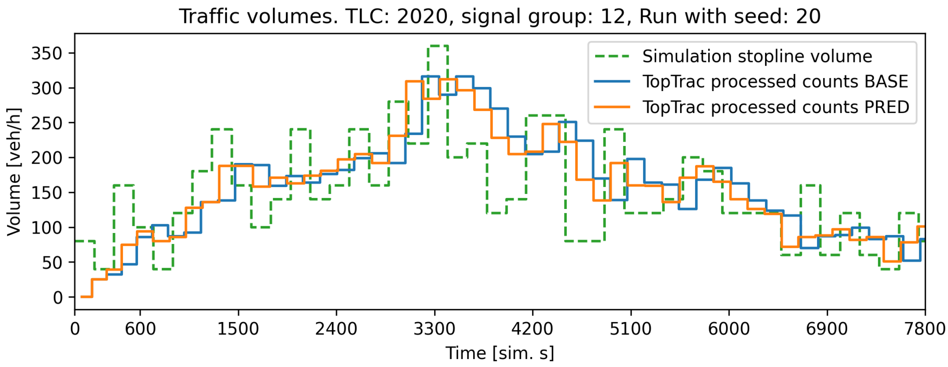

Figure 6.3: Reference (BASE) versus first predicted (PRED) processed counts versus volumes measured in the simulation. TLC, signal group and seed chosen randomly

they do not really (accurately) reflect the true traffic situation. This holds for both the base and predicted counts and for other links and seeds as well. It is hard to say which (prediction or base) is a better reflection of the true traffic volumes. The true traffic was measured in the simulation environment, every three minutes. During this second set of tests it was aimed to bypass the count processing module and be able to feed the counts directly to the traffic model. Since this would result in a quite different traffic pattern being supplied to model (instead of the same pattern but shifted in time), a slightly different methodology was employed. Instead of using TopTrac counts from a reference run, VISSIM data from a reference run was used. This data was gathered through measurements of vehicle counts every three minutes. The near-'perfect' prediction test finally was realized by tracking the simulation time and smartly timing the moments of supplying a certain flow measurement (again taking in regard the length of the measurement periods). Using the same location as was used in Figure 6.3, this method now supplied the prediction tests as shown in Figure 6.4, directly to the TopTrac traffic model.

In Figure 6.4, the prediction tests closely follow the volume that is actually present in the simulation network. Some differences can be seen which are explained as follows. Differences in the volume level are caused by rounding. The same methodology, regarding the conversion of the period and rounding up to the nearest integer was applied. Consequently, it is clear that the predicted volume is always higher or equal to the volume on which the prediction test is based. Another strategy may be to round to the nearest integer, which on average may be more accurate, but may result in underestimation of the volume sometimes. Either way, the introduced error is very marginal and a different technical setup would be required to eliminate it completely. Another difference can be seen regarding the timings. The simulation stopline volume is measured every 180 seconds, at fixed intervals. The predictions are supplied according to the (variable) intervals used by TopTrac. It is therefore impossible that the prediction tests will perfectly overlap with the true volume in time. Again, through smartly timing which traffic flow volumes are supplied to which optimization interval, the differences are kept to a

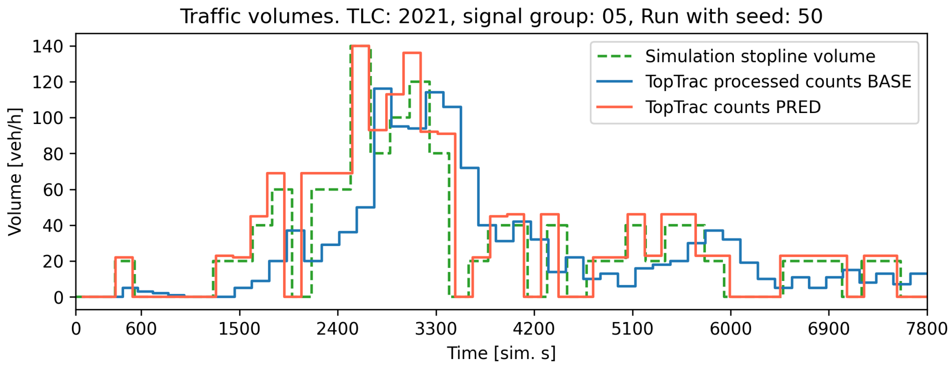

Figure 6.4: Reference (BASE) versus second predicted (PRED) volumes supplied to Toptrac, versus volumes measured in the simulation with predictions. TLC, signal group and seed chosen equal to Figure 6.3

minimum. Overall, the prediction tests very closely resemble the true volume, definitely much closer than any realistic/practical traffic volume prediction algorithm would be able to achieve.

This second methodology makes one major assumption regarding the supplied predictions tests. It assumes that the true traffic volume that passes the stop line in the reference run will equal the traffic that passes the stop line in the simulation run with predictions. Figure 6.4 shows that this assumption holds on a side direction, where the volume is unaffected by previous control decisions. The same is true for the other side directions as well. Figure 6.5 shows that this assumption does not hold on the main directions. Especially the volumes on the main directions further downstream in the network are affected by the control decisions made upstream. The intersection shown in Figure 6.5 is the sixth intersection along the arterial. At this intersection it makes sense that, when other control decisions (than in the reference simulation run) have been made upstream, errors arise in the prediction tests. Overall the prediction tests still show a better resemblance to the true traffic pattern than in the reference simulation, with a few exceptions at some time steps. Due to the errors, these prediction tests at the main directions on the downstream intersections are no longer 'perfect' and this is a point of discussion when it comes to analyzing the results of the simulations.

6.3. CONCLUSION

In this chapter, a method was developed to test the potential of demand predictions in the TopTrac coordinated traffic controller, to answer research question **2.1**: *How can tests of the potential of demand predictions of undersaturated demand be included in TopTrac in real-time?*

By intervening at the local controllers, a demand prediction test could be passed to TopTrac, without the need for modifications on the central server. To be able to provide the near-'perfect' prediction, two methodologies where adopted:

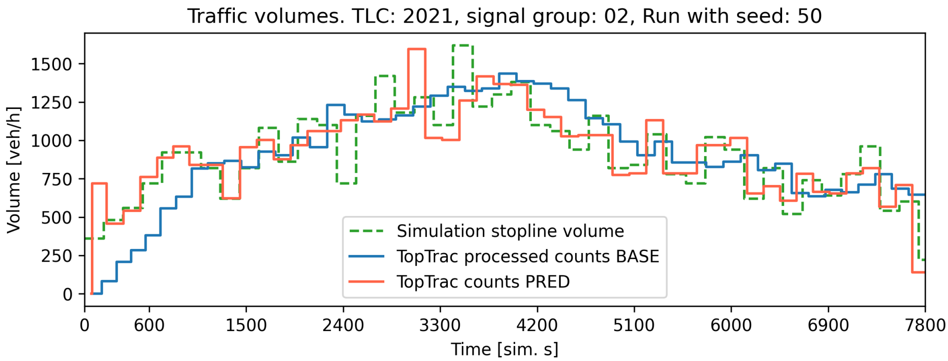

Figure 6.5: Reference (BASE) versus second predicted (PRED) volumes supplied to Toptrac, versus volumes measured in the simulation with predictions. For a main direction of the same TLC and seed as Figure 6.3

1. A prediction test based on the traffic volume that TopTrac used in a simulation with the same seed, but shifted forward in time by exactly one optimization period.

2. A prediction test based on the measured traffic volume at the stop line in a simulation with the same seed, where the TopTrac count processing module was disabled to be able to directly give the prediction test of the true traffic volume to the traffic model of TopTrac.

Numerous challenges had to be overcome while translating the traffic volume to a prediction test, amongst others, finding the correct timings and dealing with the variable measurement period length.

All prediction tests are based on traffic volumes measured at the stop line, making them at best a prediction of the lower bound of the actual demand, not a prediction of the true demand. When oversaturation occurs the stop line measurement can only measure capacity. During testing and simulating, it was observed that, in the studied network, no oversaturation occurs and all queues are resolved during the green phase. However, in another network this may not be the case, and the applied methodology is therefore not generally applicable to any network.

7

SIMULATION SETUP

This chapter goes into detail about the simulation setup used for the evaluation part of this thesis. The choice for simulation tool, some workings of this tool and the simulated network are discussed. This chapter is relevant for both the variable speed and the tests of the potential of demand predictions, as both will make use of the same tool and network.

7.1. MOTIVATION FOR DOING SIMULATIONS

Before diving into the details of the simulation tool and network, the motivation for performing a simulation should be clearly outlined. Especially in traffic engineering, simulations are the go-to method for testing certain solutions. Regarding the proposed solutions, we need some way to test whether, and to what extent, they live up to their expected usefulness. Simulations are a very accessible way to perform tests, compared to testing in practice. Testing directly in practice is expensive and quite risky. Furthermore, the effort of creating a test bed in practice is too large for the scope of this thesis. Even if this weren't the case, first showing the effectiveness and operation of a solution in a simulated environment is desired, as unproven techniques and methods bring a safety concern. As a final remark, in practice it is difficult or even impossible to test different scenario's, like demand patterns or different traffic control strategies, and to be able to gathered all the data necessary for analyzing the performance.

For this thesis, simulations are the ideal tool to evaluate the potential of the proposed solutions. In practice, it will be impossible to control the speed of traffic to reflect the proposed variable speed solution. Furthermore, only in the (controlled) simulation environment is it possible to provide the 'perfect' demand predictions introduced in chapter 6.

7.2. SIMULATION TOOL

Hegyi et al. (2001) discuss in their paper the different ideas that underline traffic simulation tools. Some models have a physiological origin, for example the Lighthill-Whitham-

Richards Model (James & Beresford, 1955) that makes a connection between waves in a river and waves in traffic or the cellular automata models (Nagel & Schreckenberg, 1992) that make a connection to how molecules in a liquid behave. Other models have a psychological origin, describing the traffic system based on the total behaviour and decisions of individuals. No single model approach is superior for every application, and the choice can heavily affect results. A choice for a specific simulation model should therefore always be motivated, which is what this section will do.

The aforementioned models, as well as many other historic models are on a macroscopic level. Hoogendoorn and Bovy (2001) give a detailed overview of the state-of-the-art in traffic modelling up till 2001. Increased computer power, desire for more detailed information and control have been major contributors to the rise of microscopic models. Microscopic simulation models are the current state of practice in the traffic simulation field. The most advanced and commonly used software being: VISSIM, AIMSUN, PARAMICS and CORSIM. These traffic simulation software packages are commercial, SUMO is a potential open-source alternative.

In this research the VISSIM software will be used for a variety of reasons, the main ones being:

- Availability: Vialis has a VISSIM license available, as well as networks configured to work with TopTrac. VISSIM is the main simulation software used at Vialis, meaning that support from colleagues is available for this software.

- Prior experience: The researcher has had some prior experience with using VISSIM, in contrast to the other software packages.

- Desired functionality: VISSIM offers all desired functionalities needed for the research.

7.3. SIMULATION RANDOMNESS

As explained in Hillebrink (2018), simulations use randomness to more accurately reproduce real-life situations. In VISSIM this randomness can for example be found in the demand pattern, desired speed, route choice, desired distance gap and a variety of decisions made by drivers while driving. The randomness is largely determined by the random seed of the simulation run. This allows to reproduce the same run, when the same seed is selected. Randomness is needed to represent reality more accurately, however because this randomness will affect the simulation results, multiple simulation runs are required to make sure that conclusions are not based on chance. Logically, these runs should have different random seeds. To make sure that certain results are not dictated by randomness, a minimum number of replications is required. In Fishman (1978) the criterion shown in Equation (7.1) was given, which is still applied in literature to determine a suitable amount of replications. It assumes that the outputs from simulation runs with different seeds are normally distributed.

$$R \geq R_i = \max\left[2, \left(\frac{S_{R_i}(Y_i)\, t_{\frac{\alpha}{2}}}{d_i}\right)^2\right] \tag{7.1}$$

Where:

R = The number of total replications [-]

R_i = The number of replications performed so far [-]

$S_{R_i}(y_i)$ = The standard deviation in the observed data (y_i) based on the number of replications performed so far. The unit depends on the unit of (y_i).

$t_{\frac{\alpha}{2}}$ = The critical value of the t-distribution at significance level α. 1.96 for a 95% confidence level ($\alpha = 0.05$) (J. Jones, 2022).

d_i = The (desired) level of precision, typically 5% of the mean of the observed data (Duives, 2021).

For this thesis, a different approach was adopted. Namely, the aforementioned method resulted in a very low number of repetitions, as the standard deviation of the data was quite small. Furthermore, the differences between the reference and test simulation were small for the simulations with prediction tests, meaning that on the basis of a very small amount of repetitions, a conclusion would be insubstantially motivated. The final approach involved performing 10 repetitions, on the basis of which it could be concluded, using the appropriate confidence intervals, whether any significant differences could be observed in the results.

7.4. SIMULATION NETWORK

The chosen network is located in Almere, in the Netherlands, and depicted in Figure 7.1. This network was chosen, because from Vialis it was known that there is typically no oversaturation in this network, which is important for the simulations of this thesis. This allows to answer the research questions on a practical cases, with traffic demand patterns from practice. Furthermore, TopTrac operates in practice on this network and the network was also suitable for testing the variable speed on a case where coordination with both the fixed speed and the variable speed was possible. In total, the coordinated arterial consists of 7 intersections, located on the Havendreef and Stedendreef. In the simulation, the neighbouring intersections at either end of the network (numbered 2015 and 2022) are included in order to have arrival patterns that more closely resemble reality.

Figure 7.2 gives a zoomed in view of one of the intersections in the network. This intersection layout is typical for this network, with 6 of the 7 coordinated intersections having this layout. The intersection has two main directions (numbered 02 and 08), both with separate left-turns (numbered 03 and 09). The cross-street directions consist of separate right turns (numbered 04 and 10) and directions for through and left-turns combined (numbered 05 and 11). The only intersection with a different layout is intersection 2020, which has a T-shaped layout.

7.5. SIMULATION EXECUTION

Every simulation is performed with a duration of 7800 seconds. The first 600 seconds are used to fill the network, so that in total, the results can be calculated on the basis of 2 hours of reliable simulation data. The gathered data consists of vehicle travel time measurements, delay measurement, located at every stop line in the network. Travel time measurements were taken along the entire arterial as well.

Demand in the simulations is the quarterly demand for morning rush hours in this network, averaged over workdays in march of 2022. This data was obtained via traffic management platform verkeer.nu. This demand gives a realistic representation of the demand that occurs in practice. Demand intervals are specified per quarter in VISSIM as well, to reproduce the morning rush hour more accurately in the simulation. Inputs are set to 'Exact', meaning that in the interval, in total the exact amount of vehicles enter via the input as specified. Randomness in demands occurs within the time interval, where VISSIM stochastically decides when and how many vehicles enter the network.

7.6. TRAFFIC CONTROLLER

The local traffic controllers used in the simulation are semi-fixed controllers. When TopTrac is enabled, the local TLC's receive the cycle time, minimal start of fixed green time (GF) and maximum end of extension green time (GX) from TopTrac in real-time. All the green windows will start some time before or exactly at GF and terminate some time before or exactly at GX. The true start and termination of the green window is determined by the local application, which uses detector info and various parameters. Such parameters, and any other settings that are not directly related to the technical setup for supplying predictions, are not changed with regards to the situation that is operational in practice. For the fixed-time simulations with the variable speed, the cycle time, GF and GX are constant across the entire simulation.

7.7. APPLYING VARIABLE SPEED THEORY TO PRACTICE

In this section the theory from chapter 4 is applied to the discussed network.

The data for the network is as follows:

7

- Segment distances: [366, 278, 277, 301, 283, 305] [m].
- Maximum speed: 50 [km/h].
- The minimum cycle times to fulfill the demand, based on the critical path method, are: [51, 49, 53, 54, 38, 52, 55] [s]. Such that the selected minimum network cycle time to fulfill the demand (C_q) is 55 seconds.
- The coordinated directions have a maximum internal offset difference of [6, 6, 6, 6, 13, 6, 6] [s] in the (efficient) signal timing plans. These differences are made possible by a different order of the main direction left-turns.

According to the LP3 program from section 4.2.3, no feasible solution is possible for coordination of the start of green with a fixed speed. The main reason is that the maximum cycle time for coordination with a fixed speed cannot fulfill the minimum network cycle time for demand of 55 seconds. According to Equation 4.57 the cycle time must not exceed 50.16 seconds for coordination with the fixed speed. LP1, which does not account for the cycle time, (or LP3 with the cycle time constraint disabled) does yield a feasible solution for the fixed speed. This shows that in this case, the cycle time is normative, not the geometric requirement of the nearly simultaneous start of outbound and inbound green windows.

To allow for a large enough cycle time to fulfill the demand, coordination between some parts (other than the start) of the green window must be considered or coordination between locally sub-optimal timing plans must be considered. Potentially, this leads to longer green windows, leading to an even longer cycle time. In general, coordination of the starts of green between the locally optimal timing plans is not possible in this network.

Using the programs shows that the theory can judge the feasibility of coordination of the starts of green between (the most) efficient timing plans, but cannot judge the general feasibility of coordination with a fixed speed.

Approaching the problem with a variable speed, a feasible solution for the coordination of the starts of green of the locally optimal timing plans can be obtained with a cycle time of 55 seconds and minimum speed of 36 [km/h], without utilizing any of the freedom in internal offset difference. One of these solutions is given by the following set of speeds per segment: [48, 36, 36, 39, 37, 40] [km/h]. Note that in this solution, inbound and outbound speed per segment are equal. This solution with the variable speed is not optimal, for example a solution with a higher average speed may be possible.

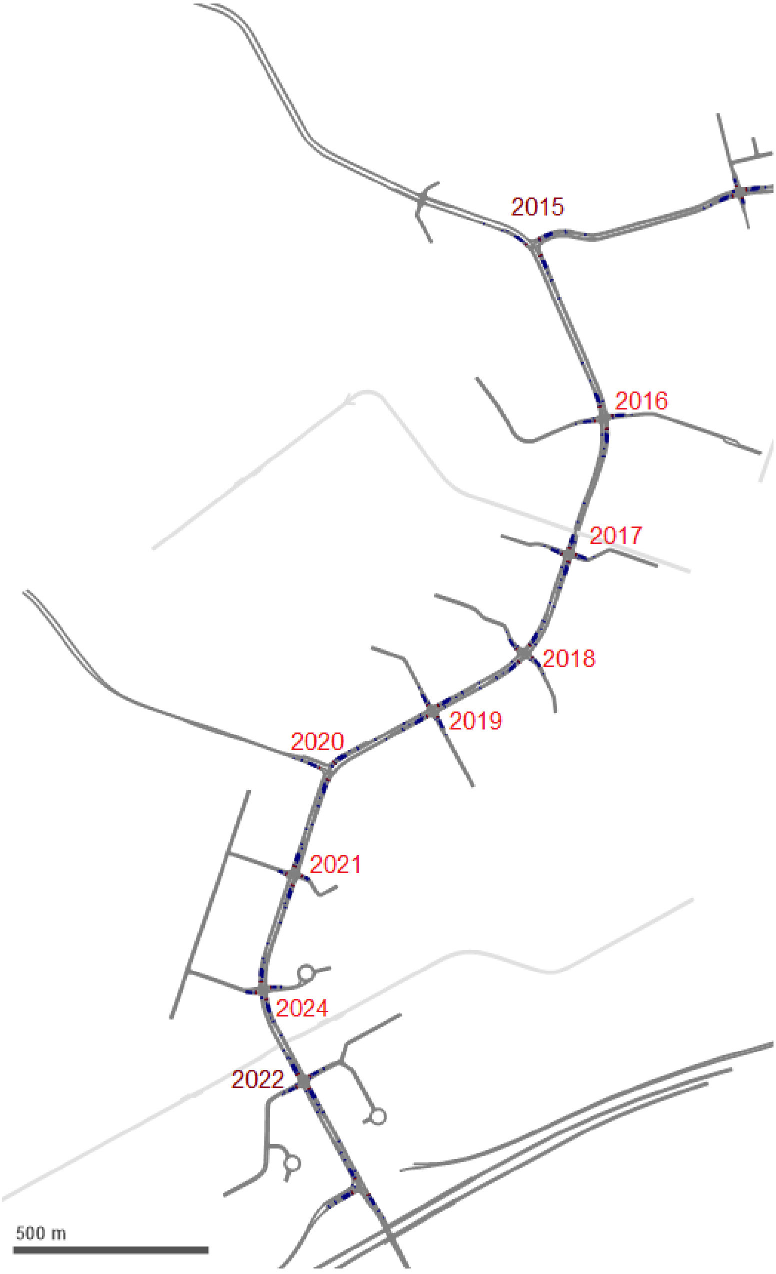

Figure 7.1: Simulated VISSIM network: Havendreef-Stedendreef Almere, Netherlands.

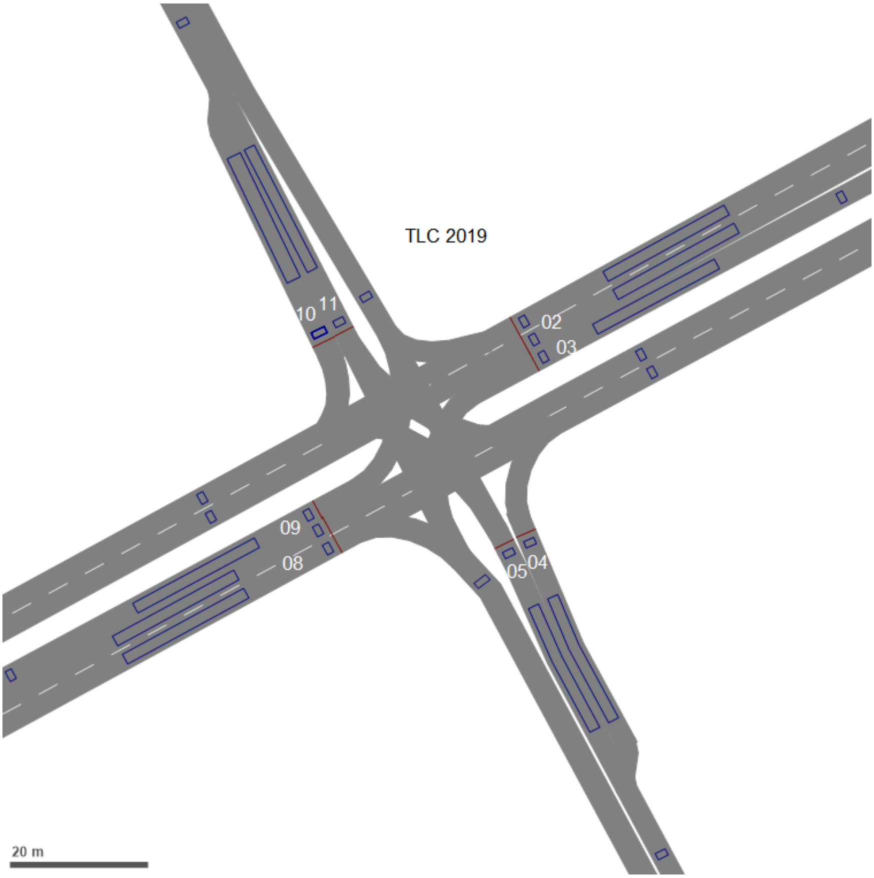

Figure 7.2: Close up of one of the intersections of the network (TLC 2019). This is the layout that is exactly the same on 6 of the 7 intersections in the network.

8

SIMULATION RESULTS

This chapter presents the results of the simulations. The first section, section 8.1 looks at the results of the simulations with the variable speed and answers research question **1.4**: *What are the benefits, in terms of reduced stops and delay, of applying variable speeds to coordinated pre-timed semi-fixed signal timing plans?*

The second section, section 8.2 looks at the results of the simulation with the tests of the potential of demand predictions and answers research question **2.2**: *Supposing that the (undersaturated) demand is known and TopTrac gets a 'perfect' prediction, what are the gains in terms of reduced stops and delay for a typical morning rush hour?*
Finally, in the third section, a deeper analysis is performed into the TopTrac traffic model outputs resulting from the demand prediction tests.

8.1. RESULTS OF VARIABLE SPEED SIMULATIONS

Tables 8.1, 8.2, 8.3, 8.4, 8.5 and 8.6 show the 95%-confidence intervals of numerous performance indicators like stops, total time spent and delay for different aggregation levels. The intervals are calculated on the basis of simulation data from 10 runs, using Equation 8.1. It should be noted, regarding the total time spent, that this indicator includes partially completed routes (unfinished at the end of the simulation) and that no vehicles were obstructed from entering the network.

$$\left[\mu - \frac{Z_{\alpha/2}\cdot\sigma}{\sqrt{x}},\ \mu + \frac{Z_{\alpha/2}\cdot\sigma}{\sqrt{x}}\right] \tag{8.1}$$

Where:
μ = The mean of the data.
σ = The standard deviation of the data.
x = The number of simulation runs, here equal to 10.
$Z_{\alpha/2}$ = The Z-score corresponding to the two-sided probability of a standard normal distribution. For a 95%-confidence interval ($\alpha = 0.05$), the value is 1.96.

Table 8.1: Overview of stop results of variable speed simulations for the whole network, based on 10 runs and 2 hours of data.

Simulation	**Total nr. of stops [#]**	**Mean nr. of stops [#/veh]**	**Nr. of veh. participating in measurements [#]**
Fixed speed	11500	0.46	24827
Conf. interval	[11386, 11614]	[0.46, 0.47]	[24642, 25012]
Var. speed	8569 (-25.48%)	0.35 (-25.47%)	24823
Conf. interval	[8486, 8653]	[0.34, 0.35]	[24634, 25013]

Table 8.2: Overview of total time spent results of variable speed simulations for the whole network, based on 10 runs and 2 hours of data.

Simulation	**Total time spent in network [h]**	**Mean time spent in network [s/veh]**	**Nr. of unique veh. in network [veh]**
Fixed speed	548.56	212.19	9307
Conf. interval	[545.7, 551.5]	[211.1, 213.3]	[9306, 9308]
Var. speed	554.98 (+1.17%)	214.68 (+1.17%)	9307
Conf. interval	[552.5, 557.5]	[213.7, 215.6]	[9306, 9308]

Table 8.1 shows that the mean number of total stops in the network was lowered from 11500 to 8569 in the variable speed simulations, an improvement of 2930 (25.5%). Table 8.2 shows that the total time spent in the network increased by 6.4 hours (1.2%). This number equates to spending about 2.5 seconds extra in the network, on average, per vehicle. The total time spent in the network was included as a time-based indicator, instead of vehicle delay. The reason for this choice is that the variable speed simulations include different desired speed decisions on the main directions in VISSIM. Vehicle delay computation by VISSIM is affected by desired speed, whereas total time spent in the network is not. Since the desired speed decisions on the side directions stayed equal between both simulations, VISSIM vehicle delay results can only be used as a reliable delay indicator for the side directions.

Table 8.3: Overview of stop results with a variable speed simulation for the main directions (02 and 08), based on 10 runs and 2 hours of data.

Simulation	**Total nr. of stops [#]**	**Mean nr. of stops [#/veh]**	**Nr. of veh. participating in measurements [#]**
Fixed speed	7834	0.40	19606
Conf. Interval	[7728, 7939]	[0.40, 0.40]	[19415, 19796]
Var. speed	4880 (-37.70%)	0.25 (-37.68%)	19598
Conf. Interval	[4796, 4965]	[0.25, 0.25]	[19403, 19793]

Table 8.3 shows that, on the coordinated directions, the mean number of total stops was reduced by 2953 (38%) in the variable speed simulation. Comparing this to the overall network reduction of (on average) 2930, it is clear that the coordinated directions are

Table 8.4: Overview of stop results with a variable speed simulation for the side directions in the network (04, 05, 10, 11 and 12), based on 10 runs and 2 hours of data.

Simulation	Total nr. of stops [#]	Mean nr. of stops [#/veh]	Nr. of veh. participating in measurements [#]
Fixed speed	2562	0.74	3459
Conf. Interval	[2545, 2578]	[0.74, 0.75]	[3456, 3462]
Var. speed	2561 (-0.02%)	0.74 (-0.12%)	3462
Conf. Interval	[2537, 2585]	[0.73, 0.75]	[3459, 3465]

Table 8.5: Overview of delay results with a variable speed simulation for the side directions in the network (04, 05, 10, 11 and 12), based on 10 runs and 2 hours of data.

Simulation	Total veh. lost hours [veh.h]	Avg. delay per veh. [s]	Nr. of veh. participating in measurements [#]
Fixed speed	26.51	27.59	3459
Conf. Interval	[26.22, 26.8]	[27.29, 27.89]	[3456, 3462]
Var. speed	21.33 (-19.54%)	22.18 (-19.61%)	3462
Conf. Interval	[21.06, 21.61]	[21.89, 22.47]	[3459, 3465]

solely responsible for the large reduction in the number of stops. The non-coordinated directions actually saw an increase of 23 stops, though this increase is not statistically significant. Looking specifically at the side-directions, Table 8.4 shows that the number of stops on these directions is very similar between both simulations. Note that the non-coordinated directions are not the same as the side directions. Namely, the left-turns on the main arterial are not considered coordinated directions nor side directions. Table 8.5 shows that, on the side directions, the average delay per vehicle was reduced by 5.41 seconds (19.61%). This decrease can be attributed to the lower cycle time in the variable speed simulations, which was 66 seconds compared to 87 seconds in the fixed speed simulations. Overall, with an insignificant change regarding the number of stops and a significant improvement regarding the delay on the side directions, it is observed that the side directions are positively impacted in the variable speed simulation.

Table 8.6: Overview of arterial travel time results with a variable speed simulation, based on 10 runs and 2 hours of data.

Simulation	Mean outbound travel time [s]	Measured nr. of veh. [veh]	Mean inbound travel time [s]	Measured nr. of veh. [veh]
Fixed speed	203	622	194	498
Conf. interval	[202, 204]	[613, 631]	[194, 195]	[486, 510]
Variable speed	214 (+5.61%)	619	182 (-6.12%)	498
Conf. interval	[213, 215]	[607, 631]	[182, 183]	[482, 514]

Lowering of the speeds affects the travel time across the arterial. Table 8.6 shows that, in the variable speed simulation, the mean outbound arterial travel time increased with about 11.4 seconds (5.6%) compared to the fixed speed simulation. As a reminder, in the

8

variable speed simulation, the outbound (variable) speeds per segment were, by design of MAXBAND: [30, 30, 30, 50, 50, 30] [km/h]. In the simulations with a fixed speed, 50 km/h was the design speed on all segments. Using the lengths of the segments and the speeds per segment, the free flow travel time (disregarding all acceleration, deceleration and presence of signals) across the arterial would be 189 seconds for the variable speed, compared to 130 seconds for the fixed speed. Difference with the observed values can be partially explained due to additional braking in the simulation. It was observed that lane changes, running into queues, different acceleration and deceleration rates and different desired time and space headway's all caused vehicles to brake.

Table 8.6 also shows that, on the inbound direction, the mean arterial travel time decreased with about 11.9 seconds (6.1%) in the simulation with a variable speed, compared to the fixed speed. For the inbound direction the design speeds in the variable speed simulations were: [41.8, 30, 30.6, 40.1, 50, 45.3] [km/h], compared to 50 km/h on all segments in the fixed speed simulations. The same free flow travel time computation leads to values of 170 seconds of the variable speed and 130 seconds for the fixed speed. Differences with the observed values are also partially caused by additional braking, similar to the outbound direction.

Considering the free flow travel times, it is remarkable that the outbound travel time only slightly increased and the inbound travel time even decreased. This is explained based on other observations during the simulation. The timing plans with the variable speed had more bandwidth (21.1 seconds), compared to the fixed speed (7.8 seconds), such that more vehicles were able to achieve a green wave in the variable speed simulation. In both simulations, sometimes, some vehicles didn't experience a full green wave across the entire arterial. These vehicles had to wait another cycle to continue, thereby heftily increasing the average travel time. This happened more often in the simulations with a fixed speed, because of the smaller bandwidth in the timing plans. Overall, an increase in average travel time and total time spent in the network are to be expected by the (lower) variable speed and highlight a drawback of coordinating with the proposed variable speed.

To conclude if the found values are statistically significant, a two-sample t-test of the means of the data is performed using equation 8.2. The absolute values of the student's t-test values should be compared to the two-sided, 95% confidence statistic for 18[1] degrees of freedom: 2.101 (J. Jones, 2022). The values of the t-test test statistics can be found in table 8.7.

$$t = \frac{\mu_{\text{base}} - \mu_{\text{pred}}}{\sqrt{\frac{s^2_{\text{base}} + s^2_{\text{pred}}}{x}}} \tag{8.2}$$

Where:

t = The student's t-test value.

$\mu_{\text{base}}, \mu_{\text{pred}}$ = The mean of the data from the reference simulation and simulation with predictions, respectively.

$s^2_{\text{base}}, s^2_{\text{pred}}$ = The unbiased variance of the data from the reference simulation and simulation with predictions, respectively.

[1] 2 samples with 10 data points, 2 times 9 degrees of freedom

x = The number of simulations, here equal to 10.

Table 8.7: Overview of student's t-test test statistics of the 'total' results from Tables 8.1, 8.2, 8.3, 8.4 and 8.5. If the absolute value of the test statistic is greater than 2.101, the results are statistically significant.

T-test test statistics for simulation results	**Total nr. of stops**	**Total time spent in network**	**Total veh. lost hours**
Var. speed network	-38.590	3.319	
Var. speed main directions	-40.653		
Var. speed side directions	-0.038		-24.157

The value of the test statistic in Table 8.7 indeed confirms that the there was no significant change in the mean number of stops on the side directions in the variable speed simulation, compared to the fixed speed simulation. The values for the other results, including the total time spent in the network, are statistically significant.

To answer research question **1.4**: *What are the benefits, in terms of reduced stops and delay, of applying variable speeds to coordinated pre-timed semi-fixed signal timing plans?* The variable speed reduced total stops in the network by 25.5%, with all of the reduction happening on the main (coordinated) directions. However, with the variable speed, delay increased, reflected by an increase of the total time spent in the network by 1.2%. This delay increase is the result of slower travel speeds on the main directions. These slower speeds are a direct consequence of working with (slower) design variable speeds. These slower speeds lead to an increase in travel time across the arterial. The variable speed allowed to coordinate with a lower cycle time. This resulted in a decrease in delay on the side directions of 19.61%. The number of stops on the side directions was not significantly affected.

8.2. RESULTS OF DEMAND PREDICTION TESTS SIMULATIONS

Tables 8.8, 8.9 and 8.10 show the results of the demand predictions, also computed using Equation 8.1. As a reminder, the first prediction test (prediction 1) is a prediction test based on the traffic volume TopTrac used in the reference simulations, but shifted forward in time by exactly one optimization period in the simulations with prediction tests. The second prediction test (prediction 2) is a prediction test based on the measured traffic volume at the stop line by VISSIM in the reference simulations, supplied directly to the TopTrac traffic model, such that time periods of the measurement period and TopTrac optimization period overlap the most.

From the results in Tables 8.8, 8.9 and 8.10 it is clear that the prediction tests do not result in large benefits on any indicator. For all indicators, the confidence intervals overlap, showing that the difference in results between the simulations is only very marginal, if any. Most indicators seem to show a marginal improvement, but it is unclear whether this slight improvement is due to chance, caused by the randomness of the simulations. Indeed, based on a two-sample t-test of the means of the data, it can be concluded that differences in the results are not statistically significant. This t-test was performed for all indicators, using the formula presented in Equation 8.2. The values of the t-test test

Table 8.8: Overview of simulation results of traffic demand prediction tests for the whole network, based on 10 runs and 2 hours of data.

Simulation	Total nr. of stops [#]	Total veh. lost hours [veh.h]	Nr. of veh. participating in measurements [#]
Reference	9215	72.78	24829
Conf. Interval	[9144, 9286]	[72.28, 73.27]	[24640, 25018]
Prediction 1	9183 (-0.35%)	72.59 (-0.26%)	24826
Conf. Interval	[9113, 9252]	[72.06, 73.12]	[24641, 25011]
Prediction 2	9109 (-1.15%)	72.42 (-0.49%)	24825
Conf. Interval	[9023, 9195]	[71.74, 73.10]	[24642, 25008]

Table 8.9: Overview of simulation results of traffic demand prediction tests for the main directions (02 and 08), based on 10 runs and 2 hours of data.

Simulation	Total nr. of stops [#]	Total veh. lost hours [veh.h]	Nr. of veh. measured [#]
Reference	5877	41.49	19605
Conf. Interval	[5809, 5945]	[41.13, 41.85]	[19410, 19799]
Prediction 1	5851 (-0.43%)	41.55 (+0.14%)	19602
Conf. Interval	[5762, 5941]	[41.06, 42.04]	[19411, 19793]
Prediction 2	5832 (-0.76%)	41.78 (+0.68%)	19599
Conf. Interval	[5745, 5919]	[41.20, 42.36]	[19410, 19788]

statistics are shown in table 8.11.

The network total and average values, presented in this section, could be the result of, for example, an increase in performance early on in the simulation and a decrease later on in the simulation. Or perhaps there could be differences per simulation run or per intersection, or otherwise extreme outliers that are not noticeable in the computed indicator values. All of these types of results were analyzed, and in all cases, led to the same insignificant differences. Numerous tables and plots, showing performance over time, per direction and per intersections for various indicators are included in Appendix C. Because these results didn't prove insightful, they are further excluded from this chapter. Instead, it is much more interesting to acquire a better understanding of how these results came to be, which will be done in section 8.3.

To answer research question **2.2**: *Supposing that the (undersaturated) demand is known and TopTrac gets a 'perfect' prediction, what are the gains in terms of reduced stops and delay for a typical morning rush hour?* The tests have shown that there are no significant gains in terms of reduced stops and delay for a typical morning rush hour for TopTrac, for neither of the two 'perfect' predictions.

Table 8.10: Overview of simulation results of traffic demand prediction tests for the side directions in the network (04, 05, 10, 11 and 12), based on 10 runs and 2 hours of data.

Simulation	**Total nr. of stops [#]**	**Total veh. lost hours [veh.h]**	**Nr. of veh. measured [#]**
Reference	2497	23.61	3461
Conf. Interval	[2477, 2517]	[23.36, 23.86]	[3458, 3465]
Prediction 1	2493 (-0.16%)	23.50 (-0.47%)	3462
Conf. Interval	[2475, 2511]	[23.2, 23.79]	[3459, 3464]
Prediction 2	2473 (-0.93%)	23.28 (-1.39%)	3462
Conf. Interval	[2465, 2482]	[23.13, 23.43]	[3459, 3466]

Table 8.11: Overview of student's t-test test statistics of the results shown in Tables 8.8, 8.9 and 8.10. If the absolute value of the test statistic is greater than 2.101, the results are statistically significant.

T-test test statistics for simulation results	**Total nr. of stops**	**Total veh. lost hours**
Prediction 1 network	-0.603	-0.479
Prediction 2 network	-1.761	-0.787
Prediction 1 main directions	-0.417	0.180
Prediction 2 main directions	-0.755	0.774
Prediction 1 side directions	-0.278	-0.535
Prediction 2 side directions	-1.990	-2.013

8.3. ANALYSIS OF MODEL OUTPUT FOLLOWING PREDICTION TESTS

Simply computing results is not where the research ends. The most insight can be obtained from understanding why the results are as observed. Especially because the results of simulations with predictions are not as expected. Did the model produce different output based on the traffic volumes? And if so where and what are these differences and are they logical based on the different input? Did everything work as intended? These are question which will be answered in this section.

8

Firstly, one of the most insightful plots is that of the cycle time evolution, shown for the first prediction in Figure 8.1 and for the second prediction in Figure 8.2. Based on the input of the first prediction test, discussed in section 6.2 and shown in the same section in Figure 6.2, the output is expected. When the traffic volumes are precisely shifted forwards in time by one optimization period, the same holds for the model output. This is expected because the underlying model is of a deterministic nature. What is telling from Figure 8.1 is that already in the reference simulation, the model output was very consistent. This, despite the model input containing quite some variance in the traffic volume. Logically, when both outputs are nearly identical, the results will come out to be nearly identical as well.

Regarding the second prediction test, Figure 8.2 shows that in the simulation with this test, there is a lot more variation in the model output compared to the reference simulation. This is also expected, since the model input also contained more variation, as it bypassed the input processing module of TopTrac. However, the increased variation mainly resulted in more shifts (up and down) of the cycle time and not necessarily a large difference overall (the averages only differ by about half a second). So for this second prediction test, it is also not surprising that the results are not significantly different.

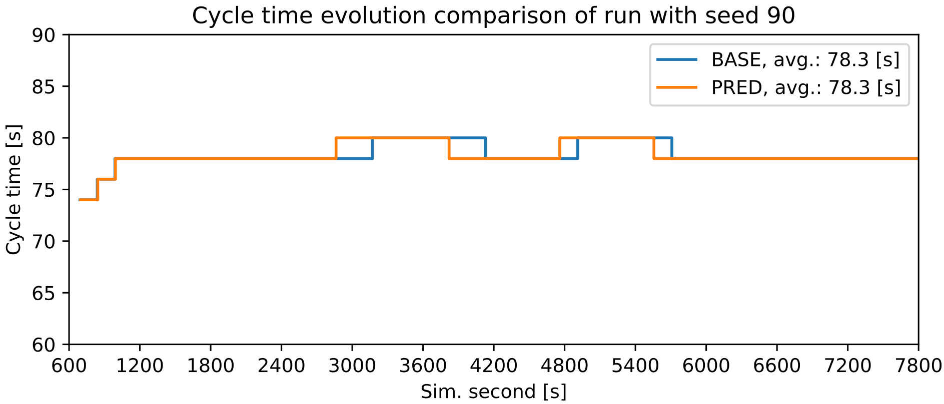

Figure 8.1: Reference (BASE) versus first predicted (PRED) Cycle Times determined by the traffic model for the simulation with seed 90.

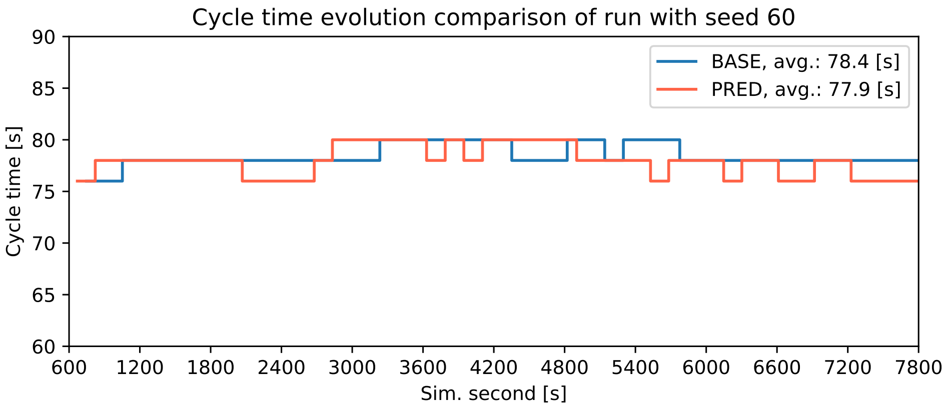

Figure 8.2: Reference (BASE) versus second predicted (PRED) Cycle Times determined by the traffic model for the simulation with seed 60.

Secondly, as the cycle time is just one of the model outputs, it is interesting to look at the other outputs as well, to see if these tell the same story. In Figure 8.3 the offsets are shown for the reference simulation and the simulation with the first prediction test. Similarly, Figure 8.4 shows the offsets for the reference simulation and the simulation with the second prediction test. Based on these outputs, a similar pattern is visible as was observed with the cycle times. For the first prediction test, the values are simply shifted forward in time. Though at a few points, some slight discrepancies can be observed. Overall, the outputs are very similar. For the second prediction test, it can again be observed that there is more variation in the output. However, on average, differences are still small. Regarding the offsets, it is not expected to observe much variation, as the speeds do not vary, only a few particular offsets allow for coordination.

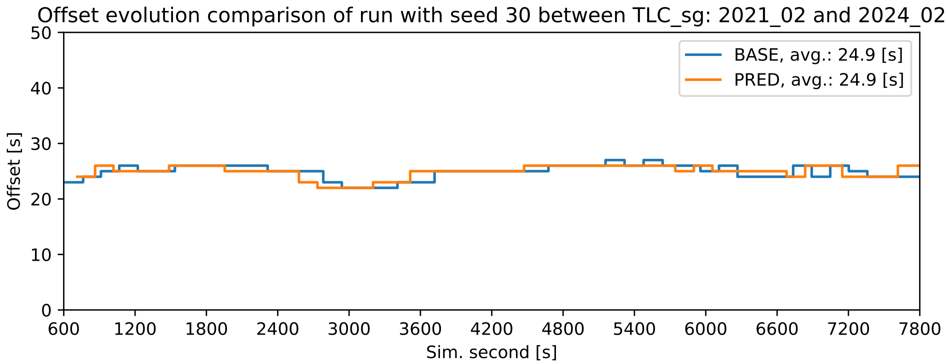

Figure 8.3: Reference (BASE) versus first predicted (PRED) offsets between the main outbound direction of TLC 2021 and TLC 2024 determined by the traffic model for the simulation with seed 30.

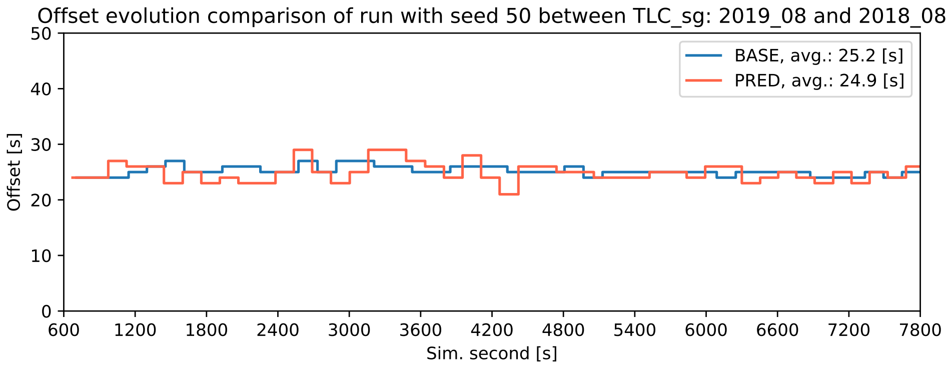

Figure 8.4: Reference (BASE) versus second predicted (PRED) offsets between the main inbound direction of TLC 2019 and TLC 2018 determined by the traffic model for the simulation with seed 60.

Besides cycle times and offsets, green time (duration) is also a model output. Regarding the green time duration, Figures 8.5 and 8.6 show the green time duration on side directions for the first and second prediction tests, respectively. These figures show that no different decisions are made on the side directions regarding the green time duration, across the whole simulation. The green time is equal to the minimum green time specified in the base Transyt plan (TopTrac is only ever allowed to increase this value). Other side directions and simulations show the same lack of difference. Similarly, for representative main directions, the green time is shown in Figures 8.7 and 8.8. These

8

figures show that the green time duration evolution on the main directions follows the same pattern as the cycle time evolution. Differences are minor and outputs do not follow the pattern of the input.

It should be noted that the figures shown in this section show the outputs produced by the model and these are outputs which are sent to the local TLC's. The local TLC's operate under a semi-fixed timing plan. The cycle time is a hard constraint, meaning that it is an output which is copied one to one by the TLC's. However, regarding the green times (and thus the offsets), these are implemented as semi-fixed minimal start of fixed green (GF) and maximum end of extension green (GX). This means that the final green time, realized in the simulation, may differ from the green time outputs of the model. To analyze these, the output (logging-)files of each local TLC had to be processed. Analyzing these values, it was observed that there are indeed differences. The local TLC's often realize the green some 5 to 10 seconds sooner, when this decision is logical, given the detection pattern. Similarly, the green may be terminated sooner than the final green time received from the model, in the absence of detections. The differences are slightly larger than what can be observed in the direct model output. However, to accurately examine these values, the detection pattern is also needed and when considering the detection pattern, the observed differences are not large and the conclusions remain the same.

Overall, this analysis of the model output has given more insight into why the results are what they are. Underlying the results of both simulations are very similar model outputs. Unsurprisingly, this leads to insignificant differences in the results. Apparently, the traffic patterns do not differ enough for the model to conclude that (very) different settings would lead to more optimized control. This, despite the input resembling a (morning) rush hour traffic pattern which includes (typical) variations in traffic volume.

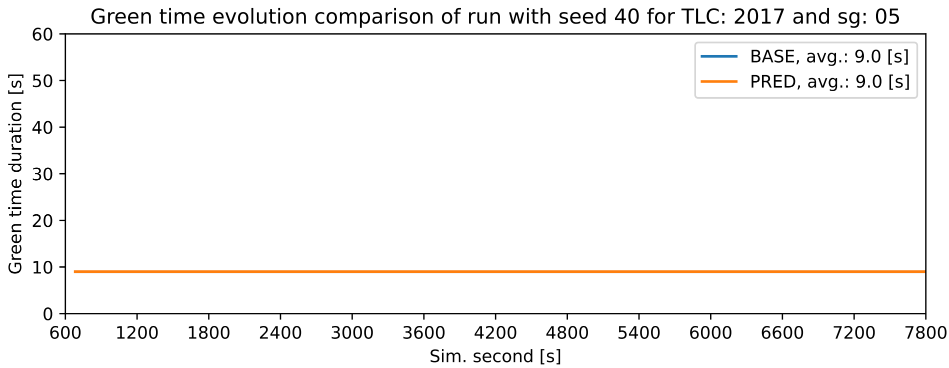

Figure 8.5: Reference (BASE) versus first predicted (PRED) Green Time Duration of signal group 05 of TLC 2017 determined by the traffic model for the simulation with seed 40.

The findings are summarized in Figure 8.9, where the green time duration is plotted against the traffic volume. This figure clearly shows that the changing traffic volume did not lead to a change in control decisions (in this figure: the green time duration).

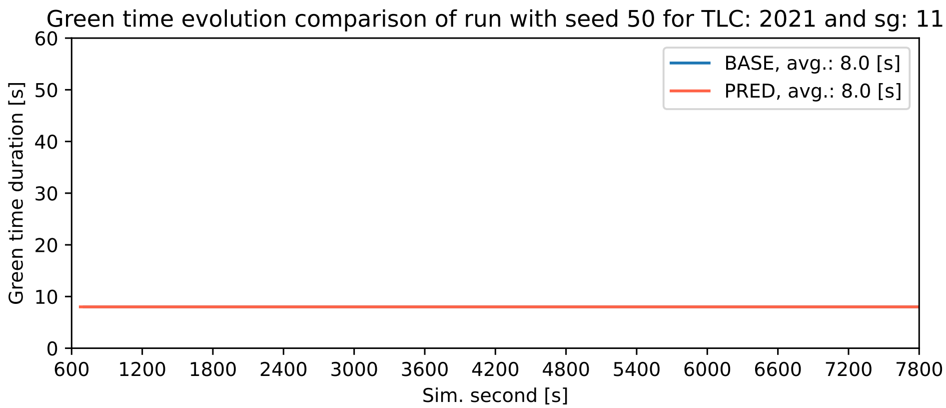

Figure 8.6: Reference (BASE) versus second predicted (PRED) Green Time Duration of signal group 11 of TLC 2021 determined by the traffic model for the simulation with seed 50.

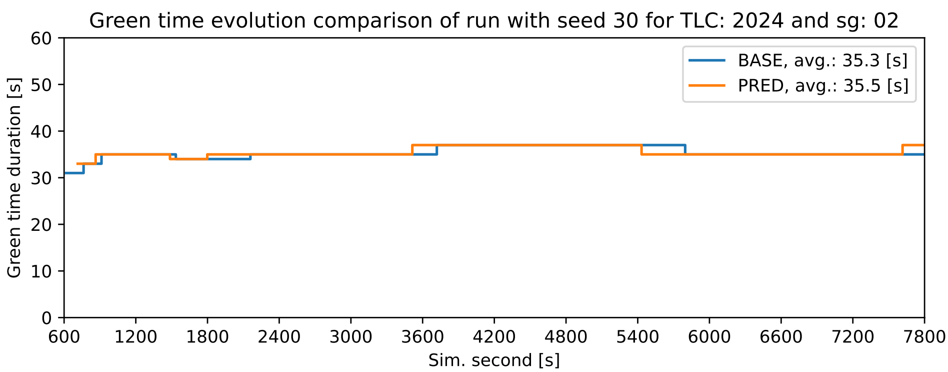

Figure 8.7: Reference (BASE) versus first predicted (PRED) Green Time Duration of signal group 02 of TLC 2024 determined by the traffic model for the simulation with seed 30.

The same pattern is observed at other signal groups, at other intersections and in other simulations.

8

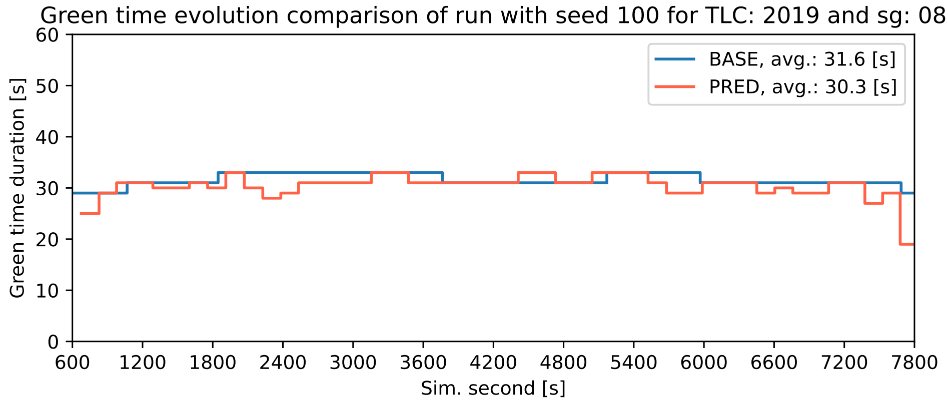

Figure 8.8: Reference (BASE) versus second predicted (PRED) Green Time Duration of signal group 08 of TLC 2019 determined by the traffic model for the simulation with seed 100.

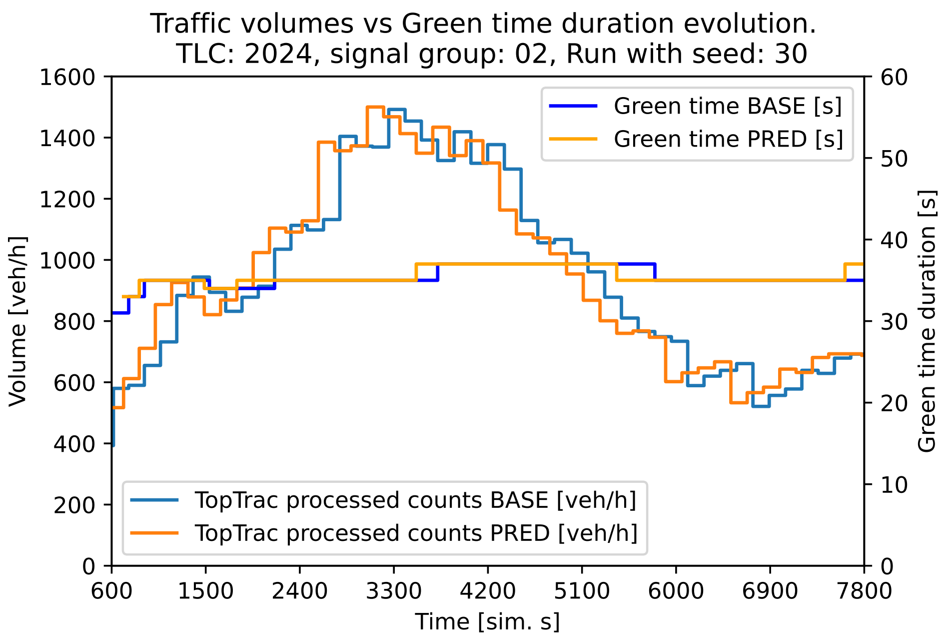

Figure 8.9: BASE volumes versus PRED volumes of the first prediction test versus BASE green time duration versus PRED green time duration of signal group 02 of TLC 2024 determined by the traffic model for the simulation with seed 30.

9
DISCUSSION

In this chapter the assumptions underlying the results and methodology, an their consequences for the conclusion, are discussed. This chapter is structured as follows: Firstly, the assumptions underlying the variable speed research are discussed. Thereafter, the assumptions regarding the demand prediction tests are discussed. Finally, some general points are discussed that apply to both the variable speed and the demand prediction research directions. The goal of this chapter is not to state all assumptions from all sections of the research, rather it only discusses the assumptions that are relevant for the conclusions presented in chapter 10 or those that are not discussed in previous chapters.

9.1. VARIABLE SPEED RESEARCH

The assumption which has the greatest effect on the conclusions of the variable speed research in chapter 4 is the assumption that coordination takes place between the starts of green of coordinated green windows. It was assumed that coordination is feasible when the starts of the coordinated green windows are aligned according to the offset. In truth, coordination is also possible between parts of the green window that do not include the starts of green, an example where this can be seen on some timing plan is shown in Figure 1.7. Moreover, queues may distort the coordination, such that aligning the starts of green cannot provide coordination to the entire platoon. Some queue clearance time would be needed to clear the queue, requiring that the coordination is aligned with some time (the queue clearance time) after the start of green.

Regarding coordination in one direction, this assumption limits the conclusion in the sense that the derived equations for the fraction of cycles that can be coordinated under different cycle times in section 4.1.1, only show the fraction of cycles where the starts of the green windows can be coordinated. Assuming there are no queues, it is argued that when the starts of green are aligned with the offset, one of the two green windows is fully coordinated. When both green windows have the same length, both are fully coordinated. The equations assume that coordination is not feasible, when the starts of green cannot be aligned with the offset. Conclusions, based on the equations, can therefore only conclude about the fraction of cycles that can be (fully) coordinated, such that both of the starts of coordinated green windows are coordinated. This is a lower bound for the amount of coordination that can happen when coordination with some other part of the green windows (for example coordinating for half of the platoon) is allowed as well.

In two directions the same assumption was made. This analysis focused on timing plans for coordination, where a network cycle time is applied, that is the lowest possible cycle time that allows to serve the demand. In the lowest cycle time to serve the demand, it may be the case that the green of the coordinated directions starts (nearly) simultaneously, for example when all directions conflict with the main directions or when the demand for the main directions far exceeds that of the other directions. Ideally, the full green windows of the coordinated directions are coordinated. This means that, ideally, the start of green is included in the coordination of at least one of the two green windows (when they are of different length) or in both green windows (when they are of equal length). The conclusions regarding the feasibility of coordination in two directions are therefore only applicable to the feasibility of the coordination of the starts of green, not coordination in general.

Another important consequence of this assumption is that coordination based on the starts of green does not consider the presence of queues. None of the equations in chapter 4 regard the presence of queues. For (semi-)fixed time signal timing plan design, the average queue clearance time, needed to clear the average queue, could be considered. Since the timing plans are (semi-)fixed, the queue clearance time is a constant across all cycles such that the conclusions of section 4.1.1 (discussed in section 4.1.2) are not affected. Regarding the conclusions of section 4.2, in section 4.2.2 it was shown that the obtained solutions can be translated in time by any constant. This constant can be set to the average queue clearance time, to account for queues on all intersections on the arterial simultaneously. This only yields good results if the average queue clearance times on all intersections, in both directions, are equal. The conclusions of section 4.2, discussed in section 4.3 regarding the feasibility of coordination between the (nearly) simultaneous starts of coordinated green windows in two directions are only applicable when disregarding the effect of unequal queues.

Throughout the research, speeds were allowed to vary between segments without constraints, as long as they stayed within the minimum and maximum speed bounds. Large speed differences in subsequent segments for example 30 to 50 to 30 kilometers per hour on 3 subsequent segments, may not be realistic for practice. In the simulation, the variable speed included some subsequent segments where these large differences occur. The simulation results indicate that the variable speed can significantly reduce the number of stops on the main directions, even when the speeds contain large differences per segment. Constraining the variation per segment should therefore only be done based on (concerns about) driver comfort. The model used to optimize the variable speed, MAXBAND, already includes constraints that can limit the maximum change in reciprocal speed on subsequent segments, if desired. In some networks, for example when the shortest segment is directly followed by the longest segment, disallowing large speed differences may reduce performance.

Another assumption, made in chapter 4, was that changing speed between subsequent segments happens instantaneously. This is not realistic in practice. The research in chapter 4 essentially assumed infinite acceleration and deceleration. However, when the variable speed is interpreted as an average speed on the segment and the bounds as bounds on the average speed, we see that the conclusions still hold. In VISSIM, default values for maximum acceleration and deceleration (per vehicle class) were used. The way VISSIM handles acceleration and deceleration is quite complicated and also includes some randomness, which is not further discussed here. The simulation results indicate that the variable speed can significantly reduce the number of stops on the main direction, even when the effects of acceleration and deceleration are included.

The model chosen and built for optimizing the variable speed, MAXBAND, was used as a basis for simulations with the variable speed as well as for simulations with the fixed speed. It was assumed that this method results in an objective comparison of the usefulness of a variable speed for over a fixed speed. While MAXBAND was found very suitable for optimizing with a variable speed (especially also considering the additional constraints), this is not necessarily the case for the fixed speed. The output produced by MAXBAND for the fixed speed led to quite a poor coordination with a high cycle time and low bandwidth. The comparison between the fixed speed and variable speed is done as objectively as possible, only changing the minimum speed parameter in MAXBAND between the two. However, choosing to use MAXBAND for the fixed speed timing plan, may be an inherent disadvantage for a coordination with a fixed speed. Consequently, the results may be skewed in favor of the variable speed, such that it is unclear whether good results come from the benefits of the variable speed or the disadvantages of using MAXBAND with a fixed speed. Section 5.3 has shown that, in some different cases, with different inputs, MAXBAND can produce efficient timing plans for the fixed speed.

During the research of the variable speed, a 100% compliance rate was assumed. Application of the variable speed for coordination in two directions has shown a significant reduction in stops, such that it is interesting to continue research into the variable speed and even test it in practice. The level of control and compliance by which the speed was tested is not realistic for practice. Though this very much depends on the way the speed is implemented. The biggest implementation differences regarding the variable speed are whether it is implemented as a speed advice or compulsory speed. Both are possible and both could realize the benefits of the variable speed to some extent, however a compulsory speed would best resemble the research regarding a variable speed in this thesis. In reality, a compulsory variable speed, such as used in this research can be realized using ISA technology (European Road Safety Center, n.d.). This technology is already required in every newly manufactured vehicle. In Levin et al. (2019), it was found that at lower compliance rates, variable speed limits can still benefit energy savings. The complying vehicles form a moving bottleneck, such that other vehicles are forced to reduce the speed. This phenomenon is also applicable to the variable speeds considered in this thesis, when compliance rates are not 100%.

Using the variable speed to coordinate between efficient signal timing plans is a difficult topic to give general remarks about. Chapters 4 and 5 have attempted to stay as general as possible in their conclusions. However, in this discussion it must be stated that the coordination between efficient timing plans, and whether the variable speed provides any benefits over the fixed speed, depends on a large amount of network specific factors. In general, differences in segment lengths and the allowed internal time difference between outbound and inbound coordinated green windows were the two most important factors relating to the usefulness of a variable speed for coordination between efficient signal timing plans. However, there are cases imaginable, with large differences in segment lengths and small internal time differences, where coordination between efficient timing plans could be possible with a fixed speed. In such a case (where the stars align), the little room available in internal time difference, the desired queue clearance times, the available green times and cycle time have to align perfectly to overcome the differences in segment lengths. These cases are not shown, nor extensively discussed, as they are not useful for a general conclusion.

9.2. DEMAND PREDICTION RESEARCH

As mentioned in chapter 6, the demand prediction tests in this thesis are based on traffic volume measurements at the stop line, which is a lower bound for true demand, at best. For the research in this thesis, this assumption has not affected the results, as it was carefully monitored that no oversaturation takes place.

Prediction tests of the true traffic volume had to deal with differences in optimization period length and time intervals that do not align. Another challenge is that downstream accuracy in the prediction test was affected by different control decisions made upstream, compared to the reference simulation. In this thesis, the accuracy of the prediction was quite high and definitely closer to the true traffic volume than in the reference simulation, partly because the effects of rounding and partly because the differences in the control decisions were only minor.

9

The potential of predictions is only tested on a single network, for a single case. The analysis of the model output, discussed in section 8.3, showed that for the investigated case, the morning rush hour traffic demand did not lead to very different control decisions by the traffic model. For predictions of demand to influence performance, it is required that control decisions are affected by changing demand. This discussion point has a large effect on the conclusion, as it is expected that in other networks, there is a lot more variation in traffic model output based on varying traffic demand input. Such that in other networks the applied methodology could lead to significant improvements. In general, TopTrac tries to establish a coordination in the network that is sufficient for the given demand. Normally, increasing demand motivates an increase in green time and/or cycle time. In the investigated case, a relatively long green time and cycle time are already applied, based on the established coordination, such that an increase is not needed (and thus not applied) to fulfill the demand. Figure 8.9 best explains this phenomenon.

9.3. GENERAL DISCUSSION

The considered demand scenario in the simulations is realistic. However, the demand may grow in the future and vary highly on other networks. The variable speed limit may improve on the number of stops, but also increases the time spent in the network, decreasing overall throughput. A decrease in throughput lowers the demand threshold that causes congestion. If the arterial is congested, the variable speed (or any other coordination designed for a green wave) will no longer be effective. Coordinated traffic signal control in general, requires a slightly different approach for over-saturated conditions. Namely, some other objective may be more important than minimizing delay and stops. Nearing congestion, maximizing throughput may be more import to avoid congestion. When congestion has happened, avoiding the blocking back of intersections and the places to 'store' the congestion also become a concern. In such a case, a different method, like the back-pressure method presented in van Rest (2017) may be useful. Demand predictions can potentially predict oversaturation. When oversaturation can be predicted, the traffic controller can switch optimization objective before the arterial becomes congested, potentially avoiding the congestion. This is an advantage of using demand predictions, which is not researched in this thesis.

10

CONCLUSION

This chapter presents the conclusions of the research of this thesis. The conclusions are presented as answers to the main research questions. These answers build on the answers to each of the sub-questions, given in the previous chapters. The sub-questions and their answers are not repeated here, but the content is used in the answer to the main research questions.

10.1. CONCLUSION OF THE VARIABLE SPEED RESEARCH

The research question regarding the variable speed research is as follows:

> *What are the benefits, for the coordination of traffic controllers, in terms of reduced stops and delay, when optimizing signal timing plans with variable speeds compared to fixed speeds in an undersaturated demand scenario?*

A theoretical analysis of the variable speed, performed in chapter 4, showed that the variable speed does not make coordination in one direction under different cycle times attractive, which was discussed in section 4.1. However, for coordination in two directions, the variable speed was shown (in section 4.2) to be able to realize a feasible coordination between constrained signal timing plans, in a way that a fixed speed is not able to. Section 4.2 showed that, with a variable speed, coordination between locally optimal signal timing plans is more often feasible. Particularly when the starts of green on the coordinated directions are (nearly) simultaneous in the locally optimal signal timing plan. To be able to use and test these findings in a more practical setting, a model is needed that can find the optimal variable speeds and coordinated signal timings in the network. Based on the surveyed literature in chapter 2, the MAXBAND model was found most suitable. This model was programmed and further expanded to create a practical coordination in chapter 5. Using the MAXBAND has produced findings that are in line with the findings of section 4.2. Namely that the relative benefits of the variable speed over the fixed speed for coordination between efficient timing plans largely depends on the differences in segment lengths and the allowed internal time difference between outbound and inbound coordinated green windows.

Using the model, coordinated signal timings were created for both the fixed speed and the variable speed. These timings were included in signal timing plans to be used in the simulation environment and network laid out in chapter 7. The simulation results, presented in section 8.1, show that the variable speed leads to a reduction in stops from 11500 to 8569 (-25.5%) in the whole network, compared to the fixed speed. This is a direct consequence of fewer stops on the coordinated directions, the uncoordinated directions stayed equal in terms of number of stops. The delay, expressed in total time spent in the network, increased from 212.2 [s/veh] to 214.7 [s/veh] (+1.2%) compared to the fixed speed. This increase in delay is caused by the slower (variable) travel speed between the coordinated directions on the arterial. The variable speed allowed to coordinate with a lower cycle time. This resulted in a decrease in delay on the side directions from 27.6 [s] to 22.2 [s] of (-19.6%). Overall, using a variable speed for a more efficient coordination in two directions has shown to benefit the coordination greatly in terms of stops on the main directions, without negatively affecting the side directions. As discussed in section 9.1, the theory is subject to numerous assumptions and the results are only based on one case. However, this case is realistic and there is no reason to assume that similar benefits can not be realized in other cases. As discussed in section 1.1.1, a reduction in stops not only increases driver comfort, it decreases fuel consumption, reduces negative side effects of noise and emissions and can decrease the amount of rear-end collisions.

10.2. CONCLUSION OF THE DEMAND PREDICTION RESEARCH

The research question regarding the demand prediction tests is as follows:

> *For TopTrac, what are the maximum benefits, in terms of reduced stops and delay, obtainable by using near-perfect demand predictions compared to normal operations in undersaturated conditions?*

In chapter 6, two different methodologies were laid out for testing near-perfect demand prediction in TopTrac. The first methodology tested demand predictions based on the inputs TopTrac received in a reference simulation. These inputs were shifted forwards in time by one optimization period with the idea that the control decisions would change from a reactive nature to a proactive nature. The second methodology tested demand predictions on the true traffic volume measured in a reference simulation. The idea behind this prediction is to improve the quality of information supplied to the traffic model in TopTrac. Both prediction tests were of a high quality. Simulations with the prediction tests were performed in the simulation environment and network discussed in chapter 7. The results of this analysis, presented in section 8.2, showed that no significantly different results were obtained in the simulations with either of the prediction test methodologies, compared to the reference simulation. As this result was not as expected, the research was expanded. In section 8.3, an analysis of the model output of the TopTrac traffic model was performed, which showed that, for the investigated network, the control decisions did not change significantly over time in any of the simulations. This, even though the traffic pattern changed significantly during the investigated morning rush hour. This observation can be explained as follows: the traffic model tries to find signal timings that create a coordination. To achieve this, longer green times on the coordinated directions and a larger cycle time is needed. This is already achieved in the beginning of the simulation, when demand has not yet peaked. When the demand spikes, the traffic is already controlled with sufficient green time and a large enough cycle time (which were based on the established coordination). So the model sees no reason to make a different control decision based on the spike in traffic volume. Further explanations look at the side directions, which have such little demand, that even at the largest demand, the queue can be fully cleared with just the minimum green time. So there is no reason for the model to increase the green time on these directions and it was observed that this stays constant across the whole simulation. Logically, when no significantly different control decisions are made, no significant difference in the final results is observed. As discussed in section 9.2, the conclusions regarding the demand prediction tests are based on a single test case. It is unclear whether this test case, where the control decisions do not change significantly over time, is an isolated case, or whether this occurs on other arterials as well. Section 10.3.3 discusses the recommendations for Vialis, based on this conclusion. Parts of these conclusions are applicable in general, to all coordinated traffic controllers. When attempting to use demand predictions in the controller, first investigate how strongly control decisions are affected by demand. Especially in larger networks with coordination in two directions, it is not trivial that for example a steep increase in demand leads to an increase in cycle time. Namely, a high (enough) cycle time may already be in place to be able to provide a coordination at a low demand.

10.3. RECOMMENDATIONS AND FUTURE RESEARCH

This section discusses the recommendations and future research directions. Future research regarding the variable speed is recommended and this is discussed in section 10.3.1. Recommendations for Vialis, based on the demand prediction research in TopTrac is discussed in section 10.3.3.

10.3.1. FUTURE RESEARCH

Regarding research into coordination with different cycle times in one direction, it would be insightful to see a performance comparison between coordination in for example 35% of cycles with different cycle times (e.g. 62 and 80 seconds), compared to coordination in 100% of cycles with equal cycle times (80 and 80 seconds). This future research should also consider coordination with a part of the green window, not only between the starts of green.

The most promising area for future research based on this thesis is regarding the use of the variable speed for coordination in two directions. Future research could look at other networks/arterials, to investigate the use of the variable speed there, in the same form as in this thesis, for example with timings generated by the MAXBAND program. Future research could also consider the effects of a low(er) compliance rate. A more advanced application of the variable speed can include a variable speed in a real-time controller. MAXBAND is not directly suitable for this application, though the computation time of the program is not an issue. Rather, MAXBAND looks for a global optimal solution, which may be a problem for real-time control, as a slight change in the input can result a large change in the output. Namely, slightly different input can result in a different local optimum (with very different output values) becoming the new global optimum. This is undesirable in real-time control, as it may lead to grave conflicts. For example, in a first optimization the speeds may be much lower than a follow-up optimization such that the two platoons may collide, or halfway across the arterial the first platoon may not receive coordination anymore, based on the second optimization. Other issues may arise when, for example, the first optimization finds a short cycle time and the follow-up optimization finds a long cycle time. The reason for these problems is that the model considers each optimization as isolated, without regards for decisions or changes of the past and future. Furthermore, MAXBAND is not directly suitable for real-time application as the program does not consider, amongst other things, side directions, minimum and maximum green times, clearance times and yellow time, which are needed for valid and safe signal timing plans in real time. Moreover, based on coordinated traffic controller architecture discussed in section 1.1.3 and the survey of controllers performed in section 2.1, not all controllers are suitable to include the variable speed. At least some central entity is needed that has an overview of the entire arterial, to decide the optimal speeds per segment. Future research into applications of the variable speed in real time should keep this in mind. The base signal timing plan can be obtained via MAXBAND, such that the real-time controller only slightly/gradually modifies the initial settings. For example, slight modifications of the speed can be used to account for the tail of the queue in real-time. Accurate measurements of the tail of the queue are possible in a Connected Vehicle environment (Rostami-Shahrbabaki et al., 2020).

Future research may also deepen the concept of the variable speed. There are more things possible if the variable speed is allowed to vary in time not only per cycle, but also per signal group. This opens the door for coordination of more than two directions. When coordinating multiple upstream signal groups with a single downstream signal group, the upstream signal groups cannot both be coordinated with the start of green of the downstream signal group. The same methodology used in this thesis, of looking at the coordination of the start of green, cannot be used to research this case. Coordinating two upstream signal groups with the start of green of one signal group direction will result in unrealistic conflicts.

Another idea to use the variable speed in some networks, is to look at networks where cyclists are present. Particularly in networks where the green windows for cyclists can be part of the same stage with the coordinated green windows for motor vehicles going the same direction as the cyclists. The (variable) speed for coordinated directions of motor vehicles can be set such that the obtained offset is two, three or four times lower than the offset required for the coordination of the cyclists, such that both can be coordinated along the same segment. Future research can investigate the feasibility of this idea.

10.3.2. Recommendations

Regarding the variable speed, it is recommended to conduct more research such as the research mentioned in section 10.3.1.

Regarding the demand prediction tests, it is recommended to anyone researching the use of demand predictions in any coordinated traffic controller (not only TopTrac), that prior to the research, some data is gathered. This data should contain, for the same time period, traffic volume data and some output data from the traffic controller. This research has shown that it should not be assumed that the coordinated traffic controller makes different control decisions based on a different traffic demand. Only when changing traffic demand leads to changing control decisions, will a prediction of this traffic demand have any effect. Gathering some data about controller output decisions in response to demand input, will give an indication about the responsiveness of the controller to changing demand.

Future work should also consider the use of demand predictions in networks where oversaturation may occur. A different methodology is needed to make predictions during oversaturation, as stop line measurements will simply measure capacity, not demand. When coordinated traffic control can switch control objective (from creating a green wave to maximizing throughput), based on demand predictions, before oversaturation happens, this may lead to significant performance gains.

10.3.3. RECOMMENDATIONS FOR VIALIS

For Vialis, it is recommend that some data is gathered from all arterials that are currently run by TopTrac. For each arterial, it is recommended to make a plot of the cycle time evolution against time for the morning rush hour. Also include the demand over time and compare the plots. Using verkeer.nu, obtaining this data will be a straightforward task. The obtained plots will indicate whether there are large differences in the control decisions based on differences in demand. Based hereon, it can be concluded if the investigated case in Almere, where there were no large changes over time, is an isolated case.

If there are other arterials showing the same pattern, any further investments into demand predictions are not recommended. The reason for this recommendation is that demand predictions will have no significant effect on arterials that are similar to the investigated arterial in Almere. Without the aforementioned data on other arterials, it is hard to say anything about the usefulness there. But it should be kept in mind that, with minor differences, the performance differences will only be minor as well. And, most importantly, the near-'perfect' prediction, shown in this thesis, can never be realized to this level of accuracy in reality.

If the investigated case in Almere is an isolated case, it is recommended to investigate another case, where there are large changes over time, to see the effect of near-'perfect' demand prediction tests there. The (software) tools built and methodology employed during this thesis can be used to investigate another case relatively quickly and cheaply, though the effects of oversaturation and different control decisions on the quality of the prediction tests, must be carefully monitored.

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10

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A

Appendix A: Mathematical proof of coordination Equation 4.12

This appendix provides the detailed proof for Equation 4.12 in section 4.1, where the derived Equation 4.11 is taken as a starting point. As a reminder, Equation A.1 resembles Equation 4.11 from section 4.1.

$$T_{off,\hat{\imath},n} = T_{off,\hat{\imath},0} + ((C_{i+1} - C_i) \cdot n) \bmod C_{i+1} \tag{A.1}$$

We are interested in the behaviour of $T_{off,\hat{\imath},n}$. Since $T_{off,i,0}$ is a constant, we are really interested in the behaviour of the left hand side of Equation A.2. This is the amount of time added to $T_{off,\hat{\imath},0}$, and when we know it's values we will also find the values of this + $T_{off,i,0}$. Thus we can remove $T_{off,\hat{\imath},0}$ from our equations for now, which is useful since we know $T_{off,\hat{\imath},0}$ is not per definition an integer.

Based on the definitions and assumptions stated in section 4.1.1, we can assume that the cycle times are integers greater than 0, where as per our choosing in section 4.1: $C_{i+1} > C_i > 0$. Then the outcome of the left hand side of Expression A.2 must be an integer as well and we define this (integer) outcome as $\delta T_{i,n}$. $\delta T_{i,n}$ is the time added to the initial offset to get the offset at the current cycle.

$$((C_{i+1} - C_i) \cdot n) \bmod C_{i+1} \equiv \delta T_{i,n} \tag{A.2}$$

The modulo congruence relation in Equation A.2 satisfies all the conditions of an equivalence relation, including the symmetry condition (Wikipedia contributors, 2022d): $a \equiv b (\bmod n)$ if $b \equiv a (\bmod n)$ for all a, b, and n. Applying the symmetry condition to Equation A.2, we get Equation A.3 and we see that we are really trying to find the values of $\delta T_{i,n} \bmod C_{i+1}$

$$\delta T_{i,n} \bmod C_{i+1} \equiv (C_{i+1} - C_i) \cdot n \tag{A.3}$$

A

Modulo

From the definition of the modulus in modular arithmetic (Wikipedia contributors, 2022d), two integers a and b are said to be congruent modulo M if M is a divisor of $a-b$. In other words there exists an integer k such that $a-b=k\cdot M$

From the Modulo definition, we can rewrite Equation A.3 to Equation A.4.

$$(C_{i+1}-C_i)\cdot n-\delta T_{i,n}=k\cdot C_{i+1} \tag{A.4}$$

Reordering Equation A.4, we get Equation A.5. Since k can be any integer (also negative) we can further rewrite Equation A.5 to A.6

$$(C_{i+1}-C_i)\cdot n-k\cdot C_{i+1}=\delta T_{i,n} \tag{A.5}$$

$$n\cdot(C_{i+1}-C_i)+k\cdot C_{i+1}=\delta T_{i,n} \tag{A.6}$$

Bézout's Identity

Bézout's Identity (Wikipedia contributors, 2022a) is a theorem that states: when two integers a and b have a greatest common divisor d ($d=\gcd(a,b)$), there exist two integers x and y such that $ax+by=d$. Moreover, using integers p and q, the integers of the form $ap+bq$ are exactly the multiples of d.

The crucial step in this proof is recognizing that Equation A.6 has the same form as the equation in the definition of Bézout's Identity. We choose p as n and q as k. Then when setting a as $(C_{i+1}-C_i)$ and b as C_{i+1} we see that $\delta T_{i,n}$ will take solutions of the form $m\cdot\gcd((C_{i+1}-C_i),C_{i+1})$, with m an integer used to signify the multiples of $\gcd((C_{i+1}-C_i),C_{i+1})$.

In other words this means that the time added to the initial offset, to get the 'ideal' offset in the current cycle ($\delta T_{i,n}$) will be a multiple of the greatest common divisor of $(C_{i+1}-C_i)$ and C_{i+1}. Referring back to Equations A.2 and A.3 and knowing the forms of the values of $\delta T_{i,n}$ we see that we obtain Equations A.7 and A.8 showing that both sets of values are equivalent.

$$((C_{i+1}-C_i)\cdot n) \bmod C_{i+1}=m\cdot\gcd((C_{i+1}-C_i),C_{i+1}) \quad \exists m \tag{A.7}$$

$$(m\cdot\gcd((C_{i+1}-C_i),C_{i+1})) \bmod C_{i+1}=(C_{i+1}-C_i)\cdot n \quad \exists n \tag{A.8}$$

We use Equation A.8 and the fact that $\gcd((C_{i+1}-C_i),C_{i+1})$ also divides C_{i+1} to see the following interesting properties:

- $C_{i+1}-\gcd((C_{i+1}-C_i),C_{i+1})$ is the greatest multiple of $\gcd((C_{i+1}-C_i),C_{i+1})$ that is not equal to C_{i+1}. Therefore it must be the maximum, as the next multiple of $\gcd((C_{i+1}-C_i),C_{i+1})$ will equal C_{i+1}, resulting in: $C_{i+1} \bmod C_{i+1}$ which equals 0.

- From Equation A.8 we see that whenever m is a multiple of $\frac{C_{i+1}}{\gcd((C_{i+1}-C_i),C_{i+1})}$, $\delta T_{i,n}$ will be 0, meaning the initial offset will once again provide coordination. More generally, we see that whenever $m \cdot \gcd((C_{i+1} - C_i), C_{i+1}) = C_{i+1}$ the cyclic values of $\delta T_{i,n}$ repeat themselves. This repetition number is denoted $R_{\hat{i}}$ and is given by Equation A.9. Since the sets given by m and n contain the same integers, the values given by n must also repeat themselves every $R_{\hat{i}}$, such that Equation A.10 is true.
- Since $\delta T_{i,n}$ only takes values of the form $m \cdot \gcd((C_{i+1} - C_i), C_{i+1})$ and every value is repeated every $R_{\hat{i}}$ times, we can conclude that $\delta T_{i,n}$ will at some point be equal to every value $m \cdot \gcd((C_{i+1} - C_i), C_{i+1})$ for all integers $m \in \{0, R_{\hat{i}} - 1\}$. Including the aforementioned maximum value. Namely, all unique values of m must lead to unique values of $\delta T_{i,n}$ within the repetition cycle.

$$R_{\hat{i}} = \frac{C_{i+1}}{\gcd((C_{i+1} - C_i), C_{i+1})} \tag{A.9}$$

$$((C_{i+1} - C_i) \cdot n) \bmod C_{i+1} = ((C_{i+1} - C_i) \cdot (n + R_{\hat{i}})) \bmod C_{i+1} \quad \forall n \in \mathbb{N} \tag{A.10}$$

In the next step we want to show that Equation A.11 is true.

$$\gcd((C_{i+1} - C_i), C_{i+1}) = \gcd(C_i, C_{i+1}) \tag{A.11}$$

To prove this, we use a property of the greatest common divisor.

Greatest common divisor

We have the greatest common divisor of a and b, $d = \gcd(a, b)$. Suppose $a > b$. Since d divides both a and b and a is larger than b, d must also divide $a - b$. Thus we have $\gcd(a, b) = \gcd(a - b, b)$ if $a > b$. This property is mentioned in Wikipedia contributors (2022c) and famously used in the Euclidean Algorithm (Wikipedia contributors, 2022b).

We can see that $C_{i+1} > (C_{i+1} - C_i)$ since $C_{i+1} > C_i > 0$. Using the property of the greatest common divisor, and choosing $a = C_{i+1}$ and $b = (C_{i+1} - C_i)$ we find that Equation A.12 must be true.

$$\begin{aligned} \gcd((C_{i+1} - C_i), C_{i+1}) &= \gcd((C_{i+1} - C_i), C_{i+1} - (C_{i+1} - C_i)) \\ &= \gcd((C_{i+1} - C_i), C_i) \end{aligned} \tag{A.12}$$

The greatest common divisor obtained from equation A.12 has the form of $\gcd(a - b, b)$ with $a > b$. Thus, by applying the property in reverse we find that Equation A.13 must be true. Furthermore, the order of the numbers inside the gcd does not matter and can be rearranged.

$$\gcd((C_{i+1} - C_i), C_i) = \gcd(C_i, C_{i+1}) \tag{A.13}$$

So, in all previous equations, $\gcd((C_{i+1}-C_i),C_{i+1})$ can be replaced with $\gcd(C_i,C_{i+1})$. We have now proven that Equation A.11 is true. Furthermore, we have discovered some interesting properties of $\delta T_{i,n}$ that will be useful when analyzing suitable offsets. Equation A.14 summarizes the most important findings/properties.

$$((C_{i+1}-C_i)\cdot n) \bmod C_{i+1} = m\cdot\gcd(C_i,C_{i+1}) \quad \exists m\in\left\{0,R_{\hat{i}}-1\right\} \tag{A.14}$$

To finalize the proof, we can use Equation A.14, to translate Equation A.1 to Equation A.15.

$$T_{off,\hat{i},n} = T_{off,\hat{i},0} + m\cdot\gcd(C_i,C_{i+1}) \quad \exists m\in\left\{0,R_{\hat{i}}-1\right\} \tag{A.15}$$

B

Appendix B: MAXBAND model input and output

This appendix reports, in detail, the steps taken to produce the input for the variable speed model (MAXBAND). Afterwards, the steps taken in converting the output of MAXBAND to signal timings plans ready for simulations are shown.

B.1. Creating the input for MAXBAND

Table B.1 shows the values of various input parameters. Most of them are derived from the simulation network and the current coordinated traffic control settings applied in practice. For example, the minimum and maximum cycle time are chosen equal to the values used by TopTrac on the same network. Regarding the speeds, 50 kilometres per hour is the maximum speed on the arterial in practice. A value of 30 kilometres per hour is seen as an appropriate lower bound, allowing a lot of variation, but still keeping the speeds realistic in the context of a potential practical application of the variable speed. Regarding this variation, no constraint was put on the maximum speed difference between consecutive segments. A value of 0.05 is used as input, but this could have been any value larger than $\frac{1}{30} - \frac{1}{50} = 0.0134$. This choice allows for maximum variation and use of the variable speed, testing it's full potential.

From traffic volume data it was found that the demand for the inbound and outbound directions is quite similar. The outbound direction starts with fewer volume (1246 veh/h) than the inbound direction (1446 veh/h). However, these values shift gradually when heading downstream and at the final intersection the outbound direction has more volume (1846 veh/h) than the inbound direction (1127 veh/h). The total demand (summed over all segments) is very close, 8410 veh/h for the outbound and 8222 veh/h for the inbound direction. Therefore the choice was made to set the weight factor at 1.

Segment lengths were measured between consecutive outbound (02) and inbound (08) stop-lines, both in VISSIM and in Google Maps. Both sources agreed to great accuracy and the values were also equal to those used by TopTrac. The values for the segment

B

Table B.1: Single input values.

Single input	**Value**
Number of signals n	7 [-]
Minimum cycle time CT_{min}	66 [s]
Maximum cycle time CT_{max}	100 [s]
Minimum speed v_{min}	30 [km/h]
Maximum speed v_{max}	50 [km/h]
Lower limit on change in reciprocal speed h	0.05 [s/m]
Upper limit on change in reciprocal speed g	0.05 [s/m]
Band weight factor k	1

lengths are shown in table B.2.

Table B.2: Segment input values.

Segment number [-]	**Length outbound [m]**	**Length inbound [m]**
1	374	358
2	265	292
3	277	277
4	305	298
5	285	281
6	305	305

The signal input values are determined as follows. At Vialis, a license is available for a traffic signal design program called COCON. This software is developed by DTV Consultants and can be a useful tool in the design of signal timing plans (DTV, n.d.-a). This software was used to assist in finding the appropriate signal input values. In COCON a file was constructed featuring all the relevant information about the traffic signals from practice. For example, this information includes traffic volumes, clearance times, capacities, minimum green and minimum red times, conflict matrices, yellow times and yellow use times. Using this information, some traffic signal analyses can be performed, which can yield, for every intersection, minimum cycle times and for every direction, appropriate green times, all to achieve a certain degree of saturation.

The red time values shown in Table B.3 are computed as 1 minus the 90% degree of saturation green time of the main direction divided by the minimum cycle time, based on the classical cycle time method by Webster (Webster, 1958). Similarly, the left-turn times shown in Table B.3 are computed as the 90% saturation green time for this direction, divided by the cycle time. For the main directions, this approach worked nicely, however, for the left-turn directions, the 90% saturation green time is often lower than the minimum green time. In these cases, not the 90% saturation green time is used, but the minimum green time is used. Nearly all intersections have a cycle time around 60 to 65 seconds and minimum green times of 6 seconds. Technically, this process could have been executed iteratively, as MAXBAND optimizes for the cycle time. However, in the end, when the MAXBAND output was converted back to signal timing plans, the left-turns would scheduled with minimum green (or a balanced saturation degree) anyway, as will be discussed later in this appendix. So therefore, this approach was chosen as more accurate, since it doesn't assume a coordination (network cycle time), but let's the model (MAXBAND) create the coordination from individual intersection settings.

COCON also shows typical average queue lengths, based on the traffic volume and the minimum cycle time. These queue lengths were converted to queue clearance times using the capacity and the cycle time. Again, this method is somewhat arbitrary, but also not completely random. In truth, the queue length calculation in COCON has no regard for the network effects present in the network, so it gives a slight overestimation of the true queue length. All in all, getting the input values exactly right is not a major concern of this research, as it is not an optimization study of the considered network. Rather an objective comparison between the fixed and variable speed is desired, which is achieved by keeping all input parameters (except the minimum speed) equal between the two model runs. Note that queue clearance times are logically equal to 0 at the start of the arterial, since the band does not encounter any queue here. In theory, a very small queue clearance time could have been specified to allow for acceleration time of the platoon.

Table B.3: Signal input values.

Signal number [-]	**Outbound red time [cycles]**	**Inbound red time [cycles]**	**Outbound left-turn time [cycles]**	**Inbound left-turn time [cycles]**	**Outbound queue clearance time [cycles]**	**Inbound queue clearance time [cycles]**
1	0.586	0.619	0.103	0.104	0	0.024
2	0.585	0.589	0.101	0.104	0.030	0.038
3	0.621	0.680	0.113	0.106	0.023	0.019
4	0.566	0.576	0.095	0.099	0.044	0.044
5	0.647	0.482	0	0.429	0.038	0.020
6	0.495	0.520	0.091	0.099	0.069	0.025
7	0.497	0.557	0.084	0.097	0.035	0

B.2. CONVERTING THE OUTPUT OF MAXBAND

The output of MAXBAND was methodologically converted back to practical signal timing plans. This method was as follows: First, the output was converted from cycles to seconds using the optimized cycle time. Then the main directions (here 02 and 08) were put in the timing plan exactly as computed by MAXBAND. The the left-turns (here 03 and 09) were placed directly after the main directions (lag-lag), and in nearly all cases, with the minimum green time duration. Then the two major conflicting side directions (here 05 and 11) were placed, the one with the lower demand before the one with the higher demand. Since these side directions often feed traffic to the main directions, it is desired to have them just before the main direction that is fed. Only one of them can be closest (as they conflict), so it was chosen to be the one with the higher demand. The other side directions (here 04 and 10) were preferably placed just before the main directions as well, but for these directions not much choice was left, and they often end up having green at the same time as the corresponding major side directions. Then finally, with all directions having a place in the signal timing plan, the green times (except those of the main directions) are adjusted to divide the load equally across the directions (i.e. obtain similar saturation degrees).

Finally, the start and end of green are converted to semi-fixed maximum start of fixed green time (GF) and maximum end of extension green time (GX), respectively. Initially, the green is scheduled to start at GF. When no other conflicts are present and there is demand for the direction, it can be given green from the start of advance green (GA). Otherwise, it must become green at least at GF. Whenever green has started, and the minimum green time has been exceeded, the green can be terminated. Except when the time of waiting green (GW) has not been passed. For all directions in the COCON timing plan, the GF and GX are implemented. Only for the main directions, the GA and GW are implemented on top of the GF and GX. This way, the side directions are as flexible as possible, started whenever there is no conflicts active and demand present and terminated whenever there is no demand or the GX has passed.

For the main directions, this flexible behaviour is not desired, as we intend to have them coordinated. So for all main directions some methodology was adopted with regards to the GA and GW. In general, upstream in the network GA is desired more closely to the GF, as we don't want to realize the green to soon, potentially having the platoon stranded somewhere downstream where the GA cannot be realized. For the same reason, downstream, GA can be chosen more freely, allowing for more potential to clear any queues. With regards to the GW the opposite is true. Upstream, we don't mind the GW to be far from the GX, when there is no demand, the green may then safely be terminated. However, more downstream, the GW should preferably be closer to GX. Downstream, it may happen that the platoon arrives a little late during the green phase, and we don't want the green to terminate before the platoon has arrived. For example, in Figure 5.4, the outbound green band at intersection 2021 is near the end of the green phase. For this direction the GW should be specified close to the GX, otherwise the green would terminate after the queue has been cleared and the minimum green time has passed.

The values for GA and GW are preferably kept as close to each other as possible between the simulations with fixed speed and variable speed. The choice was made to adopt the same values between both simulations with the network, the values are shown in Table B.4.

B

Table B.4: Values for the semi-fixed advance green (GA) and waiting green (GW) on the main directions.

Signal number [-]	**Outbound (02) direction**		**Inbound (02) direction**	
	GA to GF [s]	**GW to GX [s]**	**GA to GF [s]**	**GW to GX [s]**
1	2	20	15	10
2	7	15	10	10
3	10	15	10	10
4	10	15	10	15
5	10	15	2	15
6	5	15	5	15
7	15	20	2	20

C

Appendix C: Additional results demand predictions

This appendix shows some additional results, on top of the ones presented in Chapter 8. The results presented here are intended to show that not only on network level, but also on a direction or intersection level, and over time, results are not significantly different. It should be noted that results on lower aggregation levels, like the ones presented here, may seem to show more differences. Underneath these differences is also show a lot more variance between runs. This makes the final outcome still insignificantly different, as it cannot be excluded that differences seen are due to randomness of the simulation. A disclaimer is appropriate for this appendix: A lot of different results are shown, but it simply is not feasible, nor useful, to include results of every single measurement. Therefore, a selections is made, to show those results that I deemed most representative.

Figures C.1, C.2, C.3 and C.4 show stops results plotted over time. The horizontal axis represents the time during the simulation. As results are collected on set intervals, stepwise plots are the most accurate representation. The stops show minimal differences, and the minor observable differences don't show any consistent pattern regarding the performance of the predictions over time.

Figures C.5 and C.6 show delay results plotted over time. For these, similar to the stop results, there are no consistent patterns visible. Similarly, Figures C.7, C.8, C.9 and C.10 show arterial travel time results plotted over time. At the end of the simulation, there is a steep drop in travel time, down to 0 seconds. This drop is not the result of some magical teleportation, rather near the end of the simulation there simply was not enough time to complete a journey across the arterial, before the simulation ends. Therefore, no travel time measurements can be completed at the very end of the simulation.

Besides results varying over time, it also interesting to look at results varying over space. In Figure C.11, C.12, C.13 and C.14 the results per intersection are displayed as bar charts.

Finally, results can still vary over space, within intersections. Plots with the results per signal group are shown in Figures C.15, C.18, C.17 and C.16.

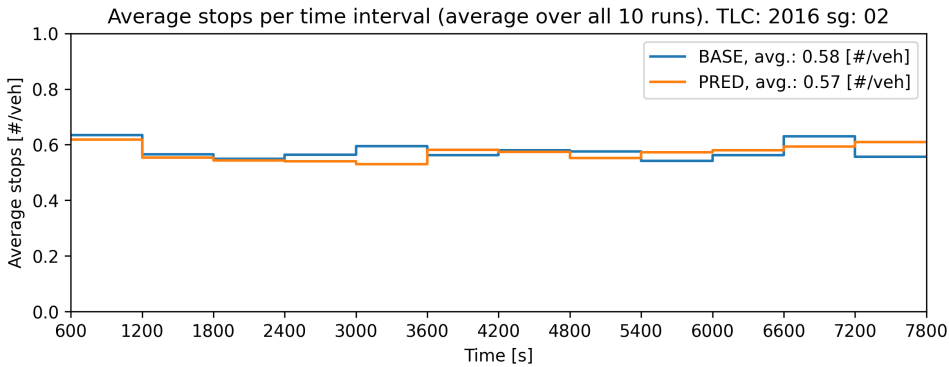

Figure C.1: Reference (BASE) versus first predicted (PRED) average stops for intersection 2016, signal group 02, average (over all runs) per time interval of the simulation

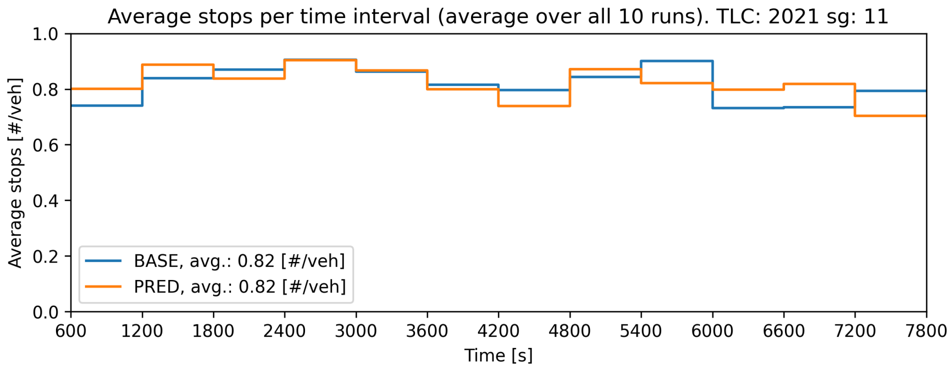

Figure C.2: Reference (BASE) versus first predicted (PRED) average stops for intersection 2021, signal group 11, average (over all runs) per time interval of the simulation

In Tables C.1 and C.2, the network average results per run are displayed. The values in this table show that the averages for both sets of simulations are very close, and there are no real outliers. This is important because, if one run with predictions performed extremely bad, while all the others did better, this may lead to the same average results, but might lead to a different recommendation or even conclusion.

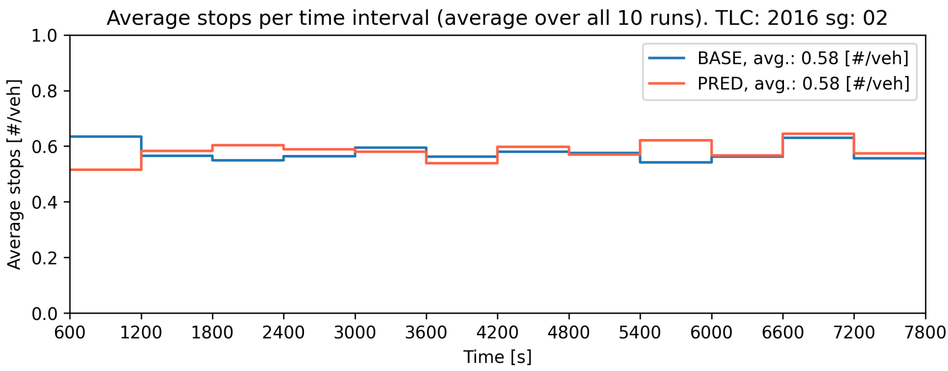

Figure C.3: Reference (BASE) versus second predicted (PRED) average stops for intersection 2016, signal group 02, average (over all runs) per time interval of the simulation

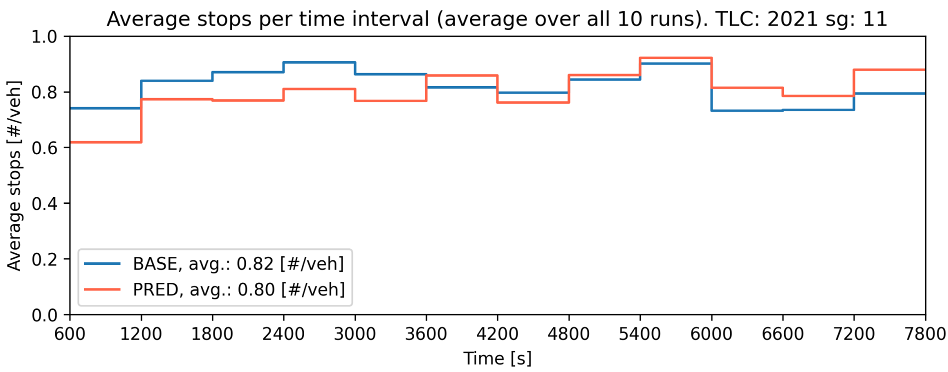

Figure C.4: Reference (BASE) versus second predicted (PRED) average stops for intersection 2016, signal group 02, average (over all runs) per time interval of the simulation

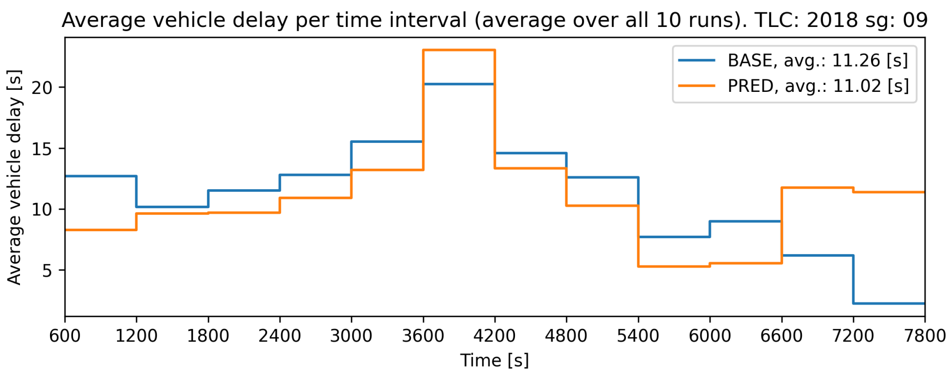

Figure C.5: Reference (BASE) versus first predicted (PRED) average delay for intersection 2018, signal group 09, average (over all runs) per time interval of the simulation

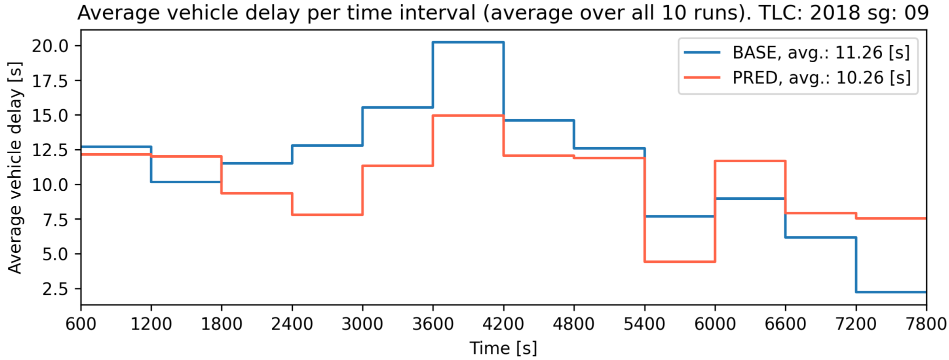

Figure C.6: Reference (BASE) versus second predicted (PRED) average delay for intersection 2016, signal group 02, average (over all runs) per time interval of the simulation

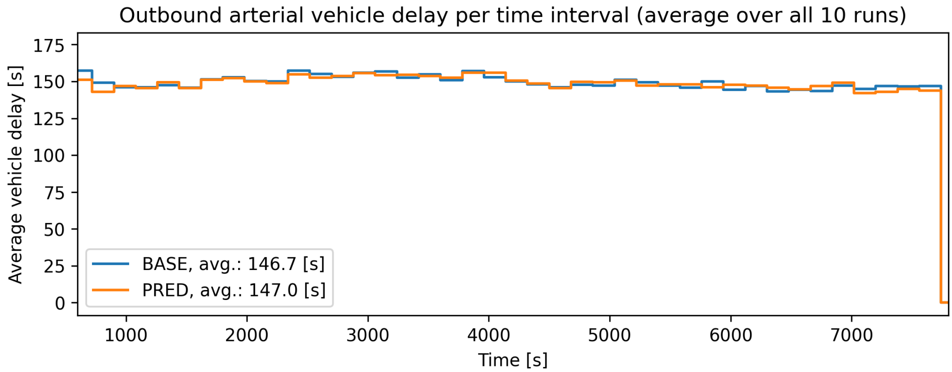

Figure C.7: Reference (BASE) versus first predicted (PRED) outbound arterial travel time, average (over all runs) per time interval of the simulation

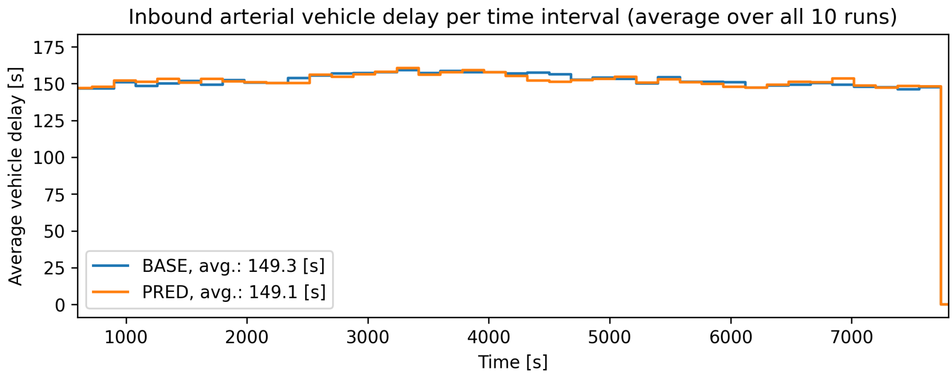

Figure C.8: Reference (BASE) versus first predicted (PRED) inbound arterial travel time, average (over all runs) per time interval of the simulation

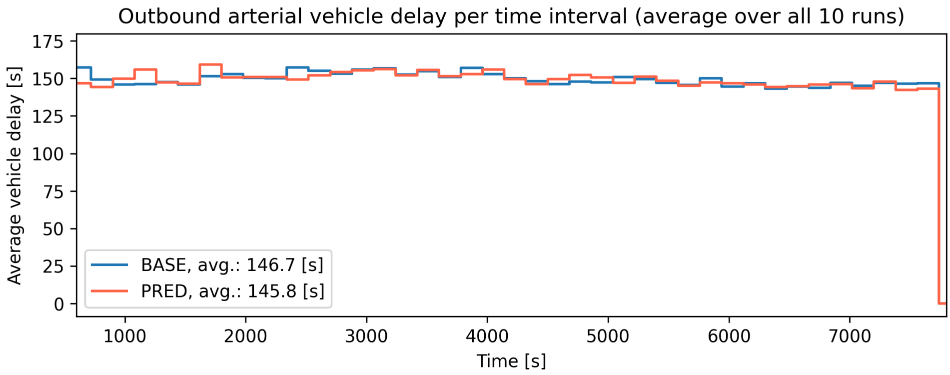

Figure C.9: Reference (BASE) versus second predicted (PRED) outbound arterial travel time, average (over all runs) per time interval of the simulation

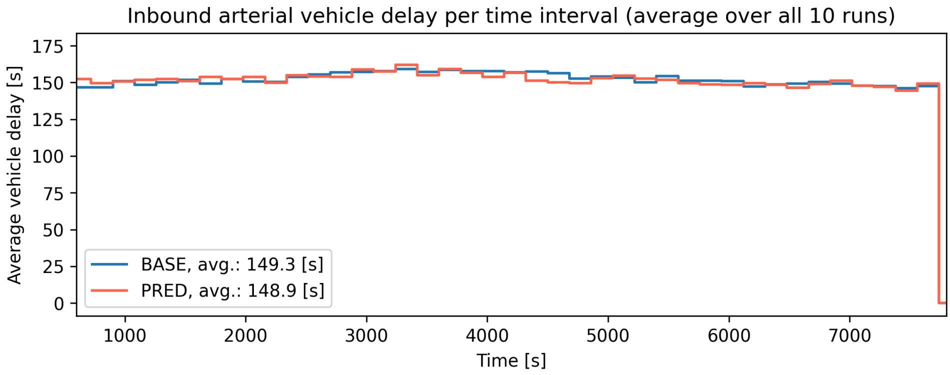

Figure C.10: Reference (BASE) versus second predicted (PRED) inbound arterial travel time, average (over all runs) per time interval of the simulation

Table C.1: Overview of simulation result of traffic demand predictions for the whole network, stops per run.

Run nr. [#]	**BASE Avg. Stops (#/veh)**	**PRED1 Avg. Stops (#/veh)**	**PRED2 Avg. Stops (#/veh)**
Run 1	0.54	0.54	0.54
Run 2	0.55	0.54	0.53
Run 3	0.54	0.55	0.52
Run 4	0.54	0.54	0.55
Run 5	0.55	0.53	0.53
Run 6	0.55	0.55	0.54
Run 7	0.53	0.55	0.53
Run 8	0.55	0.55	0.54
Run 9	0.54	0.54	0.53
Run 10	0.53	0.55	0.55

C

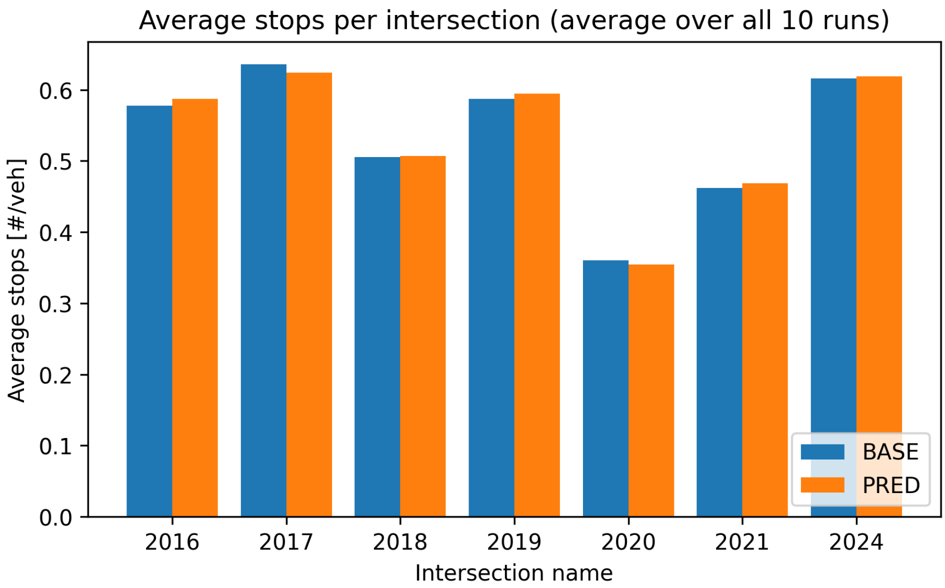

Figure C.11: Reference (BASE) versus first predicted (PRED) average stops per intersection in the network, average (over all runs)

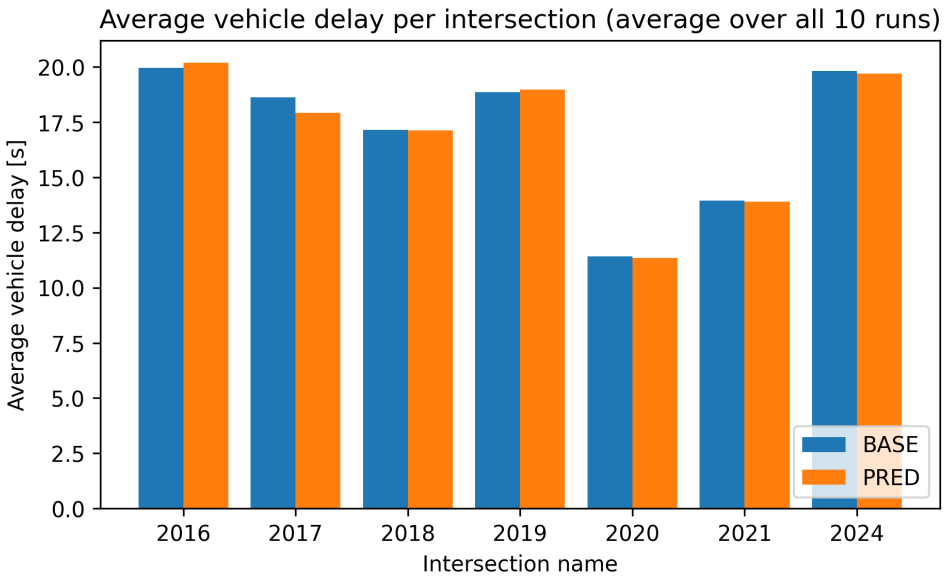

Figure C.12: Reference (BASE) versus first predicted (PRED) average delay per intersection in the network, average (over all runs)

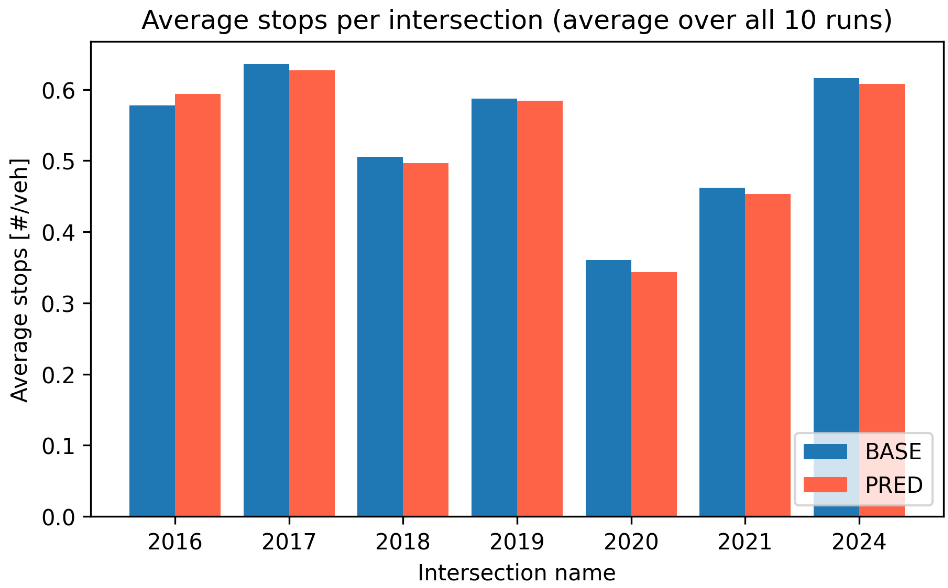

Figure C.13: Reference (BASE) versus second predicted (PRED) average stops per intersection in the network, average (over all runs)

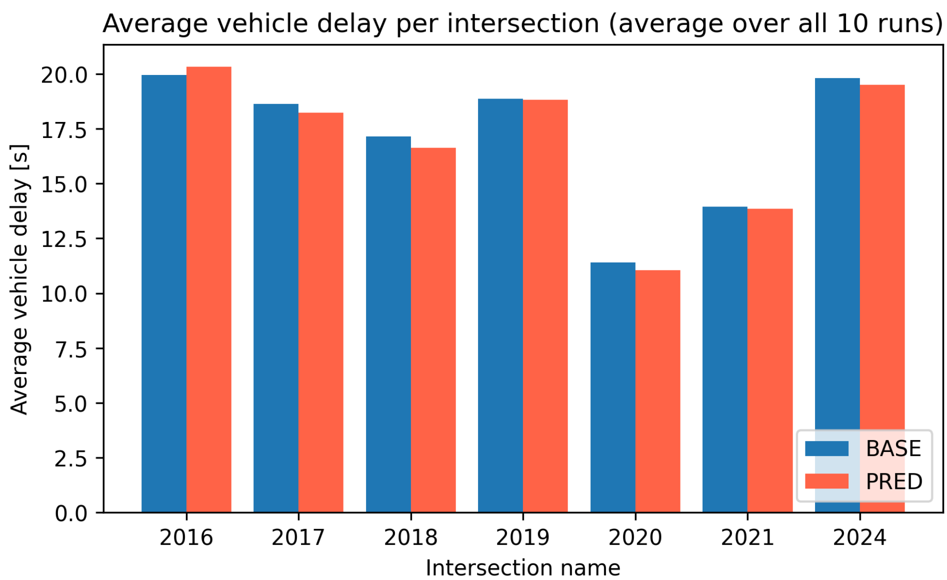

Figure C.14: Reference (BASE) versus second predicted (PRED) average vehicle delay per intersection in the network, average (over all runs)

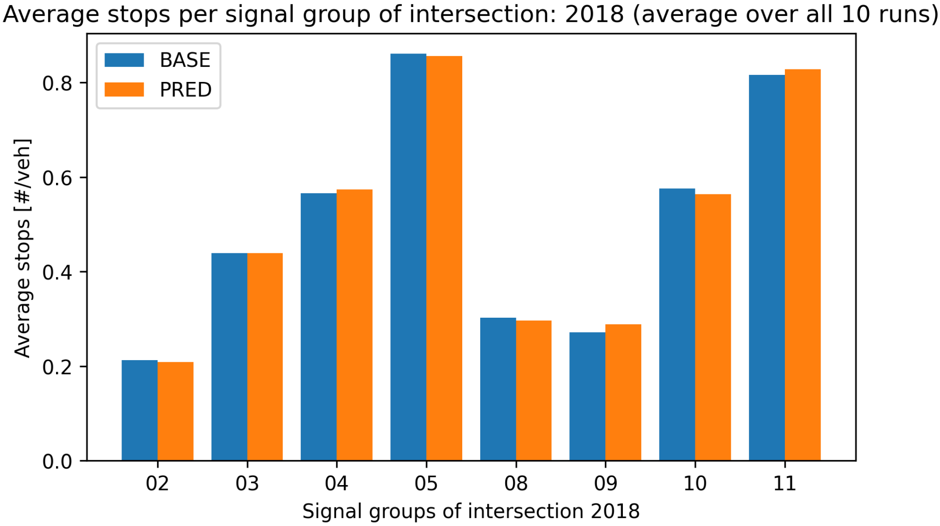

Figure C.15: Reference (BASE) versus first predicted (PRED) average stops per signal group of intersection 2018, average (over all runs)

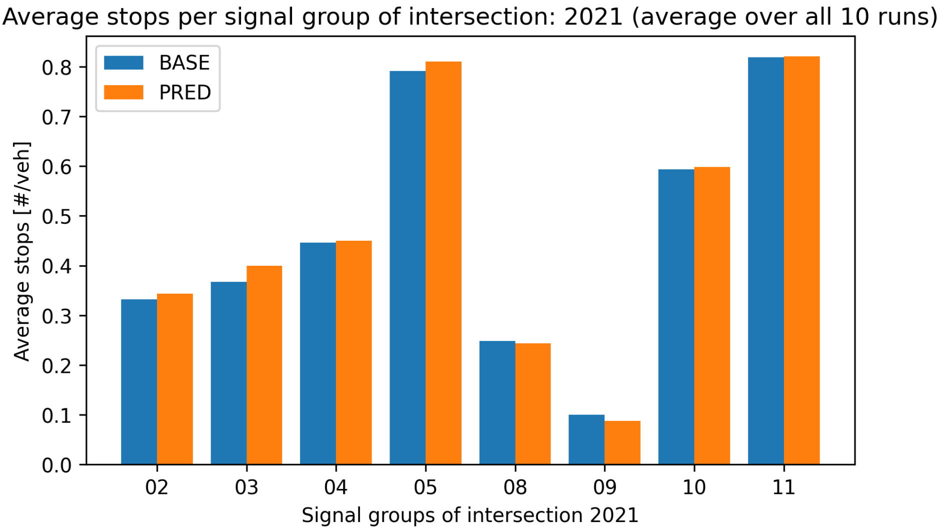

Figure C.16: Reference (BASE) versus first predicted (PRED) average stops per signal group of intersection 2021, average (over all runs)

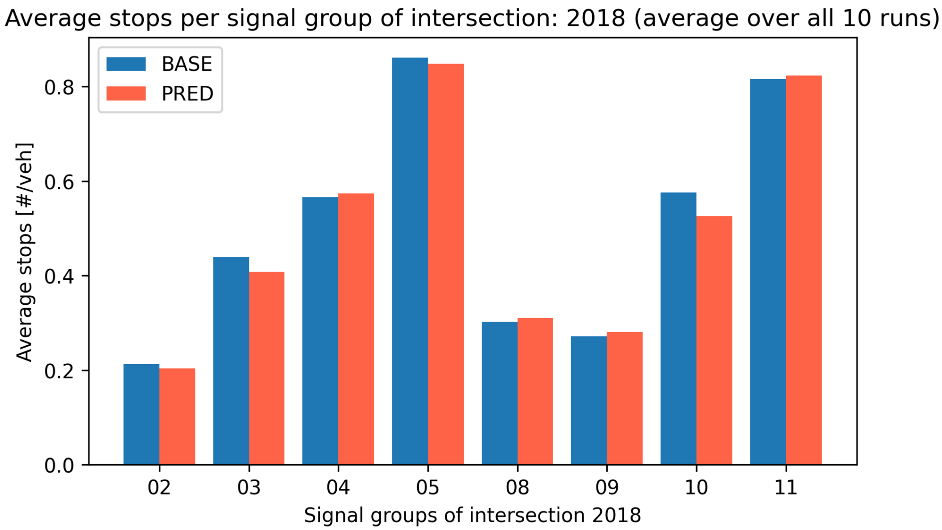

Figure C.17: Reference (BASE) versus second predicted (PRED) average stops per signal group of intersection 2018, average (over all runs)

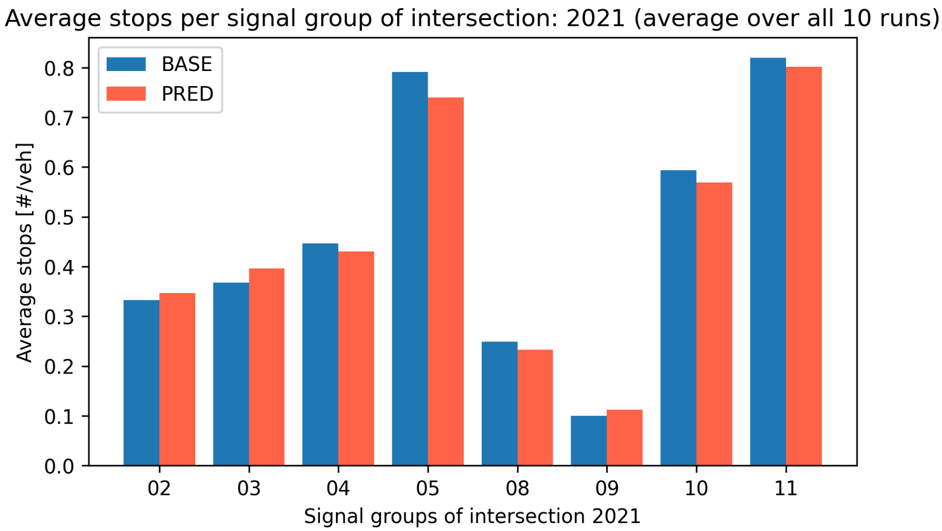

Figure C.18: Reference (BASE) versus second predicted (PRED) average stops per signal group of intersection 2021, average (over all runs)

Table C.2: Overview of simulation result of traffic demand predictions for the whole network, delay per run.

Run nr. [#]	**BASE Avg. Veh. Delay (s)**	**PRED1 Avg. Veh. Delay (s)**	**PRED2 Avg. Veh. Delay (s)**
Run 1	17.22	17.19	17.01
Run 2	17.53	16.88	16.74
Run 3	17.21	17.52	16.86
Run 4	17.48	17.62	17.71
Run 5	17.66	17.26	17.09
Run 6	17.68	17.35	17.01
Run 7	17.13	17.25	17.17
Run 8	17.52	17.15	17.37
Run 9	16.74	16.57	16.85
Run 10	17.03	17.54	17.59