Influence of Intergreen Times on the Capacity of Signalised Intersections

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Abstract

The quality of road traffic in urban networks is determined by the bottlenecks of the network, which are intersections in most cases. Traffic signals can provide high safety and sufficient capacity, particularly if conflicting streams have high volumes and the grade-separated junctions are not feasible. The signal program has to ensure that different movements do not use the same areas inside of the intersection at the same time. This is achieved by assigning different signal groups to conflicting movements and giving the right of way (green time) to these signal groups only subsequently.

The critical moments are the signal change intervals during which one signal group (or stage) loses its right of way, while another receives it. These intervals have to be long enough to make sure that all clearing vehicles left the conflict areas before any entering vehicles arrive there. Because these intervals are framed by the ending of the green interval of one stage and the beginning of green of the next, they are called *intergreen time*.

While the principle of intergreen times is as old as traffic signals are, the question for the determination of the duration of the intergreen times is still debated. A review of the international literature related to intergreen times shows the issues at hand. While it is apparent that too short intergreen times lead particularly to more right angle collisions, too long intergreen times can deteriorate the acceptance, which not only leads to a decreased capacity, but to safety problems, too.

Conspicuously, the parameters used to calculate intergreen times around the world vary more than differences in the characteristics of the traffic flow can account for. Particularly crossing times and entering times are treated quite differently. Intergreen times are still not based on a sound safety model which accounts for the random character of traffic flow. In this light, it seems inappropriate to justify any capacity reductions connected to intergreen times with a perceived safety increase. The future safety related research can be guided towards areas promising not only safety improvements, but an increase in capacity at the same time by scrutinising the capacity impacts of intergreen times.

A major gap in the research has to be seen in the insufficient knowledge of the exact influence of intergreen times on the capacity. Intergreen times are taken generally as lost times, while they are in fact partly used by vehicles to cross the intersection. The duration of signal change intervals and the effective capacity impacts depend on the intersection layout, the signal program, and the stage settings. This influence is not taken into account to full extent so far. The presented research provides the methodology to close this gap.

Because empirical research on signal change intervals faces major difficulties due to the manifold influences on the driver behaviour, a sound theoretical analysis of all processes connected with intergreen times is crucial. Consequently the emphasis in this research has been placed on such a comprehensive theoretical analysis, which leads to a transparent and flexible model to calculate the capacity of signalised intersections with reference to intergreen times. Empirical data has been gathered at seven urban intersections in Germany using video observations and speed measurements to validate the applicability of the model and obtain results on the quantitative capacity impacts of intergreen times.

The empirical research shows that effective green times at the surveyed intersections are in fact greater than signalled green times. At an example intersection analysed as part of this research the effective capacity is about 5 % greater than the capacity calculated with the effective saturation headways and the signalled green times. It is 7 % greater than the capacity according to the German Highway Capacity Manual (HBS). The U.S. Highway Capacity Manual (HCM), which is calibrated for traffic conditions in the United States, leads to an even lower capacity.

Intergreen times based on the prevailing calculation procedures are commonly longer than theoretically required, because

- the parameter values used in these procedures differ from the theoretically needed ones,
- certain parameters are not considered at all in the prevailing procedures (e.g. the entering time),
- the decisive conflict for a stage or signal group sequence does not always occur, and
- safety margins are added.

The capacity improvement potential due to minimised intergreen times was quantified based on the empirical data. While the quantitative results of the model application are based on a number of simplifications due to survey constraints, they nevertheless give a good indication on the improvement potential in general. The findings of the model application can be summarised as follows:

- Conflicts leading to very long intergreen times are commonly of low relevance for the traffic flow (turning traffic, bicycles). The difference between intergreen times for these conflicts and the effectively occurring conflicts are termed *conflict difference times*. Most of the improvement potential (up to 50 %) stems from these conflict difference times.
- Particularly under saturated conditions and at non-coordinated approaches, significant entering times can be observed. Neglecting them consequently leads to notable capacity reductions. About a third of the capacity improvement potential stems from this fact.
- While certain parameters vary significantly among different situations, their variation is small at a specific intersection. It can be concluded that it is worthwhile to analyse the influencing factors and in this way be able to predict these parameters more precisely than so far. This would reduce the requirement for great safety margins. Crossing times and clearance speeds have to be highlighted in this context.
- The variation of certain parameters can be reduced by a sensible signal program and intersection layout. Low variation needs small safety margins, which results in capacity improvements. By indicating the impending signal change from red to green, for instance, start-up lost times can be reduced. Furthermore, the interrelation of yellow time and crossing time of clearing vehicles should be further researched.

The achievements of the presented research can be summarised by

- providing a comprehensive description of the traffic flow during signal change intervals,
- providing a transparent and flexible capacity model to determine the effective capacity and the improvement potential of signalised intersections with reference to intergreen times, and
- highlighting aspects of intergreen times which lead to significant capacity reductions while either no safety improvement can be seen or it remains vague and unproven.

This research not only presents a comprehensive analysis of the reasons for capacity reductions caused by intergreen times, it, furthermore, gives a first impression on the magnitude of the improvement potential. It concludes with recommendations, how this potential may be realised, and what further research is needed to achieve this aim.

Zusammenfassung

Die Qualität des straßengebundenen Stadtverkehrs wird durch Engstellen bestimmt, welche in aller Regel die Knotenpunkte sind. Lichtsignalanlagen können hohe Sicherheit und ausreichende Qualität sicherstellen, insbesondere wenn die Verkehrsstärken sehr groß sind und planfreie Kreuzungen nicht umsetzbar sind. Das Signalprogramm muss sicherstellen, dass unterschiedliche Fahrzeugströme die gleichen Flächen im Knotenpunkt nicht zur gleichen Zeit benutzen. Dies wird erreicht, indem Ströme, die miteinander im Konflikt stehen, unterschiedlichen Signalgruppen zugewiesen werden, denen nacheinander die Freigabe erteilt wird.

Die kritischen Momente im Verkehrsablauf sind die Phasenübergänge zwischen dem Ende der Freigabe einer Signalgruppe (oder Phase) und dem Beginn der Freigabe einer anderen. Diese Intervalle müssen lang genug sein, um den räumenden Fahrzeugen das Verlassen aller Konfliktflächen zu ermöglichen, bevor einfahrende Fahrzeuge dort ankommen. Da das Intervall zwischen dem Ende der Freigabe einer Signalgruppe und dem Beginn einer anderen liegt, wird es *Zwischenzeit* genannt.

Während das Prinzip der Zwischenzeiten so alt ist wie die Lichtsignalanlage, wird die Frage nach der Bestimmung der Dauer immernoch diskutiert. Die Analyse der internationalen Fachliteratur mit Bezug zu Zwischenzeiten offenbart die ungelösten Probleme. Es ist offensichtlich, dass zu kurze Zwischenzeiten insbesondere zu Einbiegen/Kreuzen-Unfällen führen. Zu lange Zwischenzeiten gehen jedoch mit mangelnder Akzeptanz einher, was nicht nur die Kapazität mindert, sondern auch Sicherheitsprobleme verursacht.

Auffälligerweise variieren die Parameter, die in den verschiedenen Ländern benutzt werden, um Zwischenzeiten zu berechnen, mehr, als Unterschiede im Verkehrsfluss erklären können. Insbesondere Einfahrzeiten und Überfahrzeiten werden sehr unterschiedlich gehandhabt. Zwischenzeiten basieren nach wie vor nicht auf einem ausgereiften Modell zur Abschätzung der Sicherheit, das den Zufallscharakter des Verkehrsflusses berücksichtigt. In diesem Licht betrachtet erscheint es unangemessen, jegliche Kapazitätseinbußen durch Zwischenzeiten mit scheinbaren Sicherheitsgewinnen zu rechtfertigen. Die Sicherheitsforschung kann durch eine genaue Analyse der durch Zwischenzeiten verursachten Kapazitätseinflüsse in eine Richtung gelenkt werden, die nicht nur Sicherheitsgewinne, sondern auch eine Erhöhung der Kapazität verspricht.

Eine wesentliche Forschungslücke besteht in der mangelhaften Kenntnis der genauen Kapazitätseinflüsse von Zwischenzeiten. Zwischenzeiten werden grundsätzlich als Verlustzeiten betrachtet, während sie in der Realität zum Teil von Fahrzeugen zum Einfahren in den Knotenpunkt genutzt werden. Die Phasenübergangszeiten und die effektive Kapazität hängen von der Knotenpunktgeometrie, dem Signalprogramm sowie der Phaseneinteilung und Phasenfolge ab. Diese Einflüsse werden bislang nur unzureichend berücksichtigt. Die vorliegende Arbeit stellt die Methodik bereit, um diese Lücke zu schließen.

Eine empirische Betrachtung der Vorgänge während der Phasenübergänge muss zahlreiche Hindernisse meistern, da vielfältige Einflüsse das Fahrverhalten bestimmten. Eine gründliche theoretische Analyse ist unabdingbare Voraussetzung für eine solche Betrachtung. Der Schwerpunkt der vorliegenden Arbeit liegt deshalb auf einer solchen theoretischen Analyse, die in ein transparentes und flexibles Modell mündet, um die Kapazität signalgeregelter Knotenpunkte in Hinblick auf die Rolle der Zwischenzeiten zu berechnen. An sieben innerstädtischen Knotenpunkten in Deutschland wurden empirische Daten mittels Videobeobachtung und Geschwindigkeitsmessung erhoben, um die Anwendbarkeit des Modells zu belegen und um quantitative Aussagen zum Einfluss der Zwischenzeiten auf die Kapazität treffen zu können.

Die empirische Analyse zeigt, dass die effektive Freigabezeit an den untersuchten Knotenpunkten tatsächlich größer ist als die signalisierte. An einem für diese Arbeit betrachteten Beispielknotenpunkt liegt die effektive Kapazität um 5 % über der mit den Sättigungsverkehrsstärken und der signalisierten Freigabezeit berechneten Kapazität. Sie liegt sogar 7 % über der Kapazität nach dem Handbuch für die Bemessung von Straßenverkehrsanlagen (HBS). Das für amerikanische Verhältnisse geeichte Highway Capacity Manual (HCM) schlägt eine noch niedrige Kapazität vor.

Die nach den vorherrschenden Verfahren berechneten Zwischenzeiten liegen in der Regel über den theoretisch erforderlichen, weil

- die in diesen Verfahren gewählten Parameterwerte von den theoretisch erforderlichen abweichen,
- einige Parameter in diesen Verfahren nicht berücksichtigt werden (z. B. Einfahrzeiten),
- der maßgebende Konfliktfall für eine Phasen- oder Signalgruppenfolge nicht immer auftritt, und
- Sicherheitzuschläge eingerechnet werden.

Das Verbesserungspotenzial der Kapazität durch minimierte Zwischenzeiten wurde mit Hilfe von empirischen Daten quantifiziert. Die quantitativen Ergebnisse bieten einen guten Eindruck vom generellen Verbesserungspotenzial, obwohl sie auf einigen, den eingeschränkten Möglichkeiten im Rahmen der Arbeit geschuldeten Vereinfachungen beruhen. Die Ergebnisse können wie folgt zusammengefasst werden:

- Konfliktfälle, die zu besonders langen Zwischenzeiten führen (Abbiegeströme, Fahrradverkehr), haben oft nur eine untergeordnete Bedeutung für den Verkehrsfluss. Der Unterschied zwischen den für diese Fälle berechneten und den für die tatsächlich auftretenden Konfliktfälle berechneten Zwischenzeiten wird hier als *Konfliktdifferenzzeit* bezeichnet. Der Hauptteil des Verbesserungspotenzials (bis zu 50 %) rührt von diesen Konfliktdifferenzzeiten her.
- Besonders an ausgelasteten und unkoordinierten Zufahrten können nennenswerte Einfahrzeiten beobachtet werden. Ihre Vernachlässigung führt zu signifikanten Kapazitätseinbußen. Etwa ein Drittel des Verbesserungspotenzials ist durch diese Tatsache begründet.
- Einige Parameter variieren stark zwischen verschiedenen Knotenpunkten. Ihre Streuung an einem einzigen Knotenpunkt kann jedoch gering sein. Es ist also sinnvoll, die Einflussgrößen auf diese Parameter zu ermitteln, um die Parameterausprägung besser als bisher vorhersagen zu können. Dadurch können Sicherheitszuschläge verringert werden. Überfahrzeit und Räumgeschwindigkeit sind hiervon besonders betroffen.
- Die Streuung bestimmter Parameter kann durch ein sinnvolles Signalprogramm und eine gute Knotenpunktgestaltung verringert werden. Eine geringe Streuung wiederrum erfordert nur geringe Sicherheitszuschläge, wodurch die Kapazität erhöht wird. Durch die Ankündigung des Freigabebeginns können beispielsweise die Anfahrverluste reduziert werden. Darüber hinaus sollte der Zusammenhang zwischen Gelbzeit und Überfahrzeit genauer untersucht werden.

Die Bedeutung der vorliegenden Arbeit wird mit folgenden Punkten zusammengefasst:

- Bereitstellung einer umfassenden Darstellung des Verkehrsflusses während des Phasenwechsels.
- Bereitstellung eines transparenten und flexiblen Modells, um die effektive Kapazität zu bestimmen und das Verbesserungspotenzial der Kapazität signalgeregelter Knotenpunkte in Hinblick auf die Zwischenzeiten abzuschätzen.
- Ermittlung von Aspekten der Zwischenzeiten, die zu signifikanten Kapazitätseinbußen führen, obwohl kein klarer Sicherheitsgewinn erkennbar ist.

Die vorliegende Arbeit stellt nicht nur eine umfassende Analyse der Gründe für Kapazitätsverminderungen durch Zwischenzeiten dar, sie gibt auch erste Anhaltspunkte für die Größenordnung des Verbesserungspotenzials. Die Arbeit schließt mit Empfehlungen, wie dieses Verbesserungspotenzial genutzt werden könnte und welche weitere Forschung erforderlich ist, um dieses Ziel zu erreichen.

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Contents

1 Introduction

1.1 Aims and outline

1.1.1 Research motivation

Signalised intersections play an integral role in urban traffic. They determine decisively the quality of traffic flow in urban networks. They are deployed to divide conflicting traffic streams from each other. In this way, all traffic streams receive right of way successively, but (turning traffic usually being the exception) unimpeded by other streams.

A crucial part for both safety and capacity are the intervals between the different stages of a signal program during which some streams loose the right of way, others receive it. During this period, the conflict areas in the intersection are used by conflicting streams after each other. The signal program has to ensure that all areas in the intersection are cleared before conflicting vehicles reach them. This is achieved by *intergreen times* during the signal change intervals. As the name suggests, intergreen times are intervals between the ending of the green time of one stage and the beginning of green of a conflicting stage.

On first glance it seems that intergreen times just have to be long enough to achieve high safety. However, long intergreen times have two side effects. Firstly, during intergreen all respective streams do not receive right of way. Thus, intergreen times are intervals not used for improving throughput of vehicles and apparently reduce the capacity of the intersection. This is one underlying argument if multi-stage operations of traffic signals with more than two stages are avoided for capacity reasons. Secondly, if intergreen times become too long, the acceptance of clearance and stop signals suffers, which deteriorates the safety.

With the prevailing capacity equations (e.g. from TRB 2000; FGSV 2001)[1] the influence of intergreen times on the capacity can easily be calculated. However, a look at the real situation reveals, that traffic is not as deterministic as frequently postulated. As the saturation flow depends on various factors, so does the effectively used green time. One important factor is the signal program, namely the transition times and intergreen times. The existing procedures to dimension green times are manifold. While the traffic flow follows the same principles in all industrialised countries, and even traffic signals, with all respect to the differences in detail, are based on similar precepts in different countries, intergreen times are still researched with a strong national focus. A motivation for this research was to broaden the view, embrace similarities and differences of approaches to intergreen times, and scrutinise the capacity impacts of intergreen times systematically.

But even when knowing the exact influence of intergreen times on the capacity, another question raised in this thesis remains: How much can the capacity be improved by optimising intergreen times? If it would be possible to predict precisely the traffic flow at intersections, including every single vehicle and its trajectory, the intergreen times could be reduced dynamically to a minimum required value for each change interval. In this way the capacity could be maximised without compromising the safety.

Though this is only visionary theory so far, a capacity based on this assumption reveals the full potential of a signalised intersection with respect to signal change intervals. Furthermore, through a

1 TRB: Transportation Research Board of the National Academies, United States of America; FGSV: Forschungsgesellschaft für Straßen- und Verkehrswesen (Road and Transportation Research Association), Germany

closer look at the reasons for the difference between the effective and this maximum capacity safety related research can be focused on parameters which are decisive for the capacity impacts of intergreen times. Moreover, ways to improve the performance of an intersection without decreasing safety may be discerned.

1.1.2 Aims and delimitations

Based on the motivation for this thesis, the following aims can be derived:

1. Consolidate the research on intergreen times with respect to capacity issues.
2. Develop a generally applicable methodology to calculate the quantitative influence of intergreen times on the capacity of signalised intersections.
3. Determine the improvement potential of the capacity of signalised intersections by optimised intergreen times.
4. Point out capacity improving measures, which do not derogate the safety.
5. Find promising areas of safety related research with respect to intergreen times to maximise safety and capacity improvements.

The main focus of this research is the systematic development of a model to calculate both the effective capacity of signalised intersections, and to determine the maximum improvement potential of the capacity by optimised intergreen times.

As far as capacity is concerned, vehicles are of primary interest at signalised intersections in industrialised countries. Two aspects will be mostly neglected here: the influence of pedestrians, and the influence of permitted streams[2]. It will remain the task for further research to extend the model to pedestrians and permitted traffic streams.

The applicability of the model has to be proven by exemplary data. This data has to be the basis for quantitative statements on the formulated research aims. While the model has to be applicable for change intervals in any country, the application of the model as described here is based upon data gathered at German urban intersections. To apply the model to German conditions bears the advantage of illustrating one of the most detailed methods of intergreen time determination.

It has to be stressed that safety aspects are only touched upon where they influence the model. The safety levels achievable by different intergreen time determination procedures, or the consequences of changed intergreen times for safety, are excluded from the research. Whether the calculated capacity improvements can be realised or not will be subject to safety related research. However, based upon the results of this thesis it will be possible, to focus this research on areas, which promise both safety and capacity improvements.

1.1.3 Methodology and outline

The research is realised in four steps:

1. literature review
2. theoretical analysis and model development
3. exemplary model application

[2] Permitted streams commonly have to wait inside of the intersection. Their behaviour depends on the gaps in the privileged streams. This gap acceptance behaviour is not analysed here.

4. derivation of conclusions from the first three steps

The starting point for the research is a thorough international literature review with a focus on English and German publications (**Chapter 2**). The review starts with an introduction of the procedures to determine intergreen times in selected countries. The capacity related research is, furthermore, scrutinised. Most intergreen related research deals with safety aspects, but analyses the traffic flow, which is also of importance for the capacity analysis. The literature review is in this way both a sound basis for a detailed analysis of the capacity related intergreen time issues, and a presentation of the state of the art.

The literature review together with observations of the traffic flow during the change intervals is the basis for the further research. All capacity related aspects of intergreen times are thoroughly analysed (**Chapter 3**). Firstly, the parameters leading to the effective capacity are explored, secondly the optimisation potential of intergreen times, expressed by intergreen time differences, is scrutinised. All processes during the signal change intervals are investigated separately for their capacity impacts, and eventually regarded in interaction with each other.

This analysis leads to the development of a capacity model (**Chapter 4**). This model enables, in the first place, the calculation of effective and maximum capacity on the basis of detailed empirical data. The model can, however, be calibrated for defined situations. The identification of influencing factors on the model parameters, which will lead to clusters of intersections with comparable characteristics concerning individual model parameters, are the basis for the, thus, generalised model. The parameters for which the model has been calibrated do not have to be obtained specifically at the survey intersection. While the methodology for this calibration process is outlined, the clustering itself and the calibration of the model for the different clusters requires more extensive surveys than could be conducted for the presented project. The focus here is, therefore, directed at the model development.

Empirical data was gathered to apply the model exemplarily and get further insight into the traffic flow during change intervals (**Chapter 5**). The surveys have been conducted at urban intersections in the City of Darmstadt in Germany. Video observations, speed measurements, and an evaluation of signal program parameters have been conducted. The survey methodology is explained in detail. Data has been collected for all model parameters. The results are presented divided into these parameters, before the model is applied to an exemplary intersection. The measurement error involved and the achieved accuracy is laid out.

Finally, conclusions are drawn in **Chapter 6** with respect to the research aims. The conclusions are divided into the two aspects of effective capacity and improvement potential of intergreen times with respect to the maximum capacity. Recommendations are given for future empirical research.

1.2 Terminology and definitions

1.2.1 Introduction

The terminology related to intergreen times is based on the situations, common procedures, and laws in the respective countries. Research trying to embrace all those situations, procedures, and laws has to face the differences in the terminology. Some terms defined and common in one country may not exist in other countries. Furthermore, variables and abbreviations based on a language different from English are hard to understand and memorise for English readers.

To avoid unnecessary confusion by the use of abbreviations common in one country but unknown or differently used in others, here a new systematic terminology is used. It is based upon symbols used in Germany and the United States, but systematically extended. The basic structure is explained further down. On page 123 a table with all variables and symbols used with their German and U.S. equivalents is shown for reference. Variables and terms are usually introduced on their first appearance in the text. The most important terms, particularly ones used in different ways over the world, are defined below.

1.2.2 Definition of terms

Intersection is used for a junction of at least two roads with any number of legs.

Intergreen time also called *intergreen interval* or just *intergreen*, or *change and clearance interval* is the time between the end of green time of one stage and the beginning of green time of a subsequent conflicting stage. It commonly comprises transition times (i.e. yellow, yellow-and-red) and an all-red interval ("all"-red relating to beginning and ending stage only).

Stage is a state where the signalisation remains unchanged (AE also *phase*).

Stage change (signal change interval, vehicle change interval) is the time between the end of one stage and the beginning of the next.

Signal group is a group of signals where all signals show the same indication at all times.

Yellow is used instead of "amber" for the transition signal between green and red.

Post-encroachment time is used here as the time headway between the last clearing vehicle and the first entering vehicle at a conflict point, regardless of the movement.

Stream denotes a traffic movement with unique origin and destination at an intersection. Twelve streams can be present. Commonly they are numbered starting with the right turning vehicles on the nothern approach clockwise.

Movement sequence is the sequence of a certain clearing stream and specified vehicle type, followed by a specific entering stream (e.g. motorised north-bound through vehicle clears, west-bound through vehicle enters).

Conflict is a non-compatible movement sequence (i.e. a movement sequence that will lead to a collision, if no post-encroachment time occurs).

Conflict area is used here for the intersection of trajectories of conflicting movements with their real dimensions. The point used to determine entering or clearance distances is the conflict *point*.

1.2.3 Structure of variable identifiers

To enable an easy recognition of variables, the variable letter defines the unit:

t are times or intervals (usually in seconds)

l are lengths or distances (in metres)

v are speeds (in either meters per second or kilometres per hour)

q are vehicle volumes (in vehicles per hour)

C are capacities (in vehicles per hour)

The index defines the exact parameter. Variables related to entering vehicles have index "e", variables related to clearing vehicles have index "cl". A leading Greek capital Delta (Δ) denotes a difference (commonly between effective and assumed values).

2 State of the art

2.1 Introduction

This section paraphrases the state of the art in intergreen time determination and research. After a brief review of the definition and history of intergreen times, the different concepts of their determination over the world are explained (Section 2.2). This country specific review (Section 2.2) is not meant to be a complete overview of all methods applied world wide. It is meant as a short introduction into the issues at hand and the range of the methods used.

The past research into the capacity impacts of intergreen times as the main focus of the project described here is reviewed (Section 2.3). However, the capacity impacts of intergreen times cannot be regarded without considering a number of closely connected issues, namely the safety impacts of intergreen times and the yellow time dilemma (Section 2.4). The random character of traffic flow will be a major subject matter to analyse (Section 2.5). Furthermore, some special aspects in connection with intergreen times like the protection of turning vehicle streams (lead-/lag-time) received considerable attention in the past research (Section 2.6). Therefore they are touched upon here. The section closes with the conclusions which can be drawn from the state-of-the-art in respect to the capacity impacts of intergreen times (Section 2.7).

For the high degree of detail and sophistication of the German method to calculate intergreen times, and due to the surveys presented in this thesis, which are conducted in Germany, this chapter starts with the German perspective, explaining the German method in detail. The literature review related to other countries gives the differences to the German situation. In this way, commonalities and differences can easily be highlighted.

2.2 History, definition, and determination of intergreen times

2.2.1 Introduction

Intergreen times are a part of the signal program which aim at avoiding conflicts between vehicles during the change of stages. The introduction of intergreen times dates back to the widespread installation of traffic signals. In the international scientific community, the *inter-green* time is defined as the time between the end of green of one stream and the onset of green of a conflicting stream. The importance of intergreen times arises out of its significance for the safety of signalised intersections and its influence on their capacity.

Despite the consistent aim of intergreen times, the method to calculate them varies around the world. Intergreen times still attract the attention of the research community, e.g. in Germany (Boltze et al. 2006), in the USA (Click 2008; Tarnoff 2004), and Japan (Tang and Nakamura 2007a). State-of-the-art in the intergreen time calculation follows different conventions. Arasan et al. (2006) distinguished three methodologies:

- simple calculation, e.g. according to the Institute of Transportation Engineers (ITE 1999), followed widely in the U.S., and recommended by Jakob (1982)
- complex calculation according to the German Guidelines for Traffic Signals (FGSV 1992), followed in Germany and similarly in some Scandinavian countries

- probabilistic approach, e.g. proposed by EASA (1993), and advertised similarly in consequence of this research

The following sections introduce the calculation procedures and the research thereon in a number of industrialised countries to highlight differences and commonalities.

2.2.2 Germany

Background

In Germany, the first Guidelines for Traffic Signals (FGSV 1966, "Richtlinien für Entwurf, Bau und Betrieb von Lichtsignalanlagen im Straßenverkehr"/"Standards for design, construction, and operation of traffic signals in road traffic") already introduced the concept of intergreen times. The intergreen time is calculated for all possible conflict situations using crossing, clearance and entering times with the respective conflict area as the reference location. Passenger cars, public transport vehicles, bicycles and pedestrians are separately taken into account.

In the signal program, a yellow time for the traffic streams loosing right-of-way at the end of a signal stage, a yellow-and-red time for the traffic streams gaining right-of-way, and optionally an all-red time are used for the intergreen interval. This concept remained the same in the different editions of the Guidelines (FGSV 1977, 1981, 1992, 2003, 2010, the latter not being published yet)[3] with modifications of the speeds, crossing times, and reference points.

These Guidelines are approved by the legal bodies and, thus, are the standard for federal authorities with respect to traffic signal control. Deviations from the recommendations in the guidelines have to be thoroughly justified.

It should be noted, that in Germany the signal heads are mounted on the near side of the intersection only, an optional green arrow for lead-/lag-times being the exception of this rule. Signal programs are calculated to the full second. Intergreen times are, hence, rounded up to full seconds. The calculated values may be adjusted for special constellations. Railway or tram crossings (reduction of the intergreen times) and left turning traffic (prolongation of the intergreen times) are the most common special situations.

Yellow times are also laid down in the Guidelines for Traffic Signals. They are independent from the intergreen times (and vice versa) and uniform for the three classes of speed limits (3, 4, or 5 s for $\leq 50\,\mathrm{km/h}$, $60\,\mathrm{km/h}$, and $70\,\mathrm{km/h}$ respectively; speed limits are always reduced to at least $70\,\mathrm{km/h}$ at traffic signals).

Intergreen time calculation today

According to the German Guidelines for Traffic Signals (RiLSA), intergreen times are calculated by summing up the time needed by the clearing vehicle to cross the stop line and clear the conflict point and subtract the time necessary for the entering vehicle to reach the conflict point (Eq. 1).

$$t_{ig} = t_{cr} + t_{cl} - t_e \tag{1}$$

with	t_{ig}	intergreen time (Zwischenzeit, t_z)	(s)
	t_{cr}	crossing time (Überfahrzeit, $t_{ü}$)	(s)
	t_{cl}	clearance time (Räumzeit, t_r)	(s)
	t_e	entering time (Einfahrzeit, t_e)	(s)

[3] The title of the guidelines changed to "Richtlinien für Lichtsignalanlagen" (RiLSA).

Clearance time and entering time are calculated with the respective distances and speeds (Eq. 2 and Eq. 3).

$$t_e = \frac{l_e}{v_e} \quad (2)$$

$$t_{cl} = \frac{l_{cl} + l_{veh}}{v_{cl}} \quad (3)$$

with	l_e/l_{cl}	entering/clearance distance	(m)
	l_{veh}	vehicle length	(m)
	v_e/v_{cl}	entering/clearance speed	($^m/_s$)

The most recent approved guidelines (FGSV 1992, a revision being under way) distinguish six cases:

1. through traffic is clearing
2. turning traffic is clearing
3. public transport vehicles are clearing (with a mandatory stop before clearing)
4. public transport vehicles are clearing (without a mandatory stop before clearing)
5. bicycles are clearing
6. pedestrians are clearing

The cases result in different crossing times (0 to 5 s), clearance speeds (1 to 10 $^m/_s$), and vehicle lengths (0 to 15 m) regardless of local speed limits, grades, or the yellow time. For all possible conflicts, intergreen times have to be calculated with the values corresponding to the respective case. The longest intergreen time of each signal group combination or change of stages (depending on the control regime) is used in the signal program.

Whether or not a conflict has to be considered depends solely on the *possibility* of the presence of the respective vehicles or streams under legal circumstances. Since bicycles are treated as normal vehicles in the German Vehicle Code (StVO), they have to be taken into account where they are not forced to use separate crossings or signals. Intergreen times have to be calculated for all combinations of clearing and entering vehicles, as long as they have a conflict point and are not permitted simultaneously in the intersection (e.g. permitted left turning traffic) or considered compatible (e.g. right turning bicycles and through vehicles). Distances are calculated with the intersection of the respective lane centre lines as the conflict point. Only for vehicles using the same exit, the intersection of the lane border lines is taken as the conflict point. For pedestrians the whole conflict area (being the marked crosswalk) is used for the clearance distance determination. Entering times, i.e. the time needed by entering vehicles to reach the conflict point, are considered. However, a running start and entering with a constant speed of 40 $^{km}/_h$ is assumed.

Criticism

The issues discussed in Germany in the context of intergreen time calculation can broadly divided into two areas:

- the procedure itself
- the parameters used in the equations

It should be noted that in Germany, as opposed to, for instance, the United States, a uniform yellow time is used. The intergreen time calculation takes this yellow time into account through a crossing time term, but is, as opposed to the yellow time, calculated individually for every intersection. Yellow time and intergreen time are therefore dealt with independently.

As for the procedure itself, most traffic professionals agree that, due to its sophistication, high safety is achieved while keeping intergreen times as low as possible (and thus capacity high). However, JAKOB (1980) highlighted, that despite the high accuracy suggested by the German method of intergreen calculation, the random character of traffic flow on intersections is neglected. The traffic flow is influenced by weather, vehicle properties, driver behaviour, traffic situation, trip purposes etc., which are not reflected by a calculation using fixed values. Furthermore, the rounding up of the calculated values to the full second is in no way related to the conflict situation and, thus, the risk at hand.

He drew two conclusions: firstly, the high sophistication only feigns high accuracy; secondly, the achieved higher precision does not lead to higher safety. While one of his concerns, the high effort involved in the calculation procedure (the number of conflicts to be considered easily reaches 100 at a standard intersection), diminishes in importance in times of computer aided signal program development, his safety related concerns are still valid.

Anyhow, the procedure is well established, easy to understand, and transparent. For legal and responsibility issues alone, transport professionals recoil from questioning it. This discussion already shows the dilemma of finding *safe* and yet *efficient* intergreen times. The calculation method will always be a compromise between the two. The discussion, however, also shows that safety and efficiency don't have to be contradictions.

This issue directly leads to the question for the parameters to be used.

GLEUE (1974) pointed out the necessity of defining a "standard" driving behaviour as the basis for the intergreen calculation. However, he also stated that a minimum clearance speed exceeded in all but a certain low percentage of cycles is difficult to determine. Consequently, the clearance speeds recommended in the guidelines have been determined by consensus among transport professionals, as have been the vehicle lengths used today. A reason to use the comparably high value of $10\,\mathrm{m/s}$ as the clearance speed for through traffic was the capacity impact of slower speeds for large intersections with long clearance distances.

While today the entering time is always taken into account, in the old editions of the German Guidelines for Traffic Signals (RiLSA), the entering time was recommended only where it appeared safe – without a clear definition what "safe" would mean. As opposed to the compromise concerning the clearance speed, it was agreed for safety reasons, to consider the fastest possible entering vehicle (GLEUE 1977). Regardless of the research by GLEUE (1973a), showing the negligible low probability of this case, and supported by SCHNABEL (1976), this procedure is still followed today. While in case of entering behaviour a very unlikely situation is taken into account, the clearing behaviour is based on average speeds, as will be proven later on.

While the intergreen time calculation method remained the same over the decades, some parameters have been adjusted in the different editions from 1966 until 1992 (DUNKER 1993), the latter one being still in effect today. The focus shifted from a distinct emphasis on individual motorised traffic towards the consideration of all travellers (HOFFMANN 1992). While in the edition of 1992 separate chapters are dedicated to the needs of pedestrians, bicycles, and public transport, the latest edition of the RiLSA (FGSV 2010) will lead to a more intergrated approach to signal control. Furthermore, the trend towards sophisticated model based traffic actuation will be reflected.

2.2.3 United States of America

Background

The Manual on Uniform Traffic Control Devices (MUTCD 2003) as the only nation-wide legally binding standard of the United States concerning intergreen times gives only a very rough indication for the "*vehicle change intervals*", i.e. the definition of the yellow time and the all-red time. No calculation method is proposed. Even the choice whether to use an all-red time or not is left to the engineer. MUTCD (2003) conforms to the Uniform Vehicle Code. State laws which expand upon MUTCD (2003) differ, particularly regarding the circumstances under which vehicles should or should not enter the intersection during the yellow interval.

The Institute of Transportation Engineers tried to fill this gap by a lively discussion of vehicle change intervals (Technical Committee 4A-16 1985; Technical Council Committee 4A-16 1989; Institute of Transportation Engineers 1994). However, since no unanimity could be achieved on the details, only the calculation method was laid down in the Traffic Engineering Handbook (current version: ITE 1999), leaving the choice of the parameter values to the engineer. This calculation method is adopted by the majority of transport professionals (Eccles and McGee 2001). The formula underwent several revisions since its first appearance in 1941. The present form exists since 1965, incorporating the influence of grades since 1982. The changes of the formula and its components went to an average prolongation of the change interval by approximately two seconds over the years (Eccles and McGee 2001).

The ITE method differs mainly in three points from the procedure according to the German Guidelines for Traffic Signals (RiLSA):

- yellow time and intergreen time are taken account for in the same equation;
  in Germany the yellow time is prescribed (depending on the speed limit only)
- no entering time is considered;
  Williams (1977) proposed to consider the entering time, but neglect running starts; Lin and Vijakumar (1988) extended this proposal by a approach taking decelerating and accelerating behaviour into account; they empirically derived a linear equation for the calculation of the entering time; both proposals haven't been adopted so far by any manuals
- the influence of grade is incorporated;
  Rodegerdts et al. (2004) emphasised again the high relevance of both positive and negative grade of the approach lanes for the clearance times; the German Guidelines for Traffic Signals (RiLSA) only qualitatively advises traffic engineers to take grade into account

Another difference of originally only technical nature, but quite important not only for capacity considerations, is the higher precision of American signal controllers (commonly one significant figure more than in Germany). Furthermore, the determination of clearance distance is not fixed (cf. Eccles and McGee 2001). Intersection width or the distance to the end of the pedestrian crosswalk are proposed instead of more detailed lengths as in the German Guidelines for Traffic Signals (RiLSA).

The necessity of a red clearance interval is still disputed. Koonce et al. (2008) highlight the capacity reductions caused by a red clearance interval while doubting the long term improvement of the safety.

Intersection layout and particularities of signal settings

A big difference to Germany can be seen in the fact that in the U.S. the signal heads are mounted on the opposite side of intersections. Even during the clearance interval drivers can, thus, see the signals. This lead to the discussion in the U.S., whether the yellow time is a clearance interval or not (Bissell and

WARREN 1981). The authors concluded that the clearance interval starts *after the end* of yellow following the permissive yellow rule (see next paragraph).

Federalism

The lack of consistent and detailed guidelines (also due to federalism) leads to quite different intergreen times throughout the country. Attempts to harmonise the intergreen time calculation are of little avail so far. Even the legal basis for intergreen times differs among states: some states follow the permissive yellow rule (recommended by the NATIONAL COMMITTEE ON UNIFORM TRAFFIC LAWS AND ORDINANCES 2000), which allows vehicles to enter the intersections at the end of yellow, others follow the restrictive yellow rule, which requires from drivers to stop during yellow if possible, thus prohibiting a crossing of the intersection during red. While the clearance interval for the permissive yellow rule starts after the end of yellow, the restrictive rule expects at least part of the yellow interval to be used as clearance time. The resulting discussion following the restrictive rule and its consequences for intergreen times has been mentioned before.

Harmonisation initiatives

Analyses which approach to intergreen time calculation and which parameters may be the most promising and useful ones have been carried out. The parameters used in the calculation procedures for yellow time and intergreen time have been discussed by the TECHNICAL COUNCIL COMMITTEE 4A-16. The background for this discussion can be found in safety concerns and the desire for more legal certainty. Influencing factors on driver behaviour have been part of the discussion, namely the acceptance of regulations. The report recommends legal definitions for various aspects of the change interval and a defensible methodology for calculating and evaluating change intervals. The legal basis for the recommendations is the permissive yellow rule. The end of the conflict area is proposed for the determination of clearance distances.

One issue mentioned in many reports is the determination of speeds to be used in the equations. Apart from the speed limit, 15- and 85-percentile speeds are recommended. A distinction between vehicle types is not pondered.

A North Carolina DOT Task Force (CLICK 2008) came to the following conclusions:

- The ITE formula should be the basis for the intergreen time calculation.
- The calculation should be independent of region, traffic stream, and the installation of enforcement devices.
- The yellow time should be 3 s at minimum, all-red time should be at least 1 s, and the intergreen time should be rounded to the nearest 0.1 s.
- Following the ITE recommendation (TECHNICAL COMMITTEE 4A-16 1985) the 85-percentile speed should be used (or alternatively the speed limit).
- No vehicle length should be considered and the clearance distances may be calculated using straight lines (instead of exact trajectories).

These recommendations haven't been adopted widely so far.

The American Association of State Highway and Transportation Officials (AASHTO) deduced deceleration rates and reaction times of drivers, which may be used in the calculation of the intergreen times.

2.2.4 Japan

In former times, in Japan the intergreen times were not calculated, but taken from standard tables. The distance between the opposite stop lines served as the clearance distance (JSTE 1994). Nowadays, the intergreen times are calculated similarly to the ITE procedure. The intergreen time consists of the yellow time and an all-red time. With the revision of the Manual on Traffic Signal Control in 2006 (JSTE 2006) the procedure was refined by introducing conflict points (instead of intersection width) and an entering time. In this way the current procedure follows closely the method in the German Guidelines for Traffic Signals (RiLSA).

However, despite these recommendations by transport professionals, the police, as the responsible authority for traffic signal control, often tries to compensate for safety problems at intersections through a prolongation of intergreen times. This entails very long cycle times and frequently poor quality of the traffic flow (Tang and Nakamura 2007c). The very long cycle times result, as opposed to the intention, in reduced safety as has been highlighted by Suzuki et al. (2004b). The main reason being more risky behaviour of the drivers due to long waiting times.

The problems entailed provoked research into possible ways to base intergreen time settings on a more profound method, most likely resulting in both safer and more efficient signalised intersections (e.g. Tang and Nakamura 2007a).

2.2.5 Switzerland

The determination of intergreen times in Switzerland is quite similar to Germany. It is laid down in the Swiss Standard VSS (1996). The calculation incorporates a crossing time, clearance time, and entering time with different speeds and times for the vehicle types (i.e. bicycles, busses, passenger cars, trams) and pedestrians. In Switzerland, the additional time for entering vehicles to cross the stop line after the onset of green (starting response time) is considered. The intergreen times may be adjusted for special local situations, if justifiable. Distances are determined using conflict points of distinct streams. An overview of additional Swiss Guidelines with relevance for intergreen times can be found in Burnand (1996).

2.2.6 Austria

The Austrian Guidelines and Regulations for Road Construction ("Richtlinien und Vorschriften für den Straßenbau", TFSV 1998) follow the same concept as the German and Swiss guidelines. Crossing time, clearance time, and entering time are used for all possible conflicts with the longest intergreen time becoming relevant. Values are rounded up to the full second. Values no less than four seconds are recommended. The size of the conflict area is considered, as is the special behaviour of trams and cyclists.

Special consideration is taken for signals without indication of transition (green arrow), coordinated signals (extension of intergreen time for running start), and special constellations (extension possible).

In Austria the transition to red is signalled by a flashing green light before yellow. This flashing green time is not considered in the intergreen time calculation, regardless of its influence on the driver behaviour (Köll et al. 2002).

2.2.7 France

The French Interdepartmental Instructions on Road Signs ("Instruction interministérielle sur la signalisation routière", SETRA 2002) define the all-red time as the time for vehicles crossing the stop line during the last second of yellow (last second of green time for pedestrians) to clear the conflict area. Thus, no entering time is considered. Specific values for tramways and adjustments for special constellations are recommended.

2.2.8 The Netherlands

In 1992 the Dutch calculation method for the intergreen times was revised by a team of experts, leading to a new edition of the Guidelines on Clearance Times for Traffic Controllers ("Richtlijn ontruiminstijden verkeersregelinstallaties", CROW 1996). Clearance and entering times are determined for every conflict considering the conflict area dimension (cf. MULLER ET AL. 2004).

2.2.9 Alternative approaches in the research

Alternative approaches to intergreen calculation mainly focus on the yellow time determination. However, the yellow time is commonly part of the intergreen interval. Thus, these approaches are mentioned here.

Never realised was the approach pursued by EASA (1993). He developed a probabilistic model for the intergreen calculation, considering the variability and correlation of approach speed, reaction time, deceleration rate, and vehicle length. His approach focussed on the avoidance of the dilemma zone connected with the yellow time and, thus, a safety increase[4].

ECCLES and MCGEE (2001) reviewed methods that differ from the kinematic model commonly applied for the yellow time determination. They distinguish between three approaches:

- Uniform approach (constant yellow interval):
  While BENIOFF ET AL. (1980) see no proof for the advantages of a uniform yellow interval, FRANTZESKAKIS (1984) supports this idea with a dependancy of the yellow interval on the approach speed (as realised in FGSV 1992).
- Stopping probability method (OLSON and ROTHERY 1962):
  This method estimates the stopping probability of drivers as a function of the distance to the stop line. The minimum yellow change interval is subsequently calculated using this probability, the intersection width, the vehicle length, and the speed. Though their research is very old now, the idea was repeatedly revived (e.g. MAHALEL and ZAIDEL 1986; RAKHA ET AL. 2007).
- Combination of kinematic model with stopping probability method:
  WILLIAMS (1977) proposed to consider the entering time and determine the clearance and entering distances by approximating the conflict area.

GLEUE (1974) discussed the basic parameters needed to determine the "best" intergreen interval (concerning mainly safety). In addition to the parameters already utilised by the prevailing intergreen calculation methods, he came up with some additional parameters:

- time when the first entering vehicle crosses the stop line (start-up lost time, as considered in Switzerland)

[4] The dilemma zone denotes the time during which a driver can neither stop in front of the stop line, nor proceed and cross the stop line before the onset of red

- probability for moving start (vehicles not stopping before crossing the stop line)
- acceleration, deceleration
- distribution of driver behaviour (e.g. slow vehicles)

2.2.10 Comparison and conclusion

The concept of providing intergreen times to ensure a safe traffic flow during signal change intervals consisting of yellow time and all-red time is widely accepted. Intergreen times are always calculated by defining a clearance distance and assuming a clearance speed of the last clearing vehicle to determine the time needed by this vehicle to leave the conflict area.

The main differences can be seen in the following points:

Entering times are not always considered. Sometimes running starts are taken account for, sometimes vehicles accelerating from full stop are determining (with or without start-up lost times).

Grade is not always considered.

Crossing time and yellow time are sometimes equated, sometimes considered separately. Butler (1983) highlighted that slow vehicles need less yellow time, resulting in a clearance interval starting earlier than the onset of red. The same applies to different vehicle types. By distinguishing between crossing time and yellow time, some guidelines take this aspect into account.

Speed is either prescribed in the guidelines, or determined empirically. Average speeds take different vehicle types or streams implicitly into account, but do not provide for the determining conflict. No guideline distinguishes between all vehicle types (including lorries) and movements.

Distances are either taken from intersection dimensions or defined for every conflict. Most guidelines only approximate the clearance and entering distances. Sometimes the conflict area dimension is considered, sometimes it is simplified to a single point – if considered at all.

Arasan et al. (2006) compared the values resulting from the calculation according to ITE (1999) and FGSV (1992) with the results from the approach proposed by Easa (1993). The research highlights slight differences in the intergreen times between the values calculated according to the German and the U.S. method respectively, whereas the procedure proposed by Easa (1993) leads to the longest intergreen times.

Arasan and Boltze (2004) recommend the use of a detailed calculation method as the one used in Germany for at least heterogeneous traffic conditions (e.g. in India), because the shortest possible intergreen times ensuring a high level of safety could be identified by this method. Empirical proof is not provided.

No research provided substantial evidence for the differences in safety and capacity of the different methods to determine clearance distances (Boltze et al. 2006). Despite the magnitude of research dealing with intergreen times and the long history of the established procedures, still several issues haven't been solved so far. While some differences among the prevailing calculation procedures may be attributed to different traffic regulations and driver behaviour, some differences are caused by insufficient empirical evidence.

2.3 Intergreen and capacity of signalised intersections

2.3.1 Introduction

As congested traffic conditions are a daily occurrence, it is eminent to accurately analyse signalised intersection performance. The capacity of individual movements at signalised intersections is dependent on various factors, and can be calculated if the effective green time and the saturation flow rate are known.

An overview of research related to saturation flow and effective green times is given in Section 2.3.2. How the capacity may be estimated is presented in Section 2.3.3. Improvement potential is rarely considered in respect to intergreen times or the change intervals. Some notions are summarised in Section 2.3.4. While much of the presented research is several decades old now, the methodological approaches and qualitative findings remain still valid.

2.3.2 Saturation flow and effective green time

Because the capacity reduction caused by intergreen has to be converted from time units (seconds) to traffic units (e.g. vehicles) per hour, the saturation flow rate has to be taken into account. In this way the capacity reduction can also be determined in a relative change of the vehicle volumes being able to pass the intersection.

However, the definition of saturation flow according to the German Highway Capacity Manual (FGSV 2001, HBS) conflicts with the common practice. While the U.S. Highway Capacity Manual (TRB 2000) introduces an effective green time, expanding the signalled green time to the time effectively used by drivers, HBS does not. The saturation flow in HBS is determined for the effective green time (all vehicles are considered, even if they cross during yellow), but it is applied to the signalled green time.

Already in 1980 JAKOB highlighted the inconsistent consideration of intergreen times. While the crossing time is assigned to the intergreen time, it is nevertheless time, used by vehicles to cross the stop line and, hence, increases the capacity (*effective green time*). He therefore assumes a higher capacity of intersections than commonly estimated. Further research would be needed to evaluate optimal stages and stage sequences.

Consequently, the methods to determine the saturation flow rate differ from Germany to the United States. For the analysis of intersection capacity the use of effective green time (following the definition of saturation flow in HCM) appears to be advisable. A direct comparison of HCM and HBS with limited scope was conducted by WU (2003).

Saturation flow has been analysed extensively over the past decades (BONNESON 1992; HOFFMANN and NIELSEN 1994; MCMAHON ET AL. 1997; LI and PREVEDOUROS 2002; TONG and HUNG 2002; LIN and THOMAS 2005; SCHNABEL ET AL. 2005; LONG 2006, 2007; TANG and NAKAMURA 2007b, to name a few). While the prevailing methodology to determine the saturation flow is based upon headway measurements and statistical analysis of the empirical data, even neural networks have been applied to give estimates of the saturation flow (TONG and HUNG 2002).

LU (1984) compared queue discharge headways from different studies (GREENSHIELDS ET AL. 1947; GERLOUGH and WAGNER 1967; CARSTENS 1971; KING and WILKINSON 1977) with his own results. His focus was on the role of vehicle size on the headways, and he could show a significant influence of the size of the first vehicles in the queue. The research revealed numerous influencing factors on the saturation flow. Since the determination of the saturation flow depends on the time until a constant flow is achieved, a close connection to intergreen times is apparent.

Because influencing factors on driver behaviour are exhibited in connection with the capacity model development (Section 4.3), they are not further explained here.

2.3.3 Capacity estimates

Already in 1973 Berry and Gandhi analysed the approach capacity at signalised intersections. They defined the effective green time by considering start-up lost times (and thus starting response time and saturation headway) and crossing times.

Pitzinger (1981) deducts a 15 % capacity reduction by intergreen times regardless of signal program settings from past experience. Krüger (1985) had a closer look at the connection between the number of stages and the capacity of signalised intersections. His investigations showed that numerous conditions influence the capacity and safety of signalised intersections. Therefore, he couldn't derive general conclusions. He could find neither a direct relationship between the capacity of the conflict area and the intersection capacity nor a relationship between the number of stages and the capacity. These relationships are scrutinised later on.

Messer and Bonneson (1997) highlight the dependancy of the prevailing factors influencing the capacity on the general volume level and the degree of existing congestion. They mention as traditional prevailing factors the interchange geometry, the traffic mix, and the signal green splits. The latter implicitly regards intergreen times as lost time without detailed thought.

In Austria, research by Köll et al. (2004a) investigated the impact of flashing green as a transition to yellow on the capacity of signalised intersections. Due to higher stopping probabilities as opposed to a transition to red with yellow only they identified a capacity reduction of about three percent for pretimed signal control. The effect was more significant for short cycle lengths and high saturation degrees. This reveals the role of signal change intervals for the capacity of signalised intersections.

2.3.4 Capacity improvement potential

It is apparent that intergreen times reduce the capacity of signalised intersections. It is also well accepted that safety has the first priority over capacity. Stein (1986, p. 438) states that "*any philosophy that accepts crashes that could be prevented merely to save 1 or 2 seconds of signal timing is contrary to traffic safety principles*".

The difficulty arises out of the fact that longer intergreen times not automatically result in higher safety, whereas the capacity reduction caused by intergreen times is not accurately known. Furthermore, the desired level of safety is commonly not defined, and the achieved level of safety cannot be reliably determined. Therefore, not only safety aspects of intergreen times are not researched to a satisfying degree so far, but the capacity reduction caused by intergreen times needs further consideration.

Regardless of the importance of intergreen times for the capacity of signalised intersections, most past research focussed on safety issues only. Intergreen times are commonly regarded as lost times, following a deterministic traffic flow model. While TRB (2000) defines an effective green time, FGSV (2001) takes exactly the intergreen times as the times not available for the generation of traffic throughput.

The difference between signalled green time and effective green time according to the U.S. Highway Capacity Manual (TRB 2000) results form start-up lost times and green time extensions (i.e. crossing times during yellow). Both are assumed to approximately equal each other as long as the saturation degree is unexceptional. Maini (1997) suggest a calculation of the clearance lost time as the sum of the all-red clearance interval and a portion (50 % to 75 %) of the yellow change interval. This leads to about the same values as the estimation of TRB concerning the clearance lost times.

In Germany, green time extensions have to be implicitly included in the saturation flow rate. Due to an yellow-and-red signal, start-up lost times tend to be very small. TANG (2008) found start-up lost times at German urban intersections to be about one second. With crossing times commonly being more than one second, effective green time can be expected to be greater than signalled green time in Germany. This highlights the improvement potential of intergreen times, since in the U.S. empirical studies propose an effective green time equalling the signalled green time.

Whereas the saturation flow rate, being the major factor in calculating the capacity of signalised intersections, takes individual intersection parameters into account (e.g. share of heavy vehicles, grade), the lost times are assumed to be constant worst case values according to the intergreen time calculation. An optimisation potential is not determined.

In practice very often a multi-stage operation of traffic signals (i.e. more than two stages, which may be desirable for safety reasons) is not implemented because of assumed losses in capacity (also due to intergreen times). While it is apparent that a correlation exists, it has never been researched in detail. Apparently the improvement potential for the capacity of signalised intersections in context of intergreen times remains vague.

2.4 Intergreen and safety of signalised intersections

2.4.1 Introduction

Since intergreen times have a significant impact on safety, safety issues cannot be ignored completely in a research focussing on capacity issues. However, it is not the aim of this research to derive safety estimates of intergreen times or to analyse models which do so.

Nevertheless, research reveals that capacity and safety improvements don't have to contradict each other. On the contrary, a lower saturation degree may come along with a safety increase. ECCLES and MCGEE (2001) state, the other way round, that increasing intergreen times reduce the capacity, potentially lead to oversaturated conditions, and consequently to driver frustration and red light violations. The same is observed by TANG and NAKAMURA (2007a). Moreover, many questions concerning the maximum safety level achievable by optimal intergreen times, and, particularly, the question what *optimal* intergreen times – from a safety point of view – are, still remain unanswered as will be shown further down.

If measures exist which can improve both safety *and* capacity, it is worthwhile to analyse safety not regardless of capacity issues. To give an impression on the safety issues at hand, the following sections give a concise overview of the safety related research in the context of intergreen times. Intersections of safety and capacity improvement potential can, thus, more easily be determined.

In Section 2.4.2 general safety related research of intergreen times is summarised. Commonly the impact of yellow timing and the duration of intergreen times cannot or is not clearly distinguished. Therefore Section 2.4.3 highlights research focussing on the determination of the yellow interval.

2.4.2 Safety assessment of intergreen times

What are "safe" intergreen times?

GLEUE (1974) already emphasised the influence of intergreen times both on the safety and capacity of signalised intersections. He stated that absolute safe intergreen times cannot be achieved. Thus, a subjective high level of safety should at least be aimed at. Safety problems would occur not only with short intergreen times, but with long intergreen times, too. JOURDAIN (1986) summarises this observation by stating that "*intergreens should always be as short as possible consistent with safety to encourage respect and immediate action from all drivers*".

To improve the safety of signalised intersections by intergreen, the possible conflict situations which can be avoided by intergreen times have to be identified. The decisive conflict situations have to represent the worst probable case of conflict potential. PITZINGER (1981) proposed to cover 90 % of all vehicles for the calculation of intergreen times, disregarding the 10 % of non-compliant drivers, based on empirical investigations. At the same time he questioned the consideration of slow vehicles, which leads to the question, which share of the speed distribution of all vehicles can be regarded as being slow. This highlights also the lack of a consistent and widely accepted safety margins.

DUNKER ET AL. (2003) point out that the intergreen times shouldn't be generally extended due to rare special situations (e.g. trams with very long clearing times). Special situations should be individually taken account of by changing the start of the green times. Whether traffic engineers can rely on the responsibility of drivers and their law compliance or should integrate higher safety margins has been discussed by STEIN (1986). He concluded that driver behaviour is irrespective of state laws. Inadequate intergreen times, hence, would increase crash rates.

The role of the post-encroachment time

Relevant for the intergreen time calculation is the consideration of entering times. Basically, the question can be expanded to the required, or useful, headway between clearing and entering vehicles at the conflict area. The time gap between clearing and entering vehicles was coined "post-encroachment time (PET)" by ALLEN ET AL. (1978). While ALLEN ET AL. focused on turning movements, the concept remains valid for all conflicts of crossing vehicles. The code of practice around the world spans from not considering entering times at all (as, for instance, in the United States) and, therefore, fostering very long post-encroachment times, to calculating the intergreen times in a way that theoretically no post-encroachment time occurs at the conflict area at all (as in Germany).

If the entering time is considered, some discussion focusses on the determination of entering speeds. In FGSV (1992), the entering speed is determined for a moving start. A moving start, however, is in most situations very unlikely as has been shown by GLEUE (1973a). He calculated probabilities for a moving start under various conditions and came up with values not higher than six percent (under standard conditions even lower). Only coordinated signal control, where moving starts are more probable, haven't been scrutinised by him. His results are supported by research recently conducted by TANG (2008): he came up with a empirically determined moving start ratio of about 5 % for left-turning traffic and less than 4 % for through traffic. TARNOFF and ORDONEZ (2004a) even proposed that the clearance interval shouldn't provide sufficient time for clearing, but only sufficient time for the entering vehicle to notice the conflicting clearing vehicle or pedestrian and therefore adjusting its speed.

The reciprocal value of the post-encroachment time can be seen as an indicator of aggressive driving behaviour. TANG (2008) compared this indicator between Germany and Japan. His observations showed higher values for German intersections, i.e. more aggressive driving behaviour. If this value is taken as an safety indicator, cultural difference have to be taken into account, reflecting the perceived safety of drivers.

Apart from the entering speed, the perception reaction times and the distance of stopped vehicles to the stop line have a signifcant influence on the entering time. However, nearly no research can be found scrutinising this aspect of intergreen times. Particularly the role of a transition time between red and green (e.g. red-and-yellow signal or count down counter) has been rarely analysed. GLEUE (1974) showed that the effective crossing time of the stop line does not depend on the duration of the transition time. Androsch (1974) confirmed this observation for coordinated signals. Weber (1983) presents a study on the safety and capacity related impacts of a red-and-yellow interval. While no strong evidence could be found, still a red-and-yellow interval of two seconds is recommended.

The "optimal" duration of intergreen times

Quite often, the notion of longer intergreen times coming along with a safety improvement, and, consequently, a safety decline by a shortening of intergreen times, is propagated. However, a number of researchers express doubts on this.

Lin and Vijakumar (1988), for instance, couldn't prove a significant safety decline by intergreen times shorter than the values deemed necessary by calculation. On the other hand, they highlight the connection between long intergreen times and low driver compliance. The same correlation is pointed out by Schnabel (1976) and Tang and Nakamura (2007a). Jakob (1980) could show, by varying the intergreen times at an exemplary urban intersection and calculating the probability of conflicts, that drivers are well capable of adjusting to shorter intergreen times without a safety decline.

A comparison of common practice in the United States (Lin and Vijakumar 1988) revealed that most intergreen times are shorter than the recommended values according to the Traffic Engineering Handbook. About 80 % of some 50 tested intersections didn't comply with the recommended values of ITE, as reported by (Retting et al. 2002). Apparently, transport professionals fear the low compliance for long change intervals (Hulscher 1984). The research by Lin and Vijakumar (1988) came to the conclusion that the ITE method lies on the conservative side. They compared the yellow demand (i.e. the 85-percentile of the crossing times) with the ITE recommended values. No dangerous conflicts could be observed regardless of short yellow times.

On the other hand, Zador et al. (1985) could show that inadequate clearance times correlate with higher crash rates. They stated that the recommended values by the Institution of Transportation Engineers are in many cases even too short. A connection between the frequency of red light running and short intergreen times has been revealed by several research projects (e.g. Behrendt 1970; Harders 1981; van der Horst and Wilmink 1986). However, advantages of extending the yellow time while reducing the all-red interval have not been proven.

The contradictions in the research may lie to a large extend in the differing methodologies. Conflicts and – even more so – accidents are rare events. Therefore, most studies are based on an interaction analysis, not on revealed conflict occurences. The safety indicator chosen and the conclusions drawn from it introduce notable bias into the research. On the other hand, drivers are influenced by numerous factors, not only yellow times or intergreen times. Consequently, the reliability of research results, that are based on several assumptions and focus on few parameters only, has always carefully to be assessed. It is a challenge to achieve significant sample size in safety related research. One countermeasure can be longitudinal studies incorporating control sites.

An extensive longitudinal study on safe intergreen times was carried out by Retting et al. (2002). Intersections not conforming to the ITE values have been divided into experimental intersections, where the signal timing have been adjusted, and control intersections. Crash rates have been observed during a 3-year study period after the changes took effect. The results show an eight percent decline in the crash frequency as compared to control sites. The decrease in crashes with injured and involving pedestrians or cyclists was even higher. Because the signal timing adjustments involved both the yellow and all-red-period, the effects cannot solely attributed to the intergreen times.

Further safety related aspects of intergreen times

Apart from crashes inside of the intersection (usually right-angle crashes), MAHALEL and ZAIDEL (1986) recommend to take rear-end collissions into account, too. They cite research by CONRADSON and BUNKER (1972) which analysed the repercussions of longer yellow timing on not only right-angle crashes, but on rear-end collissions, too. The research showed that longer yellow times with shorter all-red times reduce rear-end, but increase right-angle collissions and vice versa. This highlights the importance of balanced concepts assessing the overall situation, not regarding detailed aspects separately.

It has been stated that moving the stop line away from the intersection (resulting in longer intergreen times, as long as entering times are not considered at all or only for moving starts, as is the case in the U.S.) leads to more red light violations (ECCLES and MCGEE 2001).

TANG (2008) researched the crossing behaviour of vehicles during the intergreen interval and compared signal group based signal control with stage based signal control. Apparently the signal group based control resulted in less red light running. Because TANG draw his conclusions from a comparison of Japanese and German intersections, the observation may also partly be based in the different intergreen times.

Another aspect is highlighted by ECCLES and MCGEE (2001): the uniform design of intergreen intervals plays a major role in the adaption process of drivers to local conditions. A change of the calculation policy on one intersection may lead to a local safety increase at the cost of a decline on nearby intersections with unchanged intergreen. Research results therefore may not only depend on the researched intersection itself, but on the general local policies, too.

2.4.3 The role of yellow time

2.4.3.1 The yellow time dilemma and crossing times

When approaching a signalised intersection at the end of green, a driver has always the two possibilities of proceeding or stopping. If a driver can neither stop in front of the stop line nor cross the stop line before the onset of red, he's caught in a dilemma called yellow time dilemma. The name is derived from the *yellow time* (or amber time), introduced between green and red to exactly avoid this conflict. In some countries, not only yellow is used to indicate the change of stages, but also a flashing green signal or a count down counter.

Quite a lot of research deals with the yellow time dilemma. Driver behaviour, a pivotal aspect in the yellow time dilemma, is relevant for the clearing process of vehicles, too. The yellow time, moreover, is the link between green time and intergreen time, since it can be both part of the effective green time and part of the lost time not used for traffic throughput. The limit between the two is the crossing time. The crossing time of clearing vehicles describes what part of the yellow time is used by vehicles (in TRB (2000) called green time extension) and what part represents the clearance lost time.

2.4.3.2 Calculating the yellow time

The determination of the yellow time resulting in the highest safety has been debated for several decades (e.g. GAZIS ET AL. 1960; RETZKO 1966; OLSON and ROTHERY 1972; WILLIAMS 1977; VAN DER HORST and WILMINK 1986; TECHNICAL COUNCIL COMMITTEE 4A-16 1989; CHANDRA 1999). Yellow times differ significantly over the world. The recommendation for yellow times in the not yet published revision of the German Guidelines for Traffic Signals (RiLSA) (FGSV 2010) has again been adjusted for high speeds following experiences of transport engineers, underlining the ongoing debate, and, furthermore, the discrepancies between seemingly exact models and the reality with its numerous irregularities.

While in Germany the yellow time is a fixed value, only depending on the speed limit at the intersection, in many other countries the values may change from intersection to intersection. In the U.S., for instance, the yellow time commonly is calculated individually for every intersection. Sometimes, however, local authorities develop their own standard values for certain intersection types or their whole jurisdiction.

In the U.S., the yellow time is part of the intergreen time equation. Thus, crossing time and yellow time are equated. In Germany, however, the crossing time is based on the yellow time, but considered separately in the intergreen time formula. For turning traffic, bicycles, and in case of long yellow times (more than three seconds for high speed intersections) even for through traffic, the crossing time is assumed to be shorter than the yellow time. Only for through vehicles at intersections with a speed limit of 50 km/h, crossing time and yellow time are identical. Following the German guidelines, a separate analysis of yellow time and intergreen time is mandatory (cf. also Gleue 1973a).

The most common way of determining the yellow time, is a calculation based on reaction times, deceleration rate, and the speed of the approaching vehicles (either using the speed limit or percentiles of the speed distribution). However, in several studies it was shown, that the driver behaviour is less influenced by the yellow time or the speed, but primarily by the distance to the stop line at the onset of yellow (e.g. Hulscher 1980; van der Horst and Godthelp 1982). Nevertheless, the distance to the stop line, where half of the drivers decide to stop, depends on the yellow time duration (van der Horst and Wilmink 1986). Consequently, models were developed to estimate the stopping probability as a function of the distance to the stop line.

Alternatives to the kinematic model

Eccles and McGee (2001) analysed alternative approaches to the yellow time calculation than the kinematic model. Three procedure are discussed:

Uniform Approach While Benioff et al. (1980) couldn't find any indication that a uniform yellow interval is of advantage, Frantzeskakis (1984) supported the idea of a uniform yellow interval with the duration depending on the speed. This prodecure coincides with the one in the German Guidelines for Traffic Signals (RiLSA). Wortman et al. (1985) proposed a uniform yellow interval of four seconds.

Stopping Probability Method Olson and Rothery (1962) derived the stopping probability as a function of the distance to the intersection. The equation for the minimum yellow change interval is based on the distance where 95 % of drivers are likely to stop, the intersection width, vehicle length, and the speed

Combination of both A combination of both approaches was discussed by Williams (1977), but not further pursued.

Easa (1993) expanded on these ideas by developing a method to calculate the probability of failure (i.e. the probability of occurence of a dilemma zone) based on speed, perception-reaction time, deceleration, and vehicle length as random variables with mean and standard deviation.

2.4.3.3 Neglected influencing factors

The influence of heavy vehicles or an ageing population on the yellow time dilemma has not been researched so far (Eccles and McGee 2001). Studies, however, indicate that the former shouldn't be neglected due to the high probability of lorries being the last clearing vehicles (Technical Committee 4A-16 1985). Other studies indicated a dependency of the crossing time on the vehicle supply (Lin and Vijakumar 1988).

2.4.3.4 Consequences of improper yellow times

The meaning of the yellow signal can be interpreted in two ways: permissive rules allow the driver to cross the intersection during yellow, while restrictive rules prohibit it unless the situation requires it. Anyhow, the yellow time should be set in a way that supports compliance with its purpose.

It is agreed that too long yellow time leads to disrespect by the drivers. However, what *too long* means in numbers, is disputable. Butler (1983) and Tarnoff and Ordonez (2004a) propose yellow times in a range of three to five seconds, which can be found in the German Guidelines (FGSV 1992), too. Transport professionals in Germany even propose not to use yellow times longer than four seconds (cf. FGSV 2010). Yellow times in Austria are opposedly recommended not to be *shorter* than four seconds (TFSV 1998), but Köll et al. (2002) state that the time to the end of yellow is commonly underestimated.

Opposing to these tendencies an empirical study showed a reduction of red-light violations by a prolongation of the yellow time from three to four seconds (van der Horst and Wilmink 1986). The effect, however, could depend on the signal control policy.

One possibility to reduce the dilemma zone for drivers (i.e. situations where the driver can neither stop in front of the stop line nor proceed into the intersection during yellow, due to insufficient yellow time), is the use of a flashing green interval before the onset of yellow. However, this flashing green, as it is used, for instance, in Austria, leads to an enlarged option zone, in which drivers may both stop or proceed. This enlargement is positively correlated to rear-end collissions (Köll et al. 2002).

While already in 1966 the possible role of countdown counters instead of or in addition to yellow time was questioned by Retzko, still their possible influence on safety (and capacity!) is unknown. Nevertheless, count down counters in some countries become more frequent. A possibly outdated project revealed that traffic actuated control apparently tends to shift the drivers' decision making point away from the stop line and results in lower deceleration rates (Zegeer and Deen 1978).

A difficulty in predicting the impacts of changed yellow times lies in the many influences on driver behaviour. Drivers react differently if changes apply only locally at a single or a few intersections. The short term reaction differs from long term adaptation. Long term in this context means several years (Hulscher 1984; van der Horst and Wilmink 1986). In 2001 Eccles and McGee alleged that the influence of longer yellow intervals on driver behaviour is, despite all efforts to analyse it, still not clear, and propose further research.

2.5 Random character of traffic flow

Jakob (1980) in Germany and later on Easa (1993) in the United States conducted extensive research into intergreen times with the random character of traffic flow taken into account. Jakob (1980) highlighted the lack of achieved accuracy in spite of the detailed calculation in the method according to the

German Guidelines for Traffic Signals (FGSV 1977, which is still used in the current edition from 1992 and the drafted edition 2010). Three areas are scrutinised in JAKOB's thesis:

- the traffic flow model used for calculation of the crossing, clearing, and entering times of vehicles
- the error propagation
- the rounding at the end of the calculation procedure

He expands upon a number of problems occurring due to the assumptions concerning the traffic flow model. Concluding, he suggests to regard the conflict point as a black box, disregarding the circumstances under which vehicles appear at a certain time in this black box, but focussing solely on the fact that they appear at a certain time. In this way, however, a detailed analysis on the factors influencing the required intergreen times will not be possible. JAKOB used the fixed values of FGSV (1977) and the standard deviation for the respective parameters taken from literature to estimate the mean error to be expected for the intergreen times at some exemplary intersections. This error due to this discrepancies amounted to 0.8 s to 1.2 s. The rounding at the end of the intergreen calculation according to RiLSA to the full second is therefore not able to compensate for the error in all cases. Moreover, the difference between the intergreen time used (after rounding up) and the exact value calculated is in no way related to the probable error or to the risk in underestimating the intergreen time needed to achieve the highest possible safety level.

EASA (1993) used stochastic methods to determine the safety level at signalised intersections during the change of stages. He calculated the probability of the occurence of a dilemma zone based on speed, reaction times, deceleration rates and vehicle length as random variables with certain mean and standard deviation. TARNOFF and ORDONEZ (2004a) highlight the difficulties and inaccuracies arising out of a deterministic model for a stochastic problem. They point out the various percentiles used for different parameters in the U.S. (95-percentile for reaction time, 85- and 15-percentile for speed, 50-percentile for deceleration rates). The influence of the variability of traffic flow characteristics on the results of optimisation procedures for traffic signal control was pointed out by SCHENK (1993).

2.6 Special issues

Lead time vs. lag time

Some research looked into the pros and cons of leading versus lagging left-turn traffic (e.g. DEIST 1972; HOFFMANN and ZMECK 1982; SHEFFER and JANSON 1999; BASHA and BOX 2003). This research looked both at the safety and capacity impacts of the signal programs. Particularly protected leading left-turn stages are still subject of controversy in Germany and further research is proposed (BOLTZE ET AL. 2006). SHEFFER and JANSON (1999), however, could neither show significant differences in accident data for both types, nor in saturation headway data. Only start-up lost times seem to be lower for lagging left turns. NOYCE ET AL. (2000) also revealed an influence of stage sequence on start-up lost times.

Traffic actuated intergreen times

A discussion of the German Guidelines for Traffic Signals (BOLTZE ET AL. 2006) raised also the issue of traffic actuated intergreen times. They are sometimes realised for public transport vehicles and left turning traffic. Anyhow, no sufficient recommendations could be found for this kind of traffic actuated intergreen times. Particularly for one lane two-way facilities (bottlenecks) during road works, possibilities to deploy traffic actuated intergreen times are wanted (see FOLLMANN 1989).

Intergreen times at signalised roundabouts

Jourdain (1988) examined intergreen times at signalised roundabouts next to motorway exits. The research lead to no significantly differing results as compared to standard signalised intersections. Yellow times of three, for fast traffic of four seconds are recommended. The intergreen time should consider the line of sight with longer intergreen times for poor sight.

Heterogenous traffic

The traffic characteristics of countries with heterogenous traffic, prevailing in East Asian countries, is vastly different from homogenous traffic. Maini and Khan (2000) therefore scrutinised discharge and clearing behaviour at signalised intersections under such conditions. However, the variation of clearance speed among different vehicle types prooved to be inconsiderable.

2.7 Conclusions

The literature review supports the assumption that, as far as intergreen times are concerned, still many questions remain unanswered. *The optimal* intergreen time calculation method cannot be seen. The driver behaviour is influenced by ample factors posing major difficulties for the research. Methodologies to analyse safety and capacity performance of intersections under different intergreen time settings vary as much as the detailed results.

Some conclusions, nevertheless, can be drawn:

- Intergreen times should neither be too short nor too long from both a capacity *and* safety point of view.
- The first step towards a sensible intergreen calculation method has to be the agreement on precise performance goals (i.e. acceptable risk, balance of responsibility of drivers vs. traffic engineers etc.).
- Intergreen times are quite often based on experience instead of empirical data (either directly by the individual traffic engineer/the responsible authority, or indirectly through recommendations in guidelines).
- Perceived safety improvements due to assumptions in the intergreen time calculation and verifiable safety performance can differ notably.
- The variation of intergreen times indicates a capacity improvement potential, which may be utilised without an unacceptable safety decline.

3 Theoretical determination of effective and maximum capacity

3.1 Introduction

3.1.1 Chapter outline

This chapter presents the theoretical background for the development of a model to determine the capacity of signalised intersections under consideration of the true traffic flow during the signal change intervals. First of all, the rationale behind the focus on *capacity* is explained Section 3.1.2, as are the concepts to define capacity Section 3.1.3. The introduction closes with the basics of the capacity calculation Section 3.1.4.

Section 3.2 expands upon the difference between the signalled green time and the effective green time (green time differences), which is the basis for the calculation of the *effective capacity*. The section is split into the behaviour of the entering vehicles, the behaviour of the clearing vehicles, and the interaction between the two.

The theoretical optimisation potential of intergreen times with reference to the capacity is detailed in Section 3.3. Potential differences between parameter values assumed in the intergreen time calculation and the effective values are explained, the role of safety margins is mentioned, and the determination of decisive conflicts is expanded upon. The connection between intergreen time differences, derived from these three aspects, and the capacity is highlighted in the last part of the section.

Finally, some thought is given to the random character of traffic flow and its impact on green time and intergreen time differences (Section 3.4).

3.1.2 Rationale and derivation of terms

3.1.2.1 Reasons for the focus on capacity

The capacity is a basis for the design of traffic facilities. It is needed to optimise the size, layout and right of way regulation of intersections. At signalised intersections, the capacity is used, among others, to determine the signal timing, more specifically the cycle time and the green time split. When looking at performance measurement, most indicators are based on the capacity or on parameters related to the capacity (namely the green time t_G, the cycle time t_C, and the saturation flow rate q_S):

- volume/capacity ratio
- delays $t_d = f(t_G, t_C, q_S)$
- percentage of stops $p_s = f(t_G, t_C, q_S)$

Furthermore, the description of intersection performance by, for instance, delay leads to quite unstable results for saturated conditions (Dion et al. 2004). For the optimisation of street networks, bottlenecks have to be determined. Hence, the traffic demand has to be related to the available capacity. Moreover, the capacity is, as opposed to the number of stops and even more so delays, easy to determine. The estimation of delays at intersections has been the subject of numerous investigations (e. g. Webster 1958; Allsop 1971). An overview of available models is given by Hurdle (1984). Most of them are based on lost times, without having analysed them in detail, particularly as far as intergreen times are concerned.

This research looks into the factors influencing the capacity of signalised intersections, the lost times among them. Since the term *lost times* is used in different contexts, in this research it is evaded to avoid confusion. Statements are refered to the capacity by way of *effective green times* and *minimum intergreen times* instead. *Calculated*, *effective* and *maximum* capacities are opposed to each other as a way to expose the impacts of intergreen times.

3.1.2.2 General relationship between intergreen time and capacity

Capacity and base capacity

The capacity of signalised intersections depends on the saturation flow on the one hand, and on the signal program on the other hand. A number of factors influences both, the saturation flow and the signal program. The signal program consists of green times and intergreen times (*green time* in this context means times, at which *any one* stage has green), which fill together the cycle time. If two of the three parameters (cycle time, green times, intergreen times) are known or defined, the signal program can be set up.

The capacity of an intersection can be further reduced by the gap acceptance behaviour of permitted turning streams. The capacity without taking this constraint into account is termed *base capacity*. In the focus of this research are the intergreen times and, thus, the *base capacity*, which is referred to by the term *capacity* further on, if not specifically distinguished from the base capacity.

The basic connection between the traffic flow, the signal program, and the different capacities is illustrated in Figure 1. This basic structure will be extended in Chapter 4.

Effective green time

The signal program should be designed in a way that the traffic demand can be accommodated while ensuring a high level of safety. The green times are used to satisfy the traffic demand, while the intergreen times are introduced for safety reasons. Following this definition, intergreen times reduce the capacity. The shorter they are, the higher the capacity will be. The capacity based on these simple assumption is termed *calculated capacity*.

However, vehicles do not cross the intersection only during the signalled green times. Effectively used signal intervals (*effective green time* t_g) and *signalled green time* t_G differ by the *green time difference* Δt_{gG}. In TRB (2000), for instance, the effective green time is computed by defining *start-up lost times*, *clearance lost times*, and a *green time extension*. By determining the *effective green times*, the *effective capacity* can be calculated.

Determination of optimisation potential of intergreen times

Furthermore, the intergreen times bear an optimisation potential. Strictly speaking, intergreen times ensure that *conflict areas* are cleared before entering vehicles reach them. For deterministic driver behaviour and complete information on the traffic flow in the intersection, the intergreen times could be exactly calculated to satisfy this requirement. No headways between clearing and entering vehicles (post-encroachment time) would, in this ideal case, occur. A signal program designed with these deterministic *minimum intergreen times* $t_{ig,min}$ will lead to the *maximum capacity* of the intersection.

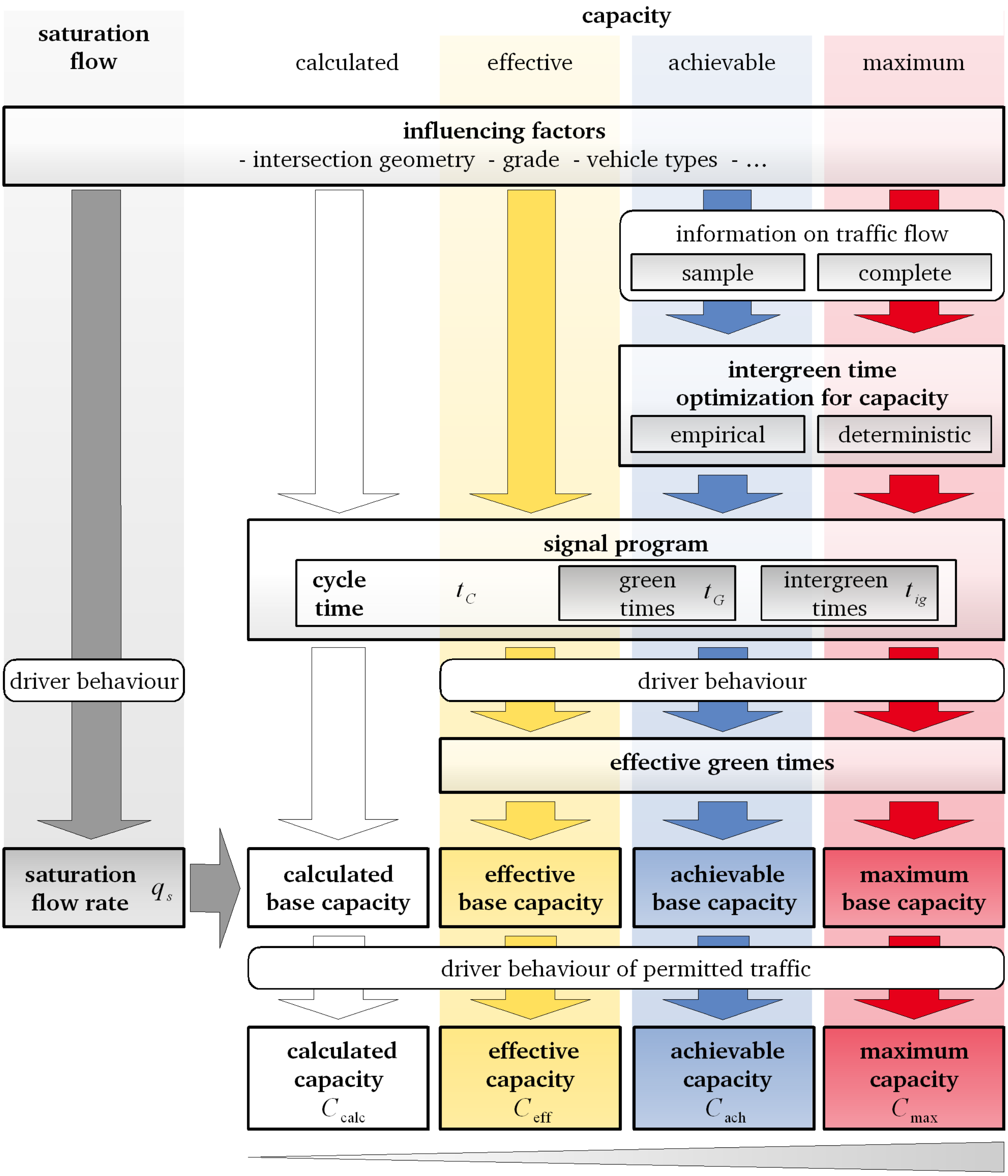


Figure 1: Relationship between intergreen time and intersection capacity

The *intergreen difference* between the minimum intergreen times $t_{\mathrm{ig,min}}$ and the actually used intergreen times t_{ig} is calculated as $\Delta t_{\mathrm{ig}} = t_{\mathrm{ig,min}} - t_{\mathrm{ig}}$. Intergreen time differences can be assigned to signal groups, leading to *green time extensions* Δt_{G}. These green time extensions can be converted to vehicle volumes and, thus, are a measure of the maximum optimisation potential of intergreen times from a capacity point of view. It has to be noted that this maximum optimisation potential can only be achieved by a controlled vehicle flow or complete information on the behaviour of every single vehicle approaching and crossing the intersection. Deterministic intergreen times would have to be calculated dynamically for every single change of stages. This won't, of course, be possible in the near future.

Achievable intergreen times

For random driver behaviour safety margins have to be added to the intergreen times, leading to *empirical intergreen times*. These empirically to be derived safety margins are not in the focus of this research. Nevertheless, the outcome of this research will be an estimate of the capacity impacts of different safety margins. The *achievable capacity* of an intersection, i.e. the capacity achievable with consideration of safety margins, will be somewhere between the *effective capacity* and the *maximum capacity*.

3.1.3 Concepts to define capacity

Different concepts of defining capacity exist. The most common one found for signalised intersections, which is followed here, is the traffic flow in vehicles per hour. Using vehicles per hour has some drawbacks:

- The number of vehicles being able to pass an intersection depends on the vehicle mix, particularly the share of heavy vehicles, motorcycles and bicycles.
- No information is given on the number of travellers crossing the intersection. While the number of vehicles crossing an intersection is a good indicator for the quality of traffic, it is in itself commonly not of primary interest. The basic aim of traffic is the transport of goods and people.

The former problem can be solved by giving the capacity in passenger car units per time. However, this would commonly presume a linear relationship between the capacity and the vehicle type (or otherwise lead to a similar approach as using vehicles instead of passenger car units). In reality, the capacity decreases not linearly with an increasing heavy vehicle share, for instance. The capacity always depends on the vehicle mix, which justifies the use of vehicles per hour. The capacity is only valid for an individual situation.

To calculate the capacity in travellers per time, the occupancy of vehicles has to be known. For intersections predominantly with car, motorcycle and bicycle traffic, the number of vehicles is a good indicator for the number of travellers passing. A linear relationship between the number of travellers and the number of vehicles can be assumed. In the case of public transport, the approach of determining the number of travellers becomes more useful, but poses additional problems. It has to be the aim of other projects to tackle these problems.

3.1.4 Basics of the capacity calculation for signalised intersections

3.1.4.1 Intersection capacity

The capacity of an intersection is the sum of the capacities of all approach lanes at the intersection (cf. FGSV 2001, Eq. 4). The approach lanes are distinct from each other. They are clearly assigned to signal groups. Thus, lanes are easier to observe than vehicle streams (as an alternative to summing up volumes on lanes; streams are more difficult to observe, because they may share the same lanes).

$$C = \sum_{i}^{n_l} C_{l,i} \tag{4}$$

where	C	intersection capacity	(veh/h)
	$C_{l,i}$	lane capacity	(veh/h)
	i	index indicating approach lane	

TRB (2000) proposes the use of *lane groups*, a concept grouping lanes with similar characteristics and identical signalisation together. This concept is based on the same principals as focussing on single lanes. The concept can also be found in FGSV (2001), even without the mentioning of the term *lane group*. To determine the capacity of lanes, it can be useful to analyse lane groups first. However, the basic procedure remains the same. The details of the capacity calculation for lane groups can be found in TRB (2000).

3.1.4.2 Lane capacity

The capacity of each lane is determined by the saturation flow and the green time. Saturation flow and green time can be defined in different ways. The most common ones, i. e. the definitions from FGSV (2001) and TRB (2000), and the one used in this study are given in Section 3.1.4.3. The traffic flow rate is the reciprocal of the time headway (Eq. 5), therefore the headway can be used as a subsitute for the saturation flow rate.

$$q = \frac{t_{\text{obs}}}{h} 3600\,{}^{\mathrm{s}}/_{\mathrm{h}} \tag{5}$$

where	q	traffic flow rate	(veh/observation time)
	t_{obs}	observation time (commonly 1 h)	(h)
	h	time headway	(s/veh)

The capacity of approach lanes at signalised intersections in comparison to free flow sections is reduced for three reasons:

- the near-side intersection geometry (i. e. curb radius, grade, lane width etc.) and the traffic situation (i. e. influences through parking, public transport vehicles etc.)
- the right-of-way regulation (i. e. permitted streams)
- the signalisation (i. e. red time, intergreen times, other lost times)

The near-side intersection geometry and the traffic situation can have both an influence on the saturation flow and the effective green time. The right-of-way regulation leads to stops during the green time with the consequent capacity reductions (capacity vs. base capacity). Of primary interest in this research, however, is the effective green time which is influenced by the signal program. The headway, nevertheless, is needed for a quantification of capacity reductions in terms of vehicles per time.

3.1.4.3 Saturation flow and effective green time

Since at signalised intersections the flow is interrupted by the signal program (conflicting streams are receiving the right-of-way subsequently), every hour consists of intervals with traffic flow and intervals without traffic flow. The intervals which serve the traffic demand are the *green times* of the different signal groups. But even during the green times, the traffic flow varies over time. Particularly the first few vehicles have greater headways than the following vehicles.

Two ways exist to derive the lane capacity from saturation flow and green time:

- The approach pursued in this research is based on the definitions in TRB (2000): a saturation headway is determined by observing vehicles in the middle of the green time under saturated conditions. The effective green time is then determined to take all capacity reductions with reference to the saturation flow into account. Hence, the effective green time is a ficticious interval which cannot be observed directly on site, but the saturation flow can be measured.

- The alternative is the approach followed in FGSV (2001): no effective green time, but the signalled green time is used, and the saturation flow is adjusted to take capacity reductions into account. Hence, the green time is known, but the saturation flow is a ficticious value which cannot be observed directly on site.

However, FGSV (2001) does not define an *effective saturation flow rate*, as is used when following this approach. Moreover, most capacity reductions occur as time differences between the *signalled green time* and the *effective green time* or between the *saturation headway* and the *individual headway*. Thus, the calculation of the capacity is facilitated by using the former approach.

Consequently an *effective green time* t_g is defined as the (ficticious) interval during which the traffic flows with the headway under free flow conditions, called the *saturation headway* h_s. All capacity changes caused by the signal change are determined as time differences with reference to the green time and the saturation headway. By relating the quotient of effective green time and saturation headway to an *observation time* t_{obs}, commonly one hour, using the *cycle time* t_C, the capacity of an approach lane can be calculated as in Eq. 6.

$$C_{\mathrm{l,eff}} = \frac{t_{\mathrm{obs}} \cdot t_{\mathrm{g}}}{h_s \cdot t_{\mathrm{C}}} = \frac{3600\,{}^{\mathrm{s}}\!/\!_{\mathrm{h}} \cdot t_{\mathrm{g}}}{h_s \cdot t_{\mathrm{C}}} \tag{6}$$

where			
	$C_{\mathrm{l,eff}}$	effective lane capacity	(veh/h)
	t_{g}	effective green time	(s)
	t_{C}	cycle time	(s)
	h_s	saturation headway	(s)

The traffic flow has to be determined at a reference location. The most straight forward location is the stop line at the approach. It is fixed for all movements, easy to determine for all lanes, and it is the reference location for the signal program design. The capacity is, thus, derived from the number of vehicles crossing the stop lines of all approach lanes.

Capacity reductions

Capacity reductions occur due to changes in drivers' following behaviour (increased headways or acceleration processes) and due to time differences between the signalled green time and the crossing of the stop line. Both reductions can be expressed in time units (*green time difference* Δt_{gG}).

The lane capacity can be therefore determined by measuring the saturation headway and investigate the difference Δt_{gG} between the *signalled green time* t_G and the *effective green time* t_g (Eq. 7 and Eq. 8. Negative green time differences, thus, mean capacity reductions. The green time differences are examined in Section 3.2.

$$t_{\mathrm{g}} = t_{\mathrm{G}} + \Delta t_{\mathrm{gG}} \tag{7}$$

and thus

$$C_{\mathrm{l,eff}} = \frac{3600\,\mathrm{s/h}}{h_s} \cdot \frac{t_{\mathrm{G}} + \Delta t_{\mathrm{gG}}}{t_{\mathrm{C}}} \tag{8}$$

where			
	$C_{\mathrm{l,eff}}$	lane capacity	(veh/observation time)
	t_{g}	effective green time	(s)
	t_{G}	signalled green time	(s)
	Δt_{gG}	green time difference	(s)
	t_{C}	cycle time	(s)
	h_s	saturation headway	(s/veh)

3.1.4.4 Connection between conflict areas and capacity

As has been explained before, the capacity depends directly on the duration of intergreen times. Intergreen times are derived from the occupation of conflict areas. The time needed by vehicles to reach or clear the conflict areas determines the intergreen times. Instead of calculating the capacity at the stop line the capacity could be directly derived from the occupation of conflict areas as has been proposed by MOSKOWITZ and WEBB (1955). This approach, though, has some major drawbacks:

- Decisive conflict area
  Every intersection has numerous conflict areas. Not all of them are decisive. For every change of stages commonly only one conflict area becomes decisive. Decisive means that the time needed by the last clearing vehicle to clear this conflict area minus the time needed by the first entering vehicle to reach it is longer than for all other conflict areas. To calculate the capacity, thus, it has to be determined which conflict area is decisive. This conflict area may only be decisive for one specific change of stages.

- Unoocupied interval of conflict areas
  During the stage changes for which a conflict area is not decisive, it may be unoccupied for some interval. This interval has to be known to calculate the capacity of the conflict area. It depends on the intersection layout and the stage settings. Furthermore, during certain stages the conflict area may not be used at all, which also depends on the stage settings and the intersection layout.

- Position of conflict area
  The exact position of the conflict area depends on the vehicle trajectories. The exact conflict point determining for a movement sequence may vary.

- Headways at the conflict area
  To calculate the capacity of a conflict area, the headways at the conflict area have to be known. For accelerated movements (e.g. entering vehicles), these headways are different from the ones at the stop line. To determine headways inside of the intersection requires an elaborate survey layout. The random variation of the conflict point has to be considered.

The capacity of the conflict areas can be determined if the stage settings, the vehicle trajectories, and the capacity at the stop line are known. It will be shown later in this report, that the capacity at the conflict area is suitable to derive conclusions for the intersection layout, particularly the role of entering behaviour and the entering distances.

3.2 Green time differences

3.2.1 Introduction

The effective green time depends on the green interval actually signalled and the processes during the change of stages, namely the behaviour of the last clearing and the first entering vehicles including their interaction. Since the *signalled green time* is known, only the *green time differences* Δt_{gG} have to be determined.

The processes leading to the green time difference are

- the entering behaviour of vehicles (start-up lost times, t_{SUL})
- the clearing behaviour of vehicles (crossing times, t_{cr})
- the interaction of vehicles (interaction times, Δt_{PE})

These processes are depicted in Figure 2. They will be explained in the following sections.

While the effective green time has been researched in detail before, interaction times are commonly not taken into account. The approach introduced here, thus, presents are more comprehensive approach. It is, furthermore, the basis for the improvement potential developed in the following section.

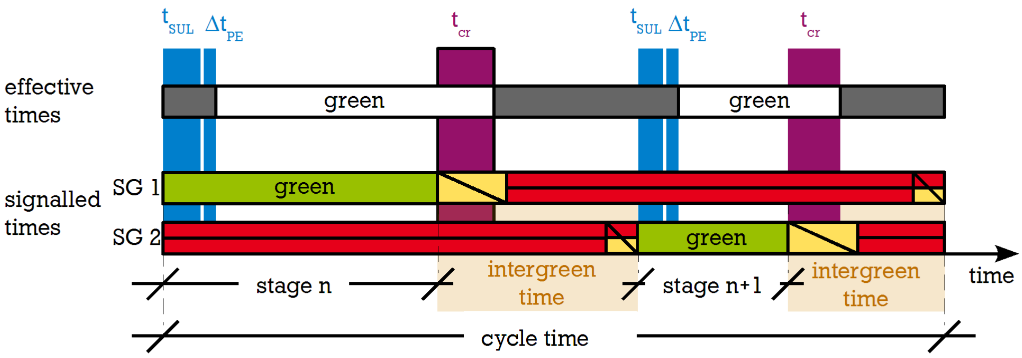


Figure 2: Illustration of the determination of effective green times during the signal cycle (t_{SUL}: start-up lost time, Δt_{PE}: interaction time, t_{cr}: crossing time, SG: signal group)

3.2.2 Behaviour of entering vehicles

The very first vehicle entering at the onset of green may have a shorter headway than the saturation headway, because it commonly does not have to cover a full vehicle length until crossing the stop line. The driver of the first vehicle, however, has to realise the signal change and react to it. This perception-reaction time is called *starting response time* t_{SR}. Commonly this starting response time includes not only the perception-reaction time, but incorporates the time needed by the first vehicle to cover the distance between its stopping position and the stop line l_{SL}. Furthermore, the starting response time can be related to the onset of green or the first indication of the impending signal change (e.g. red-and-yellow).

To avoid confusion and in consequence of the terming used in this document, the interval between the onset of green and the crossing of the stop line by the first entering vehicle is called *crossing time of the entering vehicle*. This term is consistent with the crossing time of clearing vehicles. It also can be used for entering vehicles not stopping (moving start), where the term *starting response time* cannot be applied.

The crossing time of entering vehicles depends not only on the perception-reaction time of the driver, the distance to the stop line, and the acceleration behaviour, but on the time the driver gets the indication for the impending signal change. This indication may be the green signal, a red-and-yellow signal (e.g. in Germany), a count down counter, or the signal change of some adjacent signal (e.g. red signal of crossing traffic, parallel pedestrian signal). Since this indication can vary among intersections and among drivers (e.g. local vs. non-local), the starting response time cannot be directly observed, as long as the reference time is not defined.

The entering vehicles following the leading vehicle commonly have a longer headway than the saturation headway. Their headways together with the crossing time has therefore to be compared to the saturation headway for the number of entering vehicles, to get the green time difference of entering vehicles, commonly termed *start-up lost time* t_{SUL}. In mathematical terms the sum of the headways of the entering vehicles including the crossing time minus the saturation headway times the number of entering vehicles leads to the *start-up lost time* t_{SUL}.

The sum of the additional time headways needed by the entering vehicles up to the $(k+1)^{th}$ vehicle as compared to vehicles under saturation flow, is termed *cumulated headway difference* $\Delta h(k)$ and defined according to Eq. 9.

$$\Delta h(k) = \sum_{i=1}^{k} h_i - k h_s \tag{9}$$

The index i qualifies the ordinal headway number. h_1 is the first time headway, i.e. the headway between the first and second vehicle. In this way the start-up lost times can be calculated according to Eq. 10.

$$t_{\mathrm{SUL}} = t_{\mathrm{cr,e}} + \sum_{1}^{k} h_i - (k+1)h_s = t_{\mathrm{cr,e}} + \Delta h(k) - h_s \tag{10}$$

where	t_{SUL}	start-up lost time	(s)
	$t_{\mathrm{cr,e}}$	crossing time of entering vehicle	(s)
	h_i	headway between vehicle $i+1$ and i	(s/veh)
	$k+1$	number of vehicles until saturation headway is reached	(-)
	h_s	saturation headway	(s/veh)
	$\Delta h(k)$	cumulated headway difference	(s)

The start-up lost time difference is the difference between considered and not considered start-up lost times. While TRB (2000) does consider start-up lost times, they are not considered explicitly in FGSV (2001). For German conditions the *start-up lost time difference* Δt_{SUL} equals therefore the start-up lost time ($\Delta t_{\mathrm{SUL}} = t_{\mathrm{SUL}}$). TRB (2000) states a similar magnitude of start-up lost times and crossing times of clearing vehicles and, thus, recommends to neglect the green time differences, if no empirical data is available. How many vehicles are considered to be entering vehicles (k) should be dermined empirically. About five vehicles is commonly assumed.

It has to be noted that the first vehicle may be influenced by the vehicle flow in the intersection, which is dealt with in Section 3.2.4.

3.2.3 Behaviour of clearing vehicles

Vehicles approaching the intersection at the end of the green interval always have the choice of stopping or proceeding (as long as the vehicle ahead is proceeding). Relevant for capacity calculations is always the last proceeding vehicle. If this vehicle crosses the stop line exactly at the end of green, no green time difference occurs. Usually the last vehicle crosses later, leading to a positive crossing time or (effective) green time extension t_{cr}.[5]

Research has shown that the headway of the last crossing vehicles tends to increase for very long green times (cf. LONG 2006, with reference to the Canadian Capacity Guide for Signalized Intersections and green times of more than about 50 s). However, these capacity reductions are not directly related to the signal change intervals, but to the green interval itself. Moreover, this case is an exception and is particularly rare in Germany with cycle times scarcely exceeding 120 s. Hence, it has been neglected in the model.

It should be noted that crossing times are closely related to the traffic regulations. While most countries adopt a *permissive yellow rule*, allowing drivers to cross the stop line at the end of yellow, some states in the U.S. apply a *restrictive yellow rule*, which forces drivers to clear the whole intersection during yellow, resulting in reduced crossing times.

3.2.4 Interaction of vehicles in the intersection

Each conflict area can only be used by one vehicle at a time. Physically, the conflict area may be occupied by an entering vehicle as soon as the last clearing vehicle left it. Since human drivers steer the vehicles, a post-encroachment, i.e. a time gap between the two vehicles at the determining conflict point, will have to be taken into account. The determining conflict point is the point last cleared by the clearing vehicle and first occupied by the entering vehicle.[6] This process is illustrated in Figure 3. The conflict area is shaded and the conflict point is marked by a red line.

If the clearing vehicle crosses the stop line late or at low speed, the driver of the entering vehicle may adjust his behaviour by starting later or with lower acceleration than he would without any clearing vehicle in the intersection. Capacity gains by late clearing vehicles are in this way partly compensated.

This changed entering behaviour will occur only, if

- the difference between the assumed and effective departure time of the clearing vehicle from the conflict area leads to a (theoretical) post-encroachment time not accepted by the entering vehicle's driver,

which means that for a safe intergreen time calculation

- the conflict has no significantly lower intergreen time than the determining one, and
- the last clearing vehicle is slower or later than assumed in the intergreen calculation.

For conflicts not determining for the intergreen interval, additional time may be available to compensate for the late departure of the clearing vehicle (cf. Section 3.3.4). In these situations, the entering vehicle's driver may not have to adjust his behaviour.

[5] "Extension of green time" is the term used in TRB (2000), while "crossing time" (Überfahrzeit) is utilised in FGSV (1992); here the latter term as the more precise one is applied.

[6] Details will be given in Section 3.3.2.2

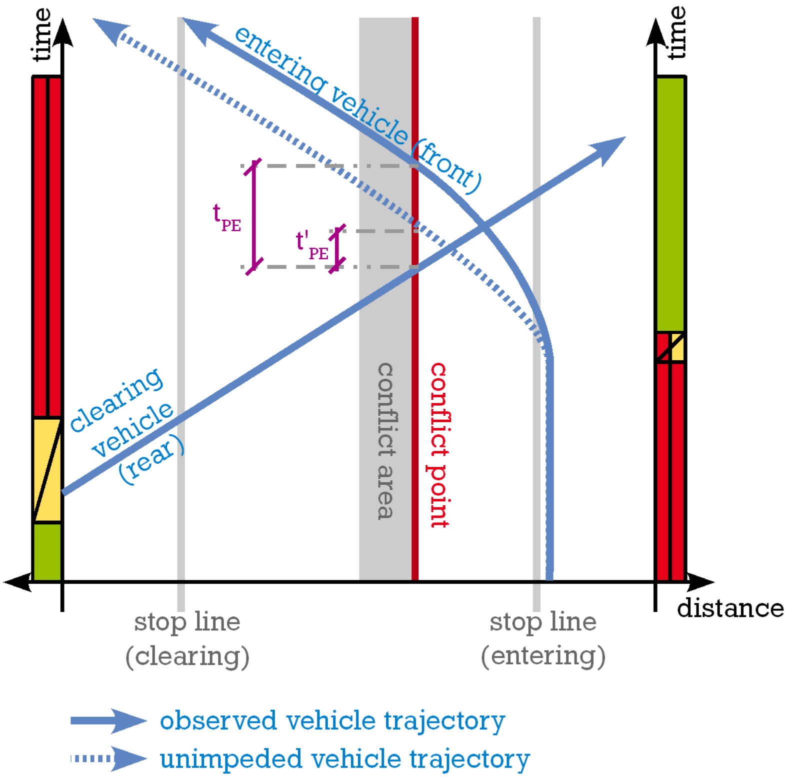


Figure 3: Interaction of vehicles during the stage change (t_{PE}: post-encroachment time), t_{PE}': theoretical post encroachment time

The difference between the theoretical post-encroachment time t'_{PE}, that would occur without the entering vehicle's driver adusting his behaviour, and the effective post-encroachment time t_{PE}, called *interaction time* Δt_{PE}, is part of the green time difference (Eq. 11).

$$\Delta t_{PE} = t_{PE} - t'_{PE} \tag{11}$$

where	Δt_{PE}	interaction time (green time difference due to prolonged post encroachment time)	(s)
	t_{PE}	adjusted post encroachment time	(s)
	t'_{PE}	theoretical post encroachment time	(s)

Problematic is the fact, that Δt_{PE} cannot be observed on site. Whether or not a driver changed his behaviour due to other vehicles in the intersection can only be guessed from indirect observations. One way is to compare starting response times and entering speeds of vehicles in critical situations with those from vehicles where the above mentioned conditions do not apply. A possible evaluation methodology is introduced in Section 5.3.6.

3.2.5 Overall green time difference

The overall green time difference Δt_{gG} consists of the sum of all time differences described in the Sections 3.2.2 through 3.2.4 with positive values resulting in a longer effective green time (Eq. 12).

$$\Delta t_{gG} = -t_{SUL} + t_{cr} - \Delta t_{PE} \tag{12}$$

3.3 Intergreen time differences: optimisation potential of intergreen times

3.3.1 Introduction

Another question arising in the context of capacity impacts of signal change intervals is the question for the improvement potential. Since safety issues are not scrutinised here, only limited statements can be made on the feasibility of the *potential* improvements. However, this potential is the basis for the identification of research needs and promising measures.

The concept of the maximum improvement potential $\Delta C_{\mathbf{max}}$

The improvement potential, indicated by the difference $\Delta C_{\max}$ between *calculated capacity* C_{calc} and *maximum capacity* C_{max}, can be identified by analysing the different components of the intergreen calculation. Intergreen times not reflecting the effective traffic flow parameters (and in this context leading to longer intergreen times) are suboptimal. If a vehicle, for instance, clears with faster speed than assumed in the calculation procedure, the intergreen time will be longer than strictly necessary.

While safe intergreen times always have to allow for unusual vehicle behaviour (cf. Section 2.4), the maximum capacity gives a measure, how many vehicles could cross the intersection, if every change interval would be adjusted to the effectively clearing and entering vehicles of every cycle. Overall this leads to the use of *average* values instead of percentiles or extreme values for times, speeds, and distances (Section 3.4).

Significance of the maximum improvement potential

It is apparent that the maximum capacity is only a theoretical concept, underlining the maximum improvement *potential*. This potential is subject to safety requirements. As long as vehicles are steered by humans and no complete information on the relevant parameters of every single vehicle is available, the driver behaviour can only be estimated and not precisely be foreseen. Nevertheless, the maximum improvement potential gives an impression on the price we pay in terms of capacity for (at least supposedly) safe intergreen times.

Even more important: by analysing the improvement potential in detail, aspects of intergreen times leading to a particularly significant capacity deterioration can be identified and dealt with. As will be shown later on in this research, the capacity could sometimes be notably improved by slight changes of traffic regulations at an intersection (e.g. turning restrictions) or changes in the stage sequence or stage design.

Last but not least, the maximum capacity improvement potential can guide the safety related research towards a direction promising high gains in efficiency.

The role of a post-encroachment time (PET)

The maximum capacity is only restrained by the conflict areas. Each conflict area can only be used by one vehicle at a time. Consequently, maximum capacity is achieved if no headway between entering and clearing vehicle (post-encroachment time) at the determining conflict area remains.

As has been explained before, a post-encroachment has to be taken into account. The conflict area has to be cleared some time *before* the entering vehicle arrives (cf. Section 3.2.4). The known intergreen time calculation methods, however, do not take this consideration explicitly into account. The method described in ITE (1999) and detailed in Koonce et al. (2008) disregards the entering time and realises in this way a sufficient post-encroachment time. In FGSV (1992), the headway between the clearing and entering vehicles at the confllict area is not considered at all. Only calculation parameters on the safe side lead to a safety gap.

The magnitude of the minimum required (or 'safe') post-encroachment time is, in the first place, a safety issue. Some research addressed it (e.g. JAKOB 1980; TANG 2008). For the determination of the *maximum capacity* no post-encroachment time is assumed. In the capacity model, the post-encroachment time can easily be incorporated to analyse the impacts. Only for (to a certain degree) unpredictable driver behaviour this PET is required. Hence, it is part of the difference between the *maximum capacity* and the *achievable capacity*.

Interdependance of intergreen calculation method and maximum capacity

While the maximum capacity of an intersection is independant of the used intergreen times and their calculation method, the improvement potential (i.e. the difference between calculated and maximum capacity) is not. Here the improvement potential is derived from intergreen time differences and, consequently, the used intergreen times.

The optimisation potential has, thus, to be adjusted to the intergreen calculation method. While the different components of the improvement potential are introduced in a general way as to enable adjustments to any common calculation method, the application focus is laid on the German procedure according to the the German Guidelines for Traffic Signals (RiLSA).

Different ways of improvement

For three reasons an optimisation potential can be perceived:

- the parameters relevant for the minimum intergreen time differ from the parameters used in the calculation (e.g. the entering process is not considered)
- the parameters used in the calculation may differ from the values observable in reality
- the effective parameters may be influenced by the intersection and signal program design

The first two points are more of theoretical nature. The feasibility of improvements in this aspect has to be proven by safety related research. By analysing the *average* behaviour of drivers, safety margins are highlighted. Parameters with high safety margins are worth scrutiny, while those already near the mean values are not. The results of this analysis will be a good warrant for further research.

The last point is based upon the interrelation of parameters with the different situations (intersection layout, stage sequence etc.). From this analysis, direct conclusions can be drawn on possible improvements of the capacity, most likely without affecting the safety. These improvements, however, depend on the individual intersection layout and signal program. For the theoretical basis of the capacity model only the first two points are dealt with.

Possible ways to improve the intersection capacity are analysed for an example intersection and based upon the procedure introduced henceforth. The results will be presented in the conclusions of this thesis (Section 6.4).

The minimum intergreen time

The intergreen time leading to *maximum capacity* is the *minimum intergreen time* $t_{ig,min}$ calculated according to Eq. 13 with all parameters as effective ones (the index being left out for simplicity). The parameters are illustrated in Figure 4. The conflict point ist marked red.

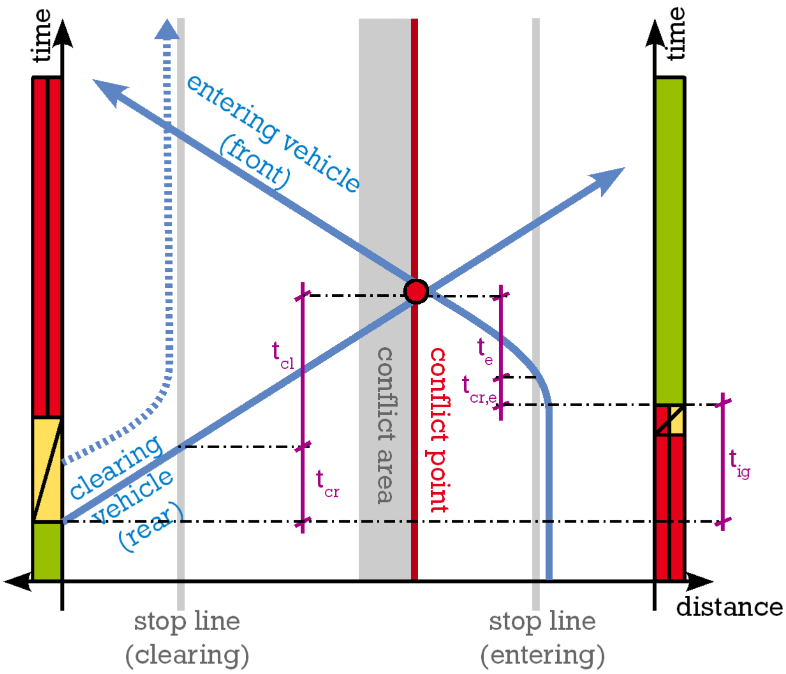


Figure 4: Elements of the minimum intergreen time
($t_{cr,e}$: crossing time of entering vehicles, t_e: entering time, t_{cr}: crossing time of clearing vehicles, t_{cl}: clearance time, t_{ig}: intergreen time)

$$t_{ig,min} = t_{cr} + t_{cl} - t_{cr,e} - t_e \tag{13}$$

where	$t_{ig,min}$	minimum intergreen time	(s)
	t_{cr}	effective crossing time (of clearing vehicle)	(s)
	t_{cl}	effective clearance time	(s)
	$t_{cr,e}$	effective crossing time of entering vehicle	(s)
	t_e	effective entering time	(s)

The difference between the *maximum capacity* and the *calculated capacity* is therefore based on differences of the underlying intergreen times, called *intergreen time differences* Δt_{ig}. Negative values of Δt_{ig} mean that the used intergreen time is longer than the effectively needed one (complete information supposed). For a safe intergreen time calculation, Δt_{ig} will always be negative.

The intergreen time equation consists of different times (cf. Eq. 13), which in their place are partly calculated using distances and speeds. In Koonce et al. (2008) the grade is taken into account in addition. The grade, however, is a constant value, which can be precisely determined. Only its influence on the acceleration behaviour may vary from the assumed one (influences on the different parameters are dealt with in Chapter 4).

Differences between the effective and assumed intergreen time can, hence, be seen in variations of

- time parameters (Section 3.3.2.1),
- distance parameters (Section 3.3.2.2 on the next page),
- speed parameters (Section 3.3.2.3 on page 44),
- safety margins (Section 3.3.3 on page 46), and
- occuring conflicts (Section 3.3.4 on page 46).

Deceleration rates are in some cases part of the intergreen time calculation equations. The reason for excluding them from the capacity model is elucidated in Section 3.3.2.4.

Time parameters calculated from speeds and distances

The entering and clearance time, if taken into account, are commonly calculated (namely in FGSV 1992) using speeds and distances (cf. p. 40 ff.). If they are not considered in the calculation method (as is the case, for instance, in ITE 1999), they appear in the capacity model as *time parameters*. Otherwise they are considered implicitely using *distance and speed parameters*.

To derive the entering and clearance times from speeds and distances not only follows the calculation method in the guidelines, but facilitates the empirical research. The entering time itself is difficult to measure with varying conflict points, while speeds are easier to capture. Furthermore, a more detailed insight into the reasons for deviating entering or clearance times is given. The parameters influencing the intergreen time difference directly as time parameters are the crossing times of entering and clearing vehicles.

3.3.2 Assumed vs. effective parameters

3.3.2.1 Variation in time parameters

Crossing time of entering vehicles

As has been justified in Section 3.2.2, the crossing time of entering vehicles with reference to the onset of green and the stop line is determining for the capacity model. While the commonly used starting response time is related to the indication of the signal change (which does not have to be the green signal) and which neglects the distance of the stopping position of the first vehicle in the waiting queue to the stop line, the crossing time incorporates these aspects.

The disadvantage of this concept is that the entering process has to be considered as a moving start (cf. p. 44 f.). Generically, the *crossing time difference of entering vehicles* $\Delta t_{cr,e}$ is the difference between the effective crossing time $t_{\mathrm{cr,e,eff}}$ and the assumed one $t_{\mathrm{cr,e}}$ (Eq. 14). Following the procedure in the German Guidelines for Traffic Signals (RiLSA) Δt_{cr} equals $t_{\mathrm{cr,e,eff}}$, because the starting response time is not considered. $t_{\mathrm{cr,e,eff}}$ can be negative, if the impending signal change is indicated to the drivers before the beginning of green (e.g. due to a yellow-and-red interval).

$$\Delta t_{\mathrm{cr,e}} = t_{\mathrm{cr,e,eff}} - t_{\mathrm{cr,e}} \tag{14}$$

Crossing time of clearing vehicles

The calculated crossing time t_{cr} takes the latest possible and legal crossing of the stop line into account. Most drivers, however, won't cross the stop line at exactly the latest possible time, which can be expressed by the difference of assumed from effective crossing time (Eq. 15).

$$\Delta t_{\mathrm{cr}} = t_{\mathrm{cr,eff}} - t_{\mathrm{cr}} \tag{15}$$

3.3.2.2 Variation in distance parameters

Differences of the effective and assumed clearing and entering distances Δl_{cl} and Δl_e can be attributed to two facts:

- The point used for the calculation of the clearing and entering distances is not the relevant conflict point (*systematic error*, $\Delta l_{\mathrm{cl/e,sys}}$).
- The effective conflict point depends on the vehicles' trajectories and, hence, varies from vehicle to vehicle (*random error*, $\Delta l_{\mathrm{cl/e,rand}}$).

Furthermore, the assumed vehicle length may differ from the effective one (Δl_{veh}).

Systematic error of clearing and entering distance

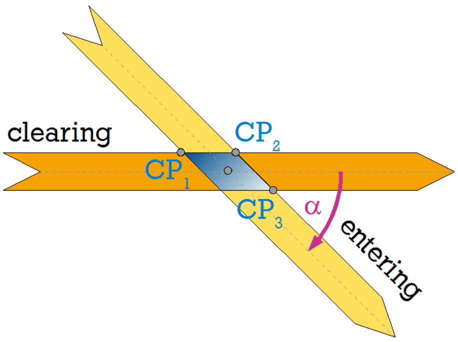
The systematic error depends on the intergreen calculation method. The main difference between these methods with respect to distances has to be seen in the definition of the *conflict area* and the resulting *conflict point* used for the determination of the entering and clearance distances. The prominent distinctions have been highlighted by Korda (1999) as follows:

- intersection of lane centre lines (e.g. in the German Guidelines for Traffic Signals (RiLSA))
- effective intersection of trajectories with effective vehicle dimensions
- intersection of lanes
- intersection of carriageways (e.g. in TRB 2000)

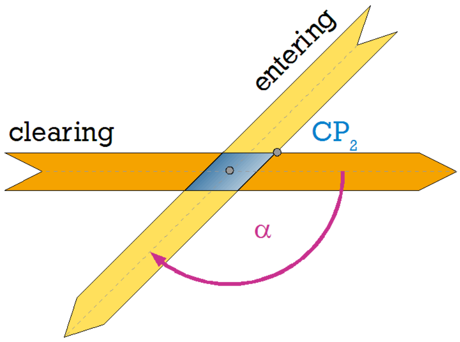
The distance differences are further scrutinised for the intergreen calculation according to the German Guidelines for Traffic Signals (RiLSA). Similar thoughts can be given to other calculation procedures. According to FGSV (1992) the intersection of the lane centre lines of the clearing and entering vehicles (for every possible conflict) is taken as the conflict point, as long as the two conflicting streams don't use the same exit lane. This follows the assumption that the conflict *area* can be reduced to a point.

In reality, the effective conflict point – i.e. the corner (or edge) of the conflict area, where the last possible intersection of clearing and entering trajectories takes place – is not in the centre, but on the edge of the conflict area. The effective entering or clearance distances differ, thus, from the assumed ones by the systematic distance error $\Delta l_{\mathrm{cl,sys}}/\Delta l_{\mathrm{e,sys}}$. The possible determining conflict points for different angles of intersection are illustrated in Figure 5.

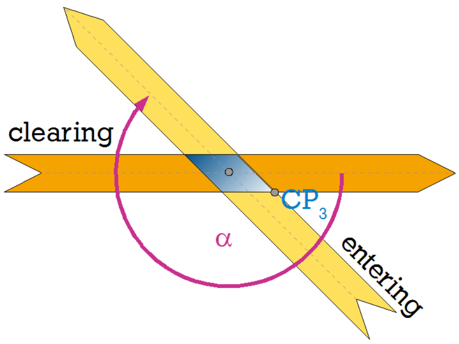
The distance error can be calculated based on the angle of intersection α of the trajectories of the last clearing and first entering vehicle (cf. Figure 5), the speed of the clearing and entering vehicle, and the lane widths. The different cases are shown in Table 1. The conditions have to be tested from top to bottom. The respective correction terms for the entering and clearance distance are specified in the last two columns. l_w denotes the lane width, the indices depicting e for entering vehicles and cl for clearing vehicles respectively.



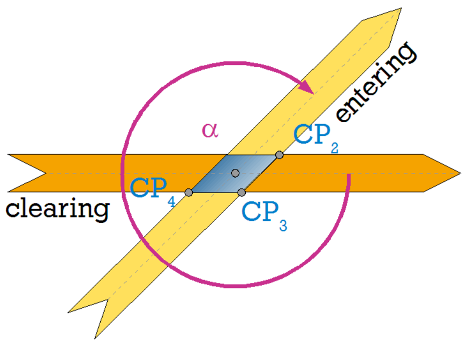
(a) Possible determining conflict points for $0° < \alpha < 90°$



(b) Determining conflict points for $90° < \alpha < 180°$



(c) Determining conflict point for $180° < \alpha < 270°$



(d) Possible determining conflict points for $270° < \alpha < 360°$

Figure 5: Possible determining conflict points (CP)

Angle	Conditions		CP	´1$_{\mathrm{cl,sys}}$	´1$_{\mathrm{e,sys}}$
$0° < \alpha < 90°$	1	$v_{\mathrm{cl}} > \frac{v_{\mathrm{e}}}{\cos\alpha}$	CP_1	$\frac{l_{\mathrm{w,e}}-l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$	$\frac{l_{\mathrm{w,e}}\cos\alpha-l_{\mathrm{w,cl}}}{2\sin\alpha}$
	2	$v_{\mathrm{cl}} > v_{\mathrm{e}}\cos\alpha$	CP_2	$\frac{l_{\mathrm{w,e}}+l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$	$\frac{-l_{\mathrm{w,e}}\cos\alpha-l_{\mathrm{w,cl}}}{2\sin\alpha}$
	3		CP_3	$\frac{-l_{\mathrm{w,e}}+l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$	$\frac{-l_{\mathrm{w,e}}\cos\alpha+l_{\mathrm{w,cl}}}{2\sin\alpha}$
$90°$	1		CP_2	$\frac{l_{\mathrm{w,e}}}{2}$	$\frac{-l_{\mathrm{w,cl}}}{2}$
$90° < \alpha < 180°$	1		CP_2	$\frac{-l_{\mathrm{w,e}}\cos\alpha+l_{\mathrm{w,cl}}}{2\sin\alpha}$	$\frac{-l_{\mathrm{w,e}}+l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$
$180° < \alpha < 270°$	1		CP_3	$\frac{l_{\mathrm{w,e}}+l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$	$\frac{l_{\mathrm{w,e}}\cos\alpha+l_{\mathrm{w,cl}}}{2\sin\alpha}$
$270°$	1		CP_3	$\frac{l_{\mathrm{w,e}}}{2}$	$\frac{-l_{\mathrm{w,cl}}}{2}$
$270° < \alpha < 360°$	1	$l_{\mathrm{w,cl}} < 2l_{\mathrm{w,e}}\cos\alpha$	CP_2	$\frac{-l_{\mathrm{w,e}}\cos\alpha-l_{\mathrm{w,cl}}}{2\sin\alpha}$	$\frac{-l_{\mathrm{w,e}}-l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$
	2	$v_{\mathrm{cl}} < v_{\mathrm{e}}\frac{l_{\mathrm{w,cl}}}{l_{\mathrm{w,cl}}-2l_{\mathrm{w,e}}\cos\alpha}$	CP_2	$\frac{-l_{\mathrm{w,e}}\cos\alpha-l_{\mathrm{w,cl}}}{2\sin\alpha}$	$\frac{-l_{\mathrm{w,e}}-l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$
	3	$v_{\mathrm{cl}} < \frac{v_{\mathrm{e}}}{\cos\alpha}$	CP_3	$\frac{l_{\mathrm{w,e}}\cos\alpha-l_{\mathrm{w,cl}}}{2\sin\alpha}$	$\frac{l_{\mathrm{w,e}}-l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$
	4		CP_4	$\frac{l_{\mathrm{w,e}}\cos\alpha+l_{\mathrm{w,cl}}}{2\sin\alpha}$	$\frac{l_{\mathrm{w,e}}+l_{\mathrm{w,cl}}\cos\alpha}{2\sin\alpha}$

Table 1: Calculation of systematic clearance and entering distance error

For the standard case of right angle intersections and a lane width of three metres, $\Delta l_{\mathrm{cl,sys}}$ and $\Delta l_{\mathrm{e,sys}}$ become

$$\Delta l_{\mathrm{cl,sys}} = -\Delta l_{\mathrm{e,sys}} = 1.5\,\mathrm{m}$$

In case of lanes intersecting at 45°, and the speeds according to the German Guidelines for Traffic Signals (RiLSA) ($v_e = 11.1\,{}^{\mathrm{m}}/_{\mathrm{s}}$, $v_{cl} = 10\,{}^{\mathrm{m}}/_{\mathrm{s}}$), the systematic errors become

$$\begin{aligned}
\Delta l_{\mathrm{cl,sys}} &= \frac{l_{\mathrm{w,e}} + l_{\mathrm{w,cl}} \cos\alpha}{2\sin\alpha} = \frac{3\,\mathrm{m}\cdot(\sqrt{2}+1)}{2} \approx \underline{\underline{3.6\,\mathrm{m}}} \\
\Delta l_{\mathrm{e,sys}} &= -\frac{l_{\mathrm{w,e}} \cos\alpha + l_{\mathrm{w,cl}}}{2\sin\alpha} = -\frac{3\,\mathrm{m}\cdot(1+\sqrt{2})}{2} \approx \underline{\underline{-3.6\,\mathrm{m}}}
\end{aligned}$$

The systematic error of curved trajectories can as an approximation be deduced to the cases above. For vehicles using the same exit, the German Guidelines for Traffic Signals (RiLSA) stipulates the use of the edges of the lanes, which leads to the effective conflict point.

The calculations above use the lane width for the determination of the effective conflict point. This width has to be corrected by the *random error*. Determining is not the edge of the lane, but the edge of the vehicles' trajectories (including the vehicle width). In this way, the total error can be deduced from the equations given in Table 1 with the *effective lane width* as is described below.

Random error of clearing and entering distance

The *random error* $\Delta l_{\mathrm{cl/e,var}}$ depends on the variation of vehicles' trajectories (Figure 6). Drivers, primarily of passenger cars, motorcycles, and bicycles, have more space at their disposal than they require, because lane width is commonly significantly greater than vehicle width. Futhermore, the lane borders may not be clear or are not respected. This applies in the first place to left turning traffic (for right side traffic as in Germany or the U.S.). Different trajectories of left turning traffic are illustrated in Figure 7.

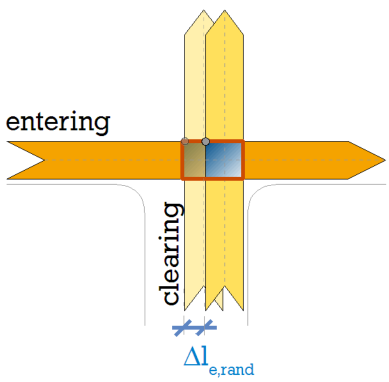


Figure 6: Variation of conflict point

For the present purpose, only variations tending to one side over the population of vehicles have an impact, because only average values are allowed for. A significant tendency is unlikely in the case of sensibly defined entering and clearing distances. Only where the entering and clearing distances are intentionally calculated with trajectories of vehicles deviating from the average expected ones, a difference as mentioned will play a role.

(a) Narrow turning radius **(b)** Wide turning radius

Figure 7: Different trajectories of left turning traffic

Because the same trajectories are used for entering and clearing process, the impact of the random error of distances will be insignificant. Even if variations can be observed, they tend to cancel each other out for the different entering and clearing processes. These variations will, however, have a noticable influence, if only intersection width is employed instead of real clearing or entering distances, as is common, for instance, in the United States.

It should be noted that the entering (and the clearance) distance are measured from the stop line. For the entering process, the crossing time of the vehicles takes the stop line distance l_{SL} already into account. Only if the crossing time of entering vehicles shall be replaced for some reason by the starting response time, the stop line distance has to be added to the entering time (cf. Section 3.2.2 and Section 3.3.2.1).

Vehicle dimension

The clearing vehicle has to clear the conflict area completely. In the intergreen calculation a constant vehicle length is taken into account which provides for a high safety. In reality vehicles may be shorter than this vehicle. The difference Δl_{veh} may lead to intergreen time differences. Very long vehicles (e.g. trams) are usually not considered in their full length, because they are clearly visible. Commonly, guidelines leave it to the engineer to set a reasonable length for those.

Some professionals (e.g. Tarnoff and Ordonez 2004a) have pointed out, that it is sufficient for a high safety level to make sure that the clearing vehicle is well visible to the entering vehicle, while others oppose this opinion (e.g. Parsonson et al. 1993). Following the former rationale, capacity gains by effectively short clearance distances will partly be compensated by interaction times (cf. Section 3.2.4). The calculated intergreen times may be too short to enable a long vehicle to clear the conflict area before the arrival of the entering vehicle. This will influence the driving behaviour of the entering vehicle. A green time difference Δt_{PE} will be the consequence. Consequently, the maximum capacity will most likely not differ significantly, whether vehicle length is considered in full or not.

Overall distance differences

The overall difference between assumed and effective clearing/entering distance is the sum of systematic error, random error, and, for the clearing distance, the variation of the vehicle length (Eq. 16

and Eq. 17). The sign of this distance depends on the effective conflict point as is given in Table 1.

$$\Delta l_{\mathrm{e}} = \Delta l_{\mathrm{e,sys}} + \Delta l_{\mathrm{e,var}} \tag{16}$$

$$\Delta l_{\mathrm{e}} = \Delta l_{\mathrm{cl,sys}} + \Delta l_{\mathrm{cl,var}} + \Delta l_{\mathrm{veh}} \tag{17}$$

3.3.2.3 Variation in speed parameters

Entering process

The entering process is in most cases an accelerated movement. Three situations can be distinguished (Figure 8):

- starting from full stop (Type 1)
- moving start (Type 2)
- moving start with deceleration and acceleration (Type 3)

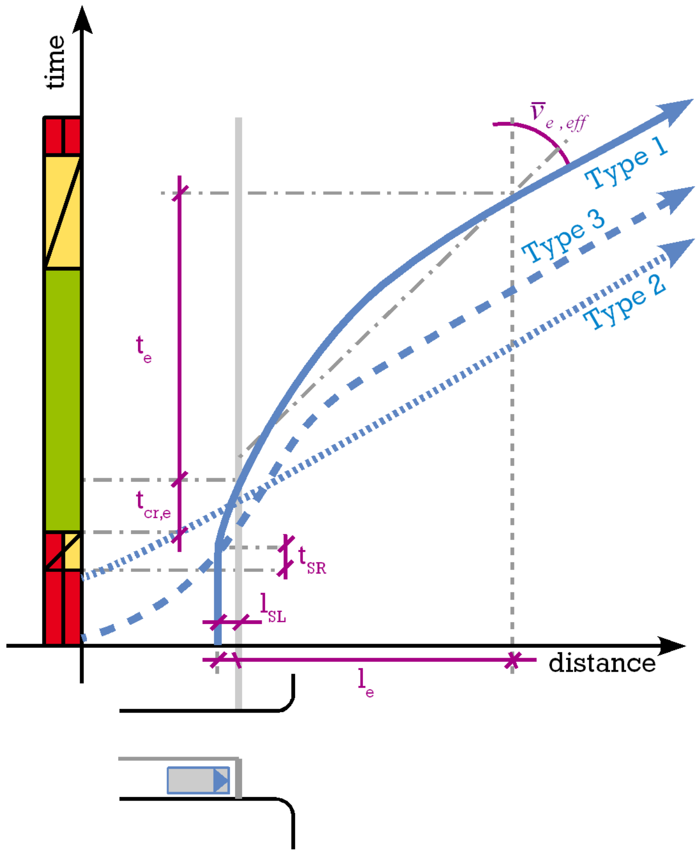


Figure 8: Parameters of the first entering vehicle
(t_{SR}: starting response time, $t_{\mathrm{cr,e}}$: crossing time of entering vehicle, t_{e}: entering time, l_{SL}: stop line distance, $\overline{v}_{\mathrm{e,eff}}$: equivalent entering speed, l_{e}: entering distance)

The moving start is defined by an arrival at the stop line with a speed $v_0 > 0\,\mathrm{m/s}$. Type 3 is a transitional type between type 1 and Type 2. The nearer the vehicle is to the stop line at the end of red, the more it has to decelerate. In the extreme, the vehicle will stop before the onset of green (Type 1). While a moving start (either in its most extreme form, Type 2, or as the moderate Type 3) may be relevant for safety considerations, it is a very rare exception. Particularly for non-coordinated and saturated approaches, the frequency of Types 2 and 3 can be neglected (cf. GLEUE 1973a). Though LIN and VIJAKUMAR (1988) recommend to regard Type 3 for the determination of the entering time, they do not justify empirically this recommendation. They still propose a linear function for the calculation of the entering time.

Non-coordinated approaches (Type 1)

The entering process of Type 1 is an accelerated movement. The entering time $t_{\mathrm{e,eff}}$ depends, thus, on the entering distance l_e, the acceleration a_e, and the maximum speed $v_{\max}$. Even for big intersections, vehicles will usually not accelerate to $v_{\max}$ before reaching the last conflict point. The entering time can be calculated, if the speed as a function of time or distance is known.[7]

If the entering speed is assumed constant (Type 2), as is the case in the German Guidelines for Traffic Signals (RiLSA), an *equivalent entering speed* $\overline{v}_{e,eff}$ has to be derived from the true accelerated movement for the entering distance to enable a direct comparison. This equivalent speed is assumed constant (as is the entering speed from the Guidelines) and leads to the same entering time for a given distance as the true entering movement. Following this procedure, the entering time difference can be calculated based on an *entering speed difference* Δv_e according to Eq. 18.

$$\Delta v_{\mathrm{e}} = \overline{v}_{\mathrm{e,eff}} - v_e \tag{18}$$

Coordinated approaches

If an approach is coordinated and vehicles commonly don't have to stop (Types 2 and 3 occur), the entering speed has to be analysed based on the quality of the coordination. In this situation, the crossing time of the first entering vehicle $t_{\mathrm{cr,e}}$ is of primary importance in addition to the entering speed. The entering speed will be a moving start. The equivalent entering speed $\overline{v}_{\mathrm{e,eff}}$ has to be derived not only depending on the entering distance, but also in relation to the crossing behaviour (crossing time and speed at the stop line).

Clearing process

For the clearing process a constant speed can commonly be assumed, with lower values for turning traffic. This speed may deviate from the actual behaviour leading to a speed difference Δv_{cl}. If accelerations can be observed between the stop line and the conflict point, an *equivalent clearance speed* $\overline{v}_{cl,eff}$ has to be defined following the same procedure as described for the entering speed (with the speed most likely being a decelerated movement). While for saturated conditions the last clearing vehicles cannot drive appreciably faster than the vehicles in front of them, for unsaturated conditions the clearance speed may be higher for vehicles clearing late (namely during yellow time; cf. GLEUE 1974; CHANG ET AL. 1985).

3.3.2.4 The role of deceleration rate and yellow time

Deceleration rates are commonly used to judge the potential braking behaviour of clearing drivers and to calculate the yellow time in a way that no dilemma zone exists. If the intergreen time depends directly on the yellow time, as, for instance, in the United States (ITE 1999), the deceleration rate impacts on the intergreen times. For the effective determination of the intergreen times, however, the breaking behaviour of vehicles is of no relevance. The last *clearing*, i.e. passing, vehicle sets the crossing time. While the crossing time is a parameter directly influencing the optimisation potential of intergreen

[7] If the speed is given as a function of distance only, the entering time has to be calculated incrementally.

times, the deceleration rate of clearing vehicles may only indirectly influence the crossing time. It is an influencing factor, but no model parameter.

3.3.3 Safety margins

If the intergreen calculation method introduces safety margins t_{saf} in terms of additional times, they have to be considered in the capacity model. The most common form of these safety margins so far are rounding differences. The German Guidelines for Traffic Signals (RiLSA), for instance, stipulate that all calculated intergreen times are rounded up to the full second. In this way, safety margins up to nearly one second materialise. These safety margins are, however, furtuitous as has been explained before.

Safety margins may be constant values. According to RiLSA they are specific to each conflict. According to the definition of the maximum capacity, safety margins are not required. Safety margins directly influence the intergreen time difference negatively.

3.3.4 Considered and effectively occuring conflicts

Conflict difference time

The intergreen interval of every signal group or every stage is always calculated for the conflict leading to the longest intergreen time. However, depending on the actual traffic this conflict doesn't occur during every change of stages. The difference between the intergreen time of the *effective* (i.e. actually occurring) conflict and the intergreen time of the *assumed* conflict may contribute to the total intergreen time difference.

This difference depends on the calculation procedure of the intergreen times. If only rough values are used which do not differ for the different conflicts of one stage change, no difference occurs. On the other hand, in this case the difference between assumed vs. effective parameters becomes significant (cf. Section 3.3.2). The difference between assumed and decisive conflict depends also on the control regime. For a stage based control, the longest intergreen interval of a change of stages is decisive. For a signal group based control, the longest intergreen interval of every signal group becomes decisive.

The difference between the determining intergreen time for a signal group (or stage) $t_{ig,s}$ and the intergreen time for the regarded conflict $t_{ig,c}$ is termed *conflict difference time* Δt_c (Eq. 19). Δt_c is always equal to zero or negative.

$$\Delta t_c = t_{ig,c} - t_{ig,s} \tag{19}$$

Movement sequences and conflicts

Following the German Guidelines for Traffic Signals (RiLSA), for every possible conflict an intergreen time has to be calculated. A conflict consists of a certain sequence of movements which are not compatible with each other at the same time at some area inside of the intersection (conflict area). A movement is defined by the stream (and hence a direction of traffic flow), and a vehicle type (in RiLSA, motorised vehicles, bicycles, and trams are distinguished). A movement sequence consists of a clearing movement and an entering movement. Commonly only the last clearing and first entering vehicle of every lane are considered.

3.3.5 Calculation of maximum capacity improvements

3.3.5.1 Methodology

The intergreen time differences vary among movement sequences. For each movement sequence an individual intergreen difference $\Delta t_{\mathrm{ig,m}}$ has to be calculated (cf. Section 3.3.5.2). While the potential movement sequences depend on the intersection layout and signal program only, the occuring ones may vary from cycle to cycle. Consequently the intergreen time difference relevant for a specific cycle depends on the movement sequences effectively occuring. For capacity estimates the average is of interest. The overall intergreen time difference for a signal group or a stage change (depending on the control regime) depends, thus, on the likeliness of each movement sequence to occur.

The individual intergreen time differences for movement sequences $\Delta t_{\mathrm{ig,m},i}$ are summarised to intergreen time differences for the respective lane combination $\Delta t_{\mathrm{ig,l}}$ based on the probability of the specific movement sequences of this lane combination p_i (cf. Section 3.3.5.3). A lane combination consists of a lane of the ending stage and a lane of the beginning stage. The intergreen times of all lane combinations $\Delta t_{\mathrm{ig,l},i}$ are further conflated to intergreen time differences for signal group combinations $\Delta t_{\mathrm{ig,s}}$ (cf. Section 3.3.5.4).

The intergreen time differences of all signal group combinations $\Delta t_{\mathrm{ig,s},i}$ (for signal group based control) are the basis for the determination of the maximum capacity improvement potential. The *intergreen time differences* have to be converted to *green time extensions* Δt_G to calculate the capacity improvement potential $\Delta C_{\max}$ (cf. Section 3.3.5.5).

> *To illustrate the methodology, an example intersection (Figure 9 on the next page) is taken for demonstration purposes. The texts refering to this example are indented and typed in italics.*

3.3.5.2 Intergreen time difference for movement sequences

With the different parameters explained in the sections before, the intergreen time difference for each movement sequence $\Delta t_{\mathrm{ig,m},i}$ can be calculated according to Eq. 20.

$$\Delta t_{\mathrm{ig,m},i} = \Delta t_{\mathrm{c}} - \Delta t_{\mathrm{cr,e}} - \Delta t_{\mathrm{e}} + \Delta t_{\mathrm{cr}} + \Delta t_{\mathrm{cl}} - t_{\mathrm{saf}} \tag{20}$$

with (cf. Appendix A.3)

$$\Delta t_{\mathrm{e}} = \frac{v_{\mathrm{e}}\Delta l_{\mathrm{e}} - \Delta v_{\mathrm{e}} l_{\mathrm{e}}}{v_{\mathrm{e}}^2 + v_{\mathrm{e}}\Delta v_{\mathrm{e}}} \tag{21}$$

$$\Delta t_{\mathrm{cl}} = \frac{v_{\mathrm{cl}}(\Delta l_{\mathrm{cl}} + \Delta l_{\mathrm{veh}}) - \Delta v_{\mathrm{cl}}(l_{\mathrm{cl}} + l_{\mathrm{veh}})}{v_{\mathrm{cl}}^2 + v_{\mathrm{cl}}\Delta v_{\mathrm{cl}}} \tag{22}$$

3.3.5.3 Intergreen time difference for lane combinations

List of movement sequences

On each lane combination (i.e. the combination of any lane of any ending stage together with any lane of any beginning stage of the respective stage change) a certain number of possible movement sequences can occur. A movement sequence is determined by the last clearing vehicle on the lane of the ending stage and the first entering vehicle on the lane of the beginning stage. Of relevance is the direction of flow of the two vehicles (defined by one out of twelve stream numbers, cf. Figure 9) and the vehicle

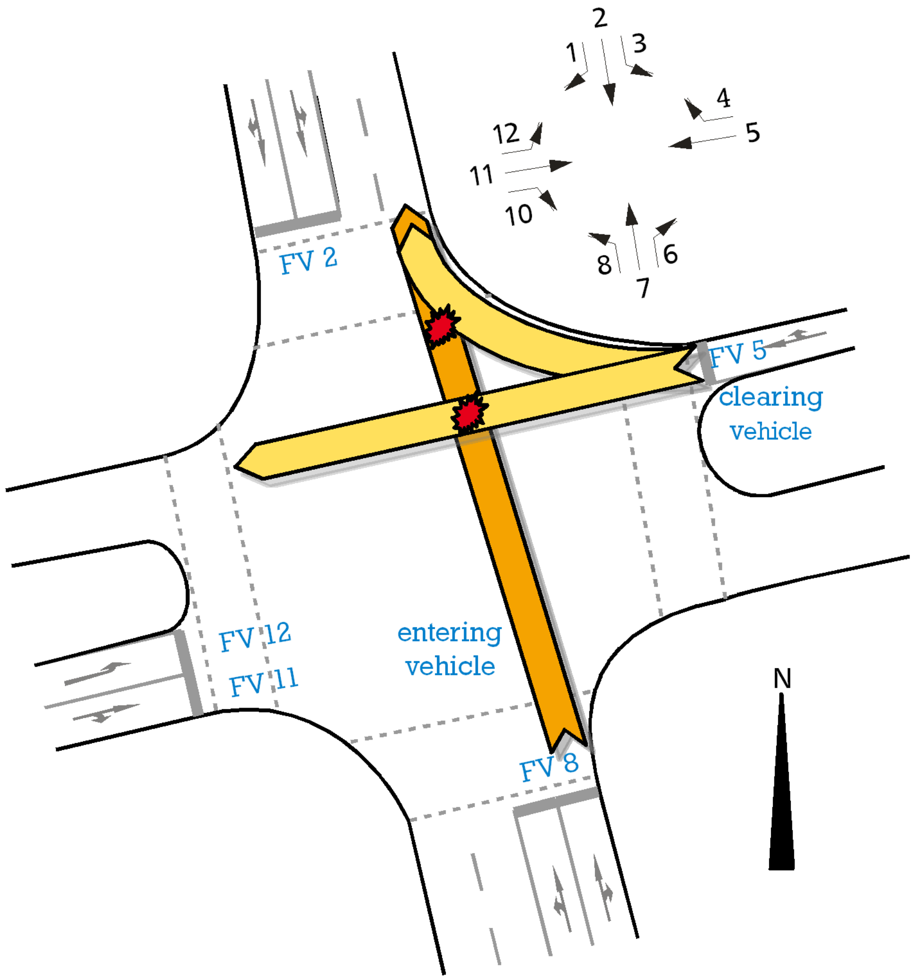


Figure 9: Example for different conflicts during the signal change

type (in RiLSA only the type of the clearing vehicle is considered). Only one of a number of possible movement sequences can occur during each cycle.

Since movement sequences can be compatible (e.g. two right turning streams), not all movement sequences are *conflicts*. For movement sequences not being conflicts, no intergreen time would be required. The intergreen time difference for this movement sequence will be at a maximum for the regarded lane combination. The higher the probability of movement sequences not being conflicts is, the higher the overall intergreen time difference of the lane combination will be.

> *The stage sequence for the example intersection Figure 9 is given in Figure 10. Here, the lane combination "Eastern lane (E) clearing" (signal group FV 5) and "Southern right lane (SR) entering" (signal group FV 8) is analysed.*
>
> *On both lanes, the vehicles may belong to one of two streams (right turning, 4/7, or through traffic, 5/8). For the clearing stream, a distinction according to the German Guidelines for Traffic Signals (RiLSA) is made between motorised vehicles (including heavy vehicles) and bicycles.*
>
> *All possible movement sequences are listed in Table 2 with the stream numbers from Figure 9. All movement sequences involving a right turning entering vehicle or a right turning clearing cyclist, the latter one assumed as being compatible with through vehicles, are no conflicts.*

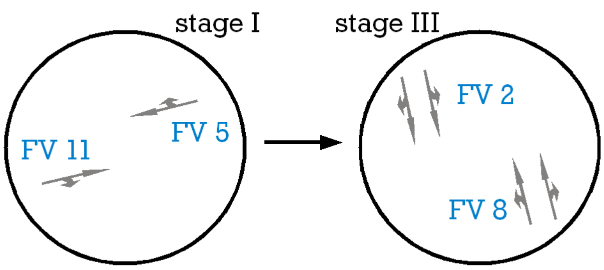


Figure 10: Part of the stage sequence for the example in Figure 9

Clearing stream	Entering stream	Clearing vehicle type	Conflict?
4	7	motorised	no
4	7	bike	no
4	8	motorised	yes
4	8	bike	no
5	7	motorised	no
5	7	bike	no
5	8	motorised	yes
5	8	bike	yes

Table 2: Movement sequences for lane combination E/SR at example intersection (Figure 9)

Out of eight movement sequences only three are conflicts. The less likely these movement sequences are, the more intergreen times effectively not required are part of the signal program (with respect to the regarded lane combination). This highlights the role of the assumed vs. effective conflicts.

The conflict tree

To systematically derive all possible movement sequences, a tree structure can be used. This tree structure is not only useful for the determination of the intergreen time differences, but also for the specification of the determining intergreen times themselves. Because for intergreen times only the conflicts are relevant, the tree structure will be called *conflict tree* henceforth.

Root of this conflict tree is a stage, the first branches lead to all signal groups which are part of this stage (Level 2). From every signal group a branch leads to every lane signalled by the respective signal group (Level 3). On the lanes different streams can be present (Level 4). The last level is presented by the vehicle types (namely bicycles and motorised vehicles for the German case; Level 5).

Such a tree is constructed for both the ending and the beginning stage of a stage change, the latter following RiLSA without Level 5 (vehicle types). These trees can be opposed to each other. The connection of the last elements of the ending stage tree with the last elements of the beginning stage tree depict the movement sequences. To all movement sequences an intergreen time difference $\Delta t_{\mathrm{ig,m}}$ can be assigned. Conflicts are highlighted by joining them with a connector.

The trees for the stage change shown in the example above (Figure 10) is rendered in Figure 11. All conflicts of the two lane combinations E/SR and WR/SL are highlighted.

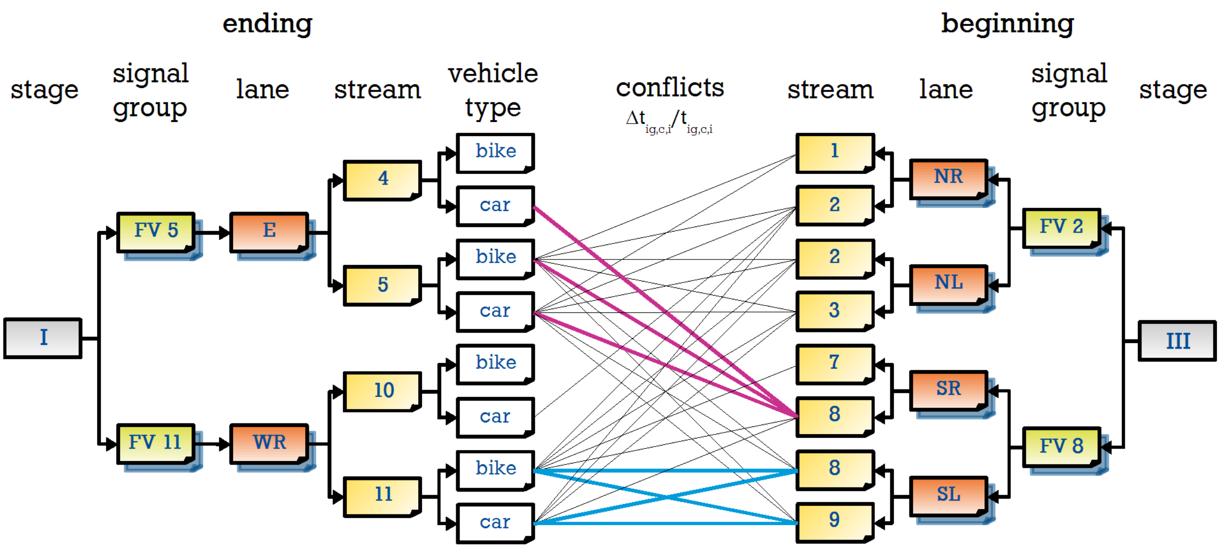


Figure 11: Conflict tree for the example from Figure 9

Likeliness of movement sequences

The intergreen time difference for a certain lane combination depends, as could be seen, on the probability of the different movement sequences. The probability of a last vehicle being of a certain type may depend on local conditions. If, for instance, most of the right turning vehicles on a certain lane arrive from one direction at an adjacent signalised intersection, the distribution of right turning vehicles is biased. It depends, in this case, on the signal program of the adjacent intersection.

However, as long as no cause is given to surmise such a connection, an equal distribution can be assumed. The vehicle type and type of movement depends, thus, only on the vehicle volumes of the respective movements (Eq. 23).

$$p_i = p(s_{cl}, s_e, vt_{cl}) = \frac{q_{s_{cl}} p_{vt_{cl}}}{\sum_j q_j} \frac{q_{s_e}}{\sum_k q_k} \tag{23}$$

where	p_i	probability of movement sequence i	(–)
	s_{cl}	stream number of clearing vehicle of movement sequence i	
	s_e	stream number of entering vehicle of movement sequence i	
	vt_{cl}	index depicting vehicle type of clearing vehicle of movement sequence i	
	$q_{s_{cl}}$	volume of stream s_{cl}	($^{veh}/_h$)
	$p_{vt_{cl}}$	share of vehicles of type vt_{cl} of stream s_{cl}	(–)
	q_{s_e}	volume of stream s_e	($^{veh}/_h$)
	$\sum_j$	sum over all streams on regarded lane of ending stage	($^{veh}/_h$)
	$\sum_k$	sum over all streams on regarded lane of beginning stage	($^{veh}/_h$)

From intergreen time differences of conflicts to intergreen time differences of lanes

Because always one of the possible movement sequences for a lane combination occurs, the sum of the probabilities for all movement sequences for a certain lane combination amounts to one. The product of the probability of each movement sequence and the intergreen time difference of the movement sequence summed up for all movement sequences of one lane combination amounts to the average *intergreen time difference of a lane combination* $\Delta t_{ig,l}$ (Eq. 24).

$$\Delta t_{\mathrm{ig,l}} = \sum_{m} \Delta t_{\mathrm{ig,m},i} p_i \tag{24}$$

By calculating the intergreen differences for the lane combinations, the conflict tree can be condensed as shown in Figure 12 with the connectors presenting lane combinations. To each of these lane combinations, an intergreen time difference $\Delta t_{\mathrm{ig,l}}$ can be assigned.

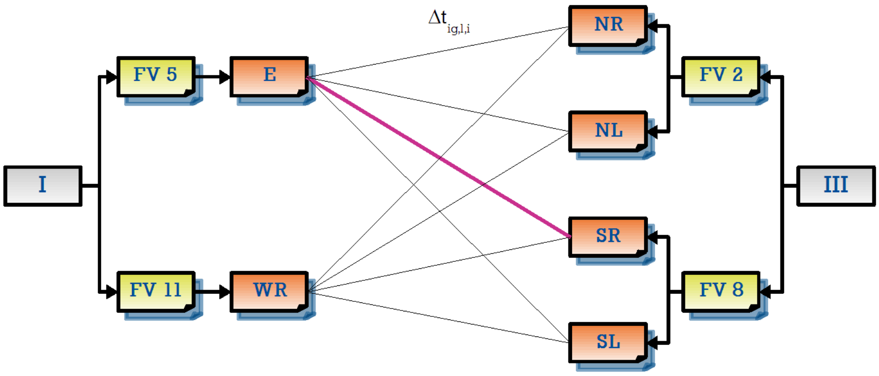


Figure 12: Conflict tree condensed to lane combinations for the example from Figure 9

The example from above is expanded with some exemplary data to illustrate the calculation of the intergreen difference for a lane combination $\Delta t_{ig,l}$*. Without loss of generality, the intergreen difference of the conflicts is reduced to the conflict difference time* Δt_c *for simplification. Table 3 gives the intergreen times for the movement sequences of the example together with the probability of each movement sequence to occur, and the resulting product of conflict difference time* $\Delta t_{c,i}$ *and the probability* p_i*. The sum of these products is the average (simplified) intergreen time difference for the lane combination "E/SR" (highlighted in Figure 12).*

movement sequence			**intergreen**		$\mathbf{{}'t_{c,i} = {}'t_{ig,m,i}}$	$\mathbf{p_i}$	$\mathbf{p_i \cdot {}'t_{ig,m,i}}$
clearing stream	entering stream	clearing vehicle type	$t_{\mathrm{ig,c},i}$	$t_{\mathrm{ig,s},i}$			
			(s)	(s)	(s)	(-)	(s)
4	7	any	0	5	5	0.03	0.15
4	8	car	5	5	0	0.10	0.00
5	7	any	0	5	5	0.17	0.85
5	8	bike	3	5	2	0.06	0.12
5	8	car	4	5	1	0.64	0.64
					$\sum$	**1.00**	**1.76**

Table 3: Example calculation of $\Delta t_{\mathrm{ig,l}}$ for lane combination "E/SR" at example intersection

Which of all lane combinations have to be considered depends on the signal program, namely the stages and their sequence. For every change of stages, all lane combinations have to be analysed, where one lane belongs to an ending signal group and one lane belongs to a beginning signal group (connectors between left and right conflict tree in Figure 12).

With the example intersection from Figure 9 and the change of stages shown in Figure 10, the following lane combinations have to be analysed:
E/NR, E/NL, E/SR, E/SL, WR/NR, WR/NL, WR/SR, WR/SL

3.3.5.4 Intergreen time difference for signal group combinations

The intergreen time differences for lane combinations have to be further accumulated according to signal groups. Since the signal program is based on signal groups, all lanes of a signal group are always signalled together. The intergreen time difference for a signal group combination (i.e. one signal group ending, another beginning) $\Delta t_{\text{ig,g}}$, thus, is the minimum of all intergreen time differences of all lane combinations relevant for the regarded signal group combination (Eq. 25).

$$\Delta t_{\text{ig,g}} = \min_{l} \Delta t_{\text{ig,l},i} \tag{25}$$

Following the example from above, the following signal group combinations have to be distinguished:
FV 5/FV 2, FV 5/FV 8, FV 11/FV 2, FV 11/FV 8

To illustrate the procedure, the conflict tree can futher be condensed (Figure 13). Now only the signal groups of the ending and beginning stage are connected with each other. To each of these connectors an intergreen time difference $\Delta t_{\text{ig,g}}$ can be assigned.

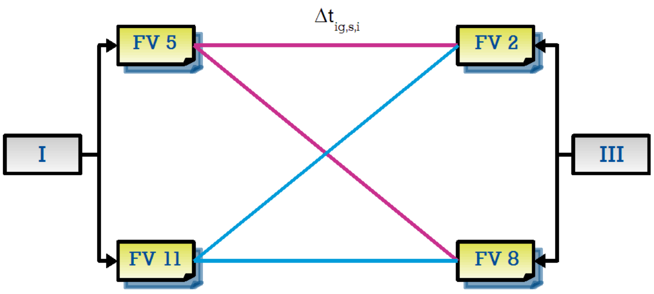


Figure 13: Conflict tree condensed to signal group combinations for the example from Figure 9

If in Figure 13 instead of the intergreen time *differences* the *intergreen times* are assigned to the connectors, the intergreen time matrix for the signal groups which are part of this conflict tree can directly be derived. Instead of the shortest intergreen time difference, the longest intergreen time would have to be inserted in Eq. 25 to accumulate intergreen times of lane combinations to intergreen times for signal group combinations.

3.3.5.5 Capacity improvement resulting from intergreen time differences

Connection between intergreen time differences and capacity improvements

The intergreen time differences for signal group combinations $\Delta t_{\text{ig,g}}$ define, how much the green times of the connected signal groups may be extended on average, without affecting the safety – a deterministic vehicle flow under complete information provided. The capacity improvements depend, hence, on these *green time extensions* Δt_G[8]. Because the capacity improvement related to one second of green time extension depends on the number of lanes and the saturation headway, the green time extensions Δt_{G} have to be related to specific lanes. This is achieved by assigning them to signal groups.

[8] Mind the difference to the green time extension e in TRB (2000)!

The green time extension of a signal group is confined by the minimum value of all intergreen time differences $\Delta t_{\text{ig,g}}$ connected to it. However, if the green time of one signal group is extended, all signal groups depending on this signal group (i.e. all signal groups connected to this signal group in the conflict tree) can only be extended by their respective maximum extension minus the extension Δt_{G} of the already extended signal group. Hence, an optimisation problem arises, because different combinations of extensions are possible.

> *If we assume that all intergreen time differences on signal group level $\Delta t_{ig,g}$ in the example from Figure 13 are four seconds. Signal group FV 2, for instance, can, thus, be extended by four seconds. However, signal groups FV 5 and FV 11 can then no longer be extended. The other way round, if signal group FV 5 is extended, FV 2 and FV 8 can no longer be extended. Hence, two solutions are possible (extension of the green times of signal groups FV 2 and FV 8, or extension of the green times of signal groups FV 5 and FV 11).*

Optimisation of green time extensions with the Simplex Algorithm

To solve this linear optimisation problem, the Simplex Algorithm can be used. The optimisation function depends on the aim of the optimisation. Commonly the green time split is correlated to the flow ratios on the respective lanes. The maximum capacity improvement for oversaturation on all lanes is achieved by considering the number of lanes of the signal groups and their saturation flow rates. For the former objective, the optimisation function can be formulated as in Eq. 26 with the weight of each signal group w_i according to Eq. 27. For the latter objective, the same optimisation function can be used, but with the weights according to Eq. 30.

$$\sum_{i=1}^{n_{\text{sg}}} \Delta t_{\text{G},i} w_i \quad \rightarrow \quad \text{max!} \tag{26}$$

$$w_i = \frac{b_i}{B} \tag{27}$$

$$b = \frac{q_i}{q_{\text{s},i}} = \frac{q_i \cdot h_{\text{s},i}}{3600\,\text{s/h}} \tag{28}$$

$$B = \sum b_i \tag{29}$$

Alternatively:

$$w_i = \frac{n_{\text{l},i}}{h_{\text{s},i}} \tag{30}$$

where	$\Delta t_{\text{G},i}$	theoretical extension of green time of signal group i	(s)
	w_i	weight of signal group i	(–)
	q_i	traffic flow of decisive lane for signal group i	(veh/h)
	$q_{\text{s},i}$	saturation flow rate of decisive lane for signal group i	(veh/h)
	b_i	flow ratio of decisive lane for signal group i	(–)
	B	sum of flow ratios of all decisive lanes	(–)
	$h_{\text{s},i}$	saturation headway of signal group i	(s)
	$n_{\text{l},i}$	number of lanes of signal group i	(–)

The contraints of the algorithm are given by the intergreen time differences (Eq. 31 for all signal group combinations ij).

$$\Delta t_{\text{G},i} + \Delta t_{\text{G},j} \leq \Delta t_{\text{ig,s},ij} \tag{31}$$

where	$\Delta t_{ig,s,ij}$	intergreen time difference between signal groups i and j	(s)
	n_{sg}	number of signal groups of the regarded stage change	(–)
	i, j	indices depicting the ending and beginning signal group respectively $i \leq n_{l,i} \wedge j \leq n_{l,j}$	

The Simplex Algorithm with the optimisation function and constraints as described here delivers the green time extensions $\Delta t_{G,i}$ for all signal groups leading to the maximum capacity improvement potential.

Calculation of the capacity improvement

After solving the problem with the Simplex Algorithm, an optimal solution for the extensions of the green time of all relevant signal groups $\Delta t_{G,i}$ is given. With this solution, the capacity improvement per hour ΔC_{max} can be calculated by converting the additional green times to capacity (Eq. 32).

$$\Delta C_{max} = \sum_{i=1}^{n_{sg}} \left(\Delta t_{G,i} \frac{n_{l,i}}{h_{s,i}} \frac{3600\,\mathrm{s}}{t_C} \right) \tag{32}$$

3.3.5.6 Further thoughts

Consideration of second vehicles

If the intergreen difference for the effective movement sequence is longer than the intergreen difference of the movement sequence consisting of the second entering and/or second clearing vehicle(s) plus their headway(s), the latter movement sequence becomes determining. This is particularly relevant, if the first movement sequence represents no conflict.

To give accurate results, the procedure described above has consequently to be extended to take the probability of the second movement sequence becoming determining into account.

Assume the first entering vehicle is a right turning bicycle followed by a through vehicle. The intergreen time difference of the first movement sequence may be 6 s and the intergreen difference for the second movement sequence may be 4 s. If now the headway between first and second vehicle h is less than $6\,\mathrm{s} - 4\,\mathrm{s} = 2\,\mathrm{s}$, the relevant intergreen time difference is $4\,\mathrm{s} + h$ instead of 6 s.

However, the higher complexity of the methodology will not lead to significant gains in accuracy. The low probability of the second movement sequence becoming determining and the inessential difference of intergreen differences justifies the neglect of these constellations.

Influence of improvements on driver behaviour

The described methodology delivers the maximum improvement potential of the intersection capacity by adjusted intergreen times. However, it cannot be precluded that an adjustment of the intergreen times will lead to a different average driver behaviour. Specifically the interaction times (Section 3.2.4) will change with shorter intergreen times. In this way, the effective improvement could differ from the calculated one. The adjustments of driver behaviour can either be forecasted by the capacity model calibrated for the changed circumstances. Due to the various influencing factors on driver behaviour, accurate results can only be obtained by longitudinal studies.

3.4 Consideration of the random character of traffic flow

From single values to averages

In the sections before, the traffic flow has been regarded on a microscopic level analysing the behaviour of single vehicles or travellers. In this way, the capacity of a single program cycle can be calculated. However, for capacity considerations average values have to be regarded. When looking on large numbers of cycles all the mentioned parameters become random variables. For the capacity calculations their average values or expectancies have to be used.

Variation and accuracy

The variation of the parameters has an impact on the achievable accuracy of the capacities derived from these parameters. Following the error propagation rules, this accuracy can be estimated based on the standard deviations of samples from the parameters. The skewness of the distributions give an indication on the likeliness of the results tending towards higher or lower values.

Possible substitutes for extensive surveys

To calculate the capacities of a specific intersection, not only the signal program has to be known, but all the aforementioned parameters have to be statistically analysed. Therefore, extensive surveys would be necessary. To reduce this effort, it is advisable to examine the influencing factors on all parameters.

If the influencing factors and their impacts are known, the capacities may be derived from observations at comparable intersections or parameters easily to be obtained. Chapter 4 deals with the explanation of the model application, part of which is the analysis of possible influencing factors. The surveys conducted as part of this research (Chapter 5) indicate some influences, even if they present no comprehensive and systematic assessment thereof.

4 The capacity model

4.1 Introduction

Chapter 3 explained the calculation of green time differences and intergreeen time differences. Green time differences are the basis for the calculation of the effective capacity, intergreen time differences lead to the capacity improvement potential and the maximum capacity. Safety margins reduce the maximum capacity and lead to the achievable capacity.

This chapter details the overall capacity model and its application (Section 4.2). To calibrate the model, the influencing factors on the model parameters have to be known. The calibration procedure is explained in Section 4.3. The model can be applied without this knowledge to a specific intersection. The results, however, are consequently only reliable for this one intersection. Chapter 5 presents the procedure and the results for such an application.

4.2 Model description

4.2.1 Introduction

The capacity model consists of four parts:

1. calculated capacity
2. effective capacity
3. maximum capacity
4. achievable capacity

All capacities are calculated from a number of parameters (model input parameters). The model input parameters are converted into model parameters. The model contains three groups of model parameters: green time differences, intergreen times, and intergreen time differences. The calculation of these parameters has been described in Chapter 3. Framework for the calculation of intergreen times and intergreen time differences is the conflict tree for the respective intersection and respective signal program.

Figure 14 on the next page illustrates the capacity model. The achievable capacity is left out for simplification. Input parameters are depicted in a blue box. The gray arrows guide to the calculated capacity, the yellow arrows highlight the steps to derive the effective capacity, and the red arrows lead to the improvement potential and the maximum capacity. Optional steps are depicted in light colors with a gray frame. The different steps of the capacity model are summarised in the following sections. Calculation procedures are introduced or referenced.

4.2.2 Calculated capacity

Framework for the capacity model is the intergreen time calculation procedure. In this procedure, intergreen times for specific movement sequences $t_{\mathrm{ig,m},i}$ (in this case only conflicts) are calculated from input parameters. These intergreen times for conflicts are converted into intergreen times for lane combinations $t_{\mathrm{ig,l},i}$ by taking the maximum intergreen time of all conflicts for the respective lane combination. The maximum intergreen time of all lane combinations is the decisive intergreen time for a signal group

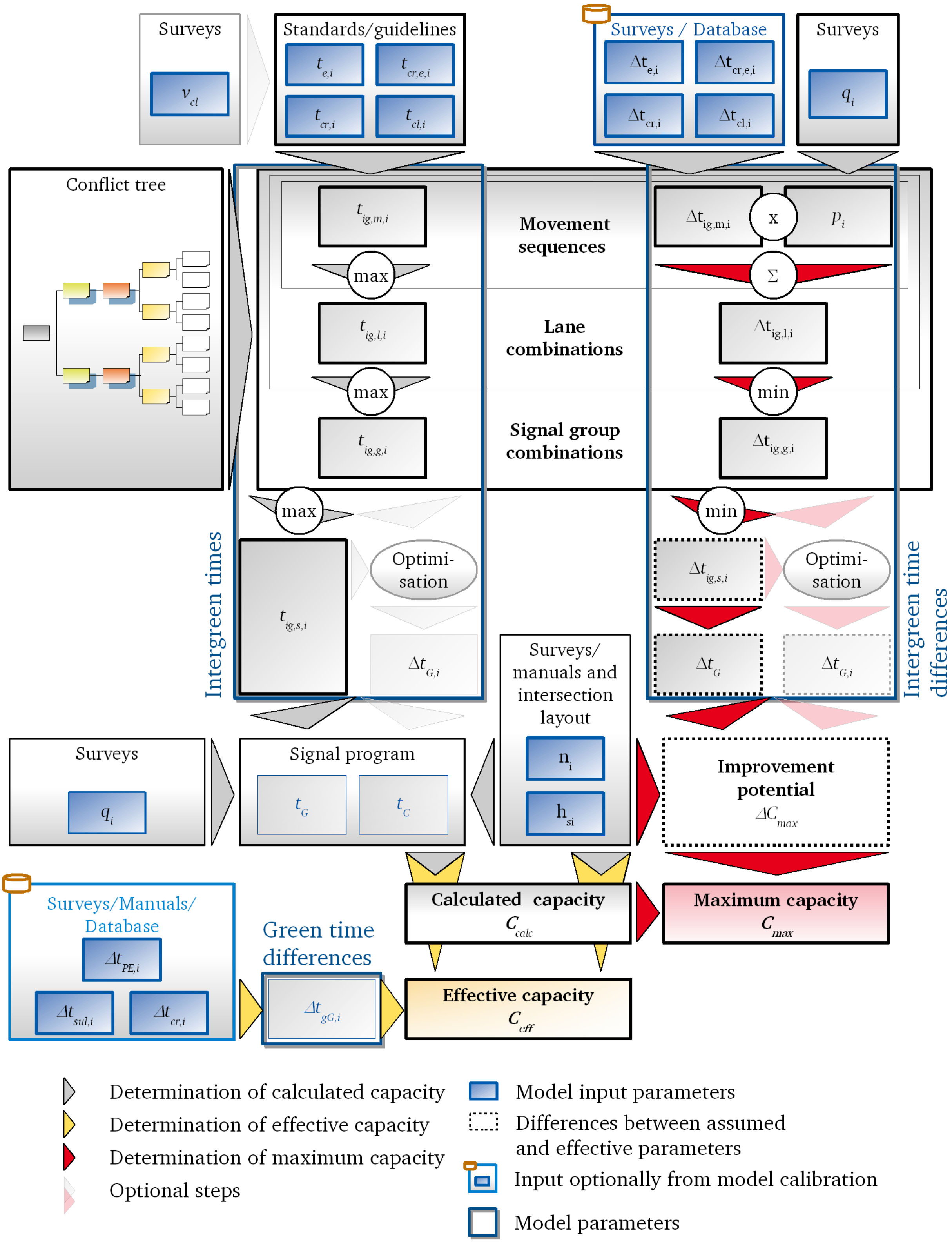


Figure 14: Illustration of the capacity model

combination $t_{\mathrm{ig,g},i}$. This procedure can be illustrated in the conflict tree: if all movements of the ending signal group under scrutiny are connected with all movements of the beginning signal group, to every connector depicting a conflict an intergreen time can be assigned. The maximum of these intergreen times is the decisive one for the analysed signal group combination.

For a stage based control, the maximum of the intergreen times of all signal group combinations $t_{\mathrm{ig,g},i}$ of a stage change becomes the decisive intergreen time $t_{\mathrm{ig,s},i}$ for this signal change interval. For a signal group based control, intergreen times have to be considered separately for all signal groups. The extension of the different signal groups is subject to constraints given by the intergreen times connected to the respective signal group. Signal groups can be displaced against each other in different ways. The procedure to optimise the offsets of the signal groups is explained in Section 3.3.5.5 on page 52. Since this is in optional procedure, it is depicted in light colours in Figure 14.

With the decisive intergreen time (and optional green time extensions for single signal groups) the signal program can be computed. The signal program delivers the green times and the cycle time, which are needed for the calculation of the capacity. The calculated capacity is determined according to Eq. 4 in connection with Eq. 6 (p. 29 f.) and the signalled green times $t_{\mathrm{G},i}$ instead of the effective green times $t_{\mathrm{g},i}$.

4.2.3 Effective capacity

If drivers make use of the transition times in a way that is not covered by common capacity calculation equations, the *effective capacity* may deviate from the calculated one (cf. Section 3.1). The difference can be expressed by the difference between signalled and effective green time Δt_{gG}.

To calculate the effective capacity C_{eff}, in addition to the saturation headway and the signal program parameters (namely cycle time and split) these green time differences have to be known. The effective capacity is determined in the same way as the calculated capacity, only the signalled green times $t_{\mathrm{G},i}$ are replaced by the effective green times $t_{\mathrm{g},i}$. The effective green times can be calculated by adding green time differences $\Delta t_{\mathrm{gG},i}$ to the signalled green times (cf. Section 3.2). Green time differences are calculated according to Eq. 12. The effective capacity (Eq. 33) is derived from Eq. 4, Eq. 6, and Eq. 12.

$$
\begin{aligned}
C_{\mathrm{eff}} &= \sum_{i}^{n_l} C_{\mathrm{l,eff},i} \\
&= \sum_{i}^{n_l} \frac{t_{\mathrm{obs}}}{h_{\mathrm{s},i}} \frac{t_{\mathrm{G},i} + \Delta t_{\mathrm{gG},i}}{t_{\mathrm{C}}} \\
&= \sum_{i}^{n_l} \frac{3600\,\mathrm{s/h}}{h_{\mathrm{s},i}} \frac{t_{\mathrm{G},i} + \left(-t_{\mathrm{SUL,i},i} + t_{\mathrm{cr}} - \Delta t_{\mathrm{PE},i}\right)}{t_{\mathrm{C}}}
\end{aligned}
\tag{33}
$$

where	C_{eff}	effective intersection capacity	(veh/h)
	$C_{\text{l,eff}}$	effective lane capacity	(veh/h)
	t_{obs}	observation time (commonly 3600 s)	(s)
	h_s	saturation headway	(s/veh)
	t_G	signalled green time	(s)
	Δt_{gG}	green time difference	(s)
	t_{C}	cycle time	(s)
	t_{SUL}	start-up lost times	(s)
	t_{cr}	crossing time of clearing vehicles	(s)
	Δt_{PE}	interaction time	(s)
	i	lane index	
	n_l	number of approach lanes	

All parameters are the mean values of random distributions which have to be obtained for all approach lanes under saturated conditions. The effective capacity calculated as in Eq. 33 is the *base* capacity (cf. Section 3.1.2.2). Due to right of way regulations of permitted streams, the final effective capacity can be lower.

4.2.4 Maximum capacity

If the behaviour of all vehicles approaching an intersection would be known in advance, the intergreen times could be reduced to minimum values. These values would ensure, that no vehicles collided with each other, i.e. all conflict areas are only used by one vehicle at a time, *and* the available time is used to a maximum potential. The *determining* conflict areas of each signal change interval would be occupied by the entering vehicle as soon as the clearing vehicle had left them. If this condition is achieved, no further improvement of the capacity concerning intergreen is possible.

The primary purpose of the capacity model is to highlight ways to improve the intersection capacity. This can best be achieved by determining the improvement potential on parameter level. The improvement potential is consequently not derived from the maximum capacity, but the other way round. All parameters influencing the calculated capacity and relating to intergreen times are empirically obtained. These empirical values are compared to the assumed values of the intergreen calculation procedure. The differences between the assumed and empirical values are converted into intergreen time differences.

Intergreen time differences are aggregated analogical to the intergreen times. The procedure differs only from two aspects. Instead of maximum values, minimum differences are decisive, because they result in maximum intergreen times (and, hence, in the decisive intergreen times). Furthermore, for intergreen time differences the likeliness of movement sequences has to be accounted for. Intergreen time differences of movement sequences $\Delta t_{\text{ig,m},i}$ are multiplied by the respective probability of their occurence p_i and summed up to give the intergreen time difference for a lane combination $\Delta t_{\text{ig,l},i}$. This procedure has been explained in detail in Section 3.3.

Intergreen times always occur between two signal groups (one ending, the other beginning). By curtailing an intergreen time, the green times of the two signal groups affected by this intergreen time can be extended *in total* by the intergreen time difference (i.e. only one green time can be extended by the full amount, or the intergreen time difference has to be split). The resulting *green time extensions* $\Delta t_{G,i}$ can eventually be converted to capacities.

Signal groups commonly depend not merely on one intergreen time. Only if all intergreen times relevant for a signal group during a signal change interval have an improvement potential, the respective signal group can be extended. The green time differences again depend on the control regime. For a stage based control, the minimum intergreen time difference of all signal group combinations of a stage change becomes determining and is converted into a green time extension. For a signal group based control, the signal groups have to be considered separately. As for the determination of the optimum intergreen times, this can be achieved by using a Simplex algorithm as has been detailed in Section 3.3.5.5. The optimisation process is again optional and, therefore, is depicted in light colors in Figure 14.

To calculate the maximum improvement potential, the following steps are required:

1. determine all potential movement sequences and their likeliness p_i
2. determine the intergreen time difference $\Delta t_{\mathrm{ig,m},i}$ for each of the movement sequences (Section 3.3.5.2)
3. determine the minimum possible intergreen time difference for each lane combination $\Delta t_{\mathrm{ig,l},i}$ (Section 3.3.5.3) and every signal group sequence $\Delta t_{\mathrm{ig,g},i}$ (Section 3.3.5.4)
4. assign the intergreen time differences to signal groups as green time extensions $\Delta t_{\mathrm{G},i}$ and calculate the maximum capacity improvement $\Delta C_{\max}$ (Section 3.3.5.5, Eq. 32)

4.2.5 Achievable capacity

The achievable capacity is less than the maximum capacity, because the assumptions made for the maximum capacity are not fulfilled in reality. Namely the knowledge of the vehicle trajectories and the occuring movements is incomplete. The intergreen time calculation has to be based on sample values of the input parameters. The random variation has to be taken into account by adding safety margins to the minimum intergreen times. The required magnitude of these safety margins depends on the desired safety level and the accuracy of the sample.

It is not in the focus of this research to analyse different methods to set these safety margins. As has been highlighted in Section 2.4, a variety of such methods are applied around the world. Commonly, the safety margins are not correlated directly to the calculated intergreen times or the underlying parameters. In Germany, for instance, the only explicit safety margins result from the rounding of the calculated intergreen time durations, and are, as such, arbitrary (cf. Section 3.3.3). ITE (1999) proposes to use percentiles of the empirically determined speed. All other parameters are only derived from experience and general surveys, or assumed not to vary.

For these reasons, the achievable capacity is not further scrutinised here. However, the capacity model offers the possibility to easily calculate the achievable capacity if the safety margins are known. The achievable capacity is calculated in the same way as the maximum capacity, but the intergreen time differences on lane level $t_{\mathrm{ig,l},i}$ are reduced by safety margins.

4.2.6 Input parameters

The input parameters are depicted in blue boxes in Figure 14 on page 58. These parameters can be obtained in different ways, depending on the desired accuracy of the model and the feasible effort. With the exception of the parameters needed for the intergreen time calculation, which are commonly prescribed by standards or proposed by manuals, all parameters can be obtained empirically by individual surveys. This leads to high effort, but delivers the most reliable results for a specific intersection.

Some parameters have to be determined individually due to their great impact on the model results and the high variation among intersections (*primary parameters*). Namely these primary parameters are the vehicle volumes and the number of approach lanes. Furthermore, the signal program is not only based on vehicle volumes and intergreen times. All other aspects influencing the signal program, as minimum required green times, constraints for the stage sequence, prescribed cycle time, etc., have been left out in Figure 14 for simplification.

Particularly for the remaining *secondary parameters* (crossing times, entering and clearance times, interaction times, start-up lost times, saturation headways) effort and desired accuracy have to be balanced. Some parameters don't vary significantly among intersections. It seems promising to analyse the factors influencing the parameters involved in the model, and, thus, being able to generalise certain aspects of the model. The model can, thus, be calibrated for well defined situations. This calibration process is described in Section 4.3.

The saturation headways can be either obtained analogical to the other secondary parameters, or they can be derived from manuals (e.g. HCM or HBS). It should be noted that the capacity model is highly sensitive to headways, because they are part of all equations leading to capacities, and they are needed to determine start-up lost times. The sensitivity to headways can be reduced by calibrating the start-up lost times and the saturation headway separately instead of calibrating the headways only. To derive saturation headways from standards or manuals means the application of a calibrated model.

The U.S. Highway Capacity Manual (HCM) assumes constant green time differences of zero seconds (signalled green time equals effective green time) for standard intersections, and slightly longer green time differences (signalled green time shorter than effective green time) for intersections under congested conditions (TRB 2000, p. 10-12 f.). This assumption represents a very general and therefore inaccurate calibration of the model for the effective capacity.

4.3 Model calibration procedure

4.3.1 Introduction

If the factors influencing the secondary input parameters of the model are known, only the characteristics of the influencing factors have to be obtained to forecast the input parameters. Because these influencing factors are usually more easily to be determined, the effort involved in the model application can be reduced.

The model calibration consist of two steps:

1. derivation of influencing factors
2. obtaining parameters for different situations with reference to the influencing factors

By either determining a correlation of certain influencing factors and input parameters of the model, or by clustering influencing factors according to similarities relating to the input parameters, a database can be established. This database contains values for the input parameters in relation to certain influencing factors. Instead of measuring input parameters individually for an intersection, this database can be used to obtain general parameter values. These parameter values obviously contain errors. The database should contain, therefore, not only average values, but also the accuracy of the parameters.

The procedure is illustrated in Figure 15. Surveys are used to discern influencing factors, to calibrate the model and, thus, fill the database. The results of surveys can also be directly fed into the model to generate results for a specific intersection (*individual application*). The values of ihfluencing factors together define a situation. This process is expanded upon in Section 4.3.2. Because the database contains the intergreen time differences, the principles and values to calculate the intergreen times have to be stored in the database, too. While in Figure 15 only the entering and clearance *times* are depicted, they may be split into speeds and distances as has been explained in Section 3.3.1.

Here only the applicability of the calibration procedure is shown. For a statistically well backed cluster analysis comprehensive data would have to be gathered. In Chapter 5 indications are given on the variability of the parameters and possible influencing factors.

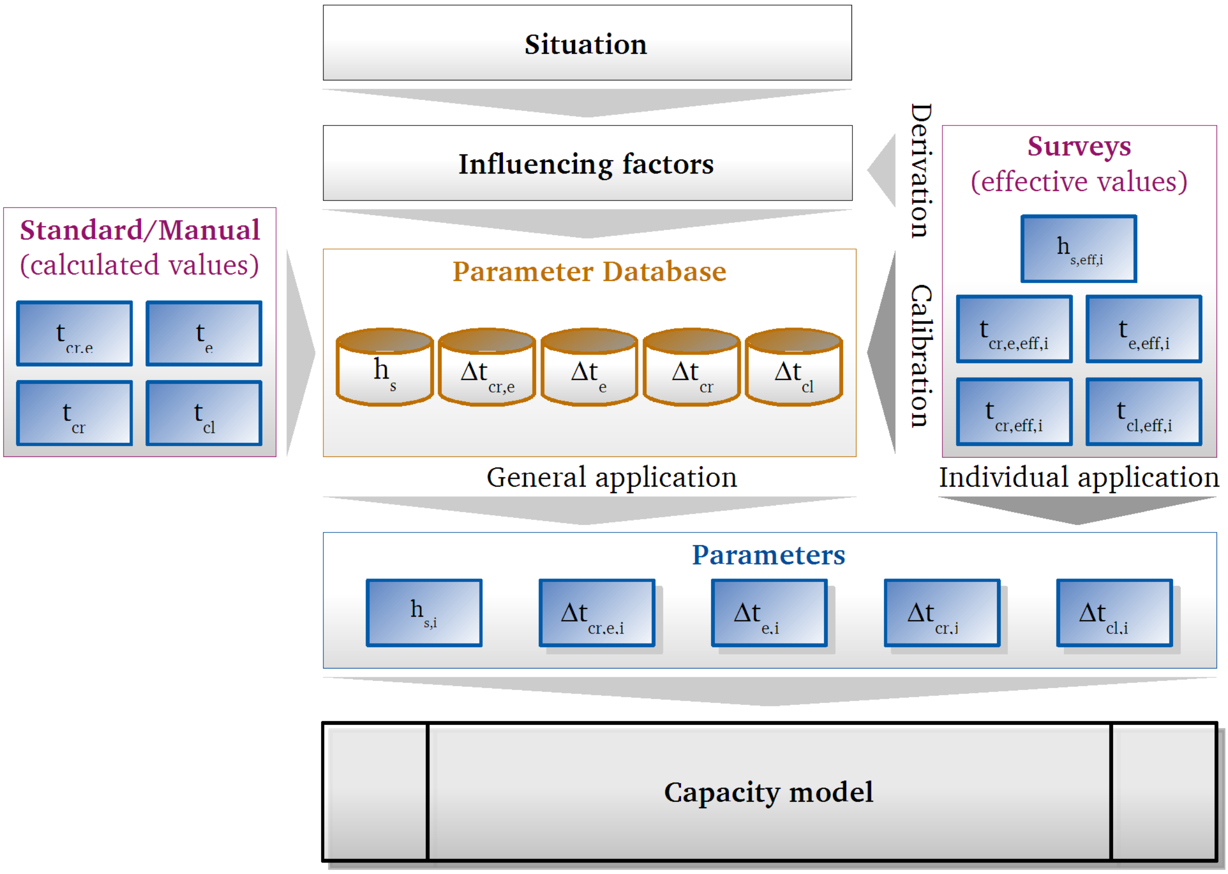


Figure 15: Illustration of the model calibration procedure

Section 4.3 gives an overview on the possible factors influencing the driver behaviour at signalised intersections. Some of these factors apparently have no significant influence or cannot be determined in situ. Some difficult to measure factors can be judged indirectly. Aim has to be to ascertain the factors of preeminent importance, and, if those factors are difficult to gauge, to work out parameters correlating with them. The compendium in the following section is backed by a literature review (see also Section 2.3). This chapter is, thus, only the basis for research enhanced by extensive surveys.

4.3.2 Topology of influencing factors

The parameters needed for the capacity model are *individual* parameters with respect to vehicles, depending on the *vehicle properties* and the *driver behaviour* at specific intersections. The driver behaviour depends on the respective *abilities*, the driver *disposition* at the time of approaching an intersection, and the local *situation* at the intersection. These influences are illustrated in Figure 16 on the following page.

The disposition and abilities of drivers approaching an intersection cannot be measured in situ. However, factors helping to forecast a probable average behaviour can be identified. Moreover, the vehicles' properties are only partially measurable. Some parameters influencing the model parameters have to be derived (e.g. the possible acceleration depends on the vehicle type).

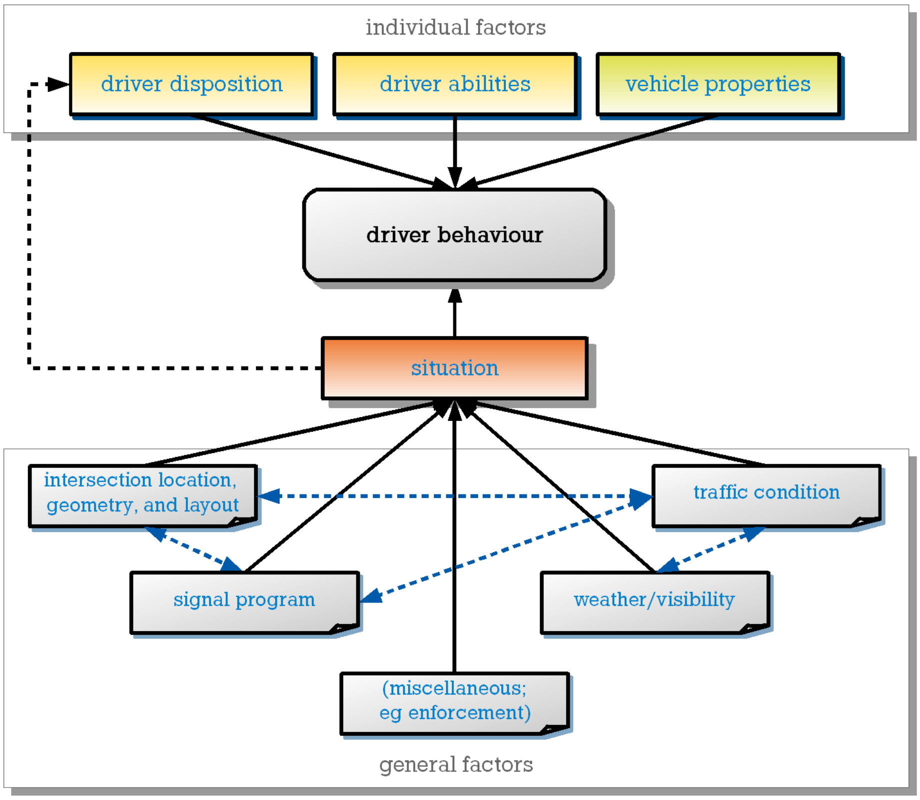


Figure 16: Factors influencing the individual driver behaviour at a signalized intersection

An important factor influencing the driver behaviour is the local *situation*. The situation is described by the intersection geometry and layout, the signal program, the weather and visibility, the traffic condition, and possible special constellations like the presence of traffic enforcement. These factors – some of them being static, some dynamic – can be determined in situ.

Even if the abilities and the disposition as well as the vehicle properties of a single approaching vehicle is fortuitous, the average is related to the local situation. Hence, the local situation bears the prominent role in clustering intersections in the present context. Furthermore, the parameters used for the clustering have to be *general* parameters (as opposed to *individual* parameters), preferably available for all intersections without the need for particular surveys. The parameters describing the local situation are general factors, while the parameters describing disposition and abilities of drivers as well as the vehicle parameters are individual factors. The following sections analyse the mentioned parameters in detail, leading to a conclusion for a clustering methodology.

4.3.3 Individual factors

4.3.3.1 Vehicle parameters

The most prominent vehicle properties are

- vehicle size (length, width, height) and turning radius,
- engine (maximum acceleration and deceleration rate, speed),
- driver assistance systems, and
- tyres (behaviour on slippery road),

of which only the vehicle size and speed can be easily acquired by measurement devices. The acceleration behaviour is strongly related to the vehicle type. Even if individual passenger cars show quite different acceleration potential, lorries will differ significantly from this behaviour. Furthermore, drivers rarely use the abilities of their cars to the full potential. In this way the actual acceleration depends mostly on the driver himself[9]. Apparently the vehicle type judged by the size of the vehicle is the most easily to be obtained parameter, from which other parameters have to be derived.

Lu (1984) not only revealed a significant influence of the vehicle size on the discharge headways of entering vehicles and, thus, on the start-up lost times. He pointed out the changes of the vehicle fleet over the time and the entailing changes of discharge headways. The vehicle fleet, thus, should be regularly checked for changes, if general factors are derived from the vehicle type.

4.3.3.2 Disposition

Driver disposition denotes the current circumstances, under which a driver steers his car, and the emotions involved. Part of the dispositon of a driver is embedded in his temper (e.g. the aggressiveness of the driving), part depends primarily on the circumstances, like the hurriedness.

Since the disposition cannot be directly observed, general factors influencing the majority of drivers, are the only way to derive some idea of the disposition. Such factors are the number of commuters or leisure travellers, or the saturation degree. Furthermore, the time (peak vs. off-peak, weekday vs. weekend) and the location of the intersection can be indicators for certain driver disposition, because the number of commuters can be partly derived from the time and location. Commuters, for instance, tend to be more stressed and, therefore, impatient than leisure travellers. This will influence their crossing behaviour at the onset of yellow, the entering behaviour etc.

The time of day has also a direct influence on driver disposition. Gleue (1974), for instance, showed a correlation of time and perception-reaction times (slowest perception-reaction times on Sundays; longer ones during the day than in the evening or in the morning). Differences of driving culture among countries were illustratively shown for Germany and Japan by Tang and Nakamura (2007a).

Hence, depending on local knowledge, intersections should be distinguished according to traffic characteristics or, indirectly concluding, according to location (central business district, urban fringe, rural area etc.). The disposition of the average driver will, moreover, change according to the time (season, day of the week, time of day etc.). The situation at adjacent intersections will also influence the driver diposition, too. The transition times and intergreen times used in the surrounding area should, therefore, be considered.

Concluding the driver disposition has to be derived from *general factors*, which are described further down.

4.3.3.3 Driver abilities

Driver abilities – or driver skills – directly influence the entering and clearing behaviour. Starting response times and crossing times will depend largely on the driver abilities in addition to the disposition. Local knowledge is of major importance for the driving behaviour. Of increasing importance becomes the age of drivers (compare Hallmark and Mueller 2004). Redshaw (2001) discussed the importance of driver skills in traffic engineering and planning in general. However, driver abilities will be as difficult to derive as driver disposition. Possible indirect factors have been mentioned in Section 4.3.3.2. As driver disposition, driver abilities have to be derived from *general factors*.

[9] Compare to the results of speed measurements conducted for this research in Section 5.3.4 and Appendix A.4.

4.3.4 General factors

The parameters describing the local situation at a particular intersection are general factors. They are irrespective of individual vehicles. Apart from exceptional and rare parameters (like enforcement), four main categories of paramters describing the local situation can be identified:

- intersection location, geometry and layout
- signal program
- traffic condition
- weather and visibility

Split into the different aspects of these categories, a magnitude of possible influences can be discerned. Comprehensive surveys and observations are needed to point out the most important independent ones. Here only a short summary of the most obvious factors is given.

4.3.4.1 Intersection location, geometry, and layout

Appreciable research dealt with the correlation of certain aspects of driver behaviour at signalised intersections with intersection location, geometry, and layout. Aspects scrutinized are

- number of approach lanes (divided into through lanes, turning lanes, mixed lanes) and intersection width (or number of crossing lanes)
- geography/type of area
- speed limit
- position of stop line relative to curb line
- grade
- parking situation, lighting and visibility of crossing traffic
- characteristics of turning traffic (trajectories, right turn on red, etc.)

Liu et al. (2007) highlight the ample factors influencing stopping behaviour and speed performance including the number of through lanes and crossing lanes, intersection geometry, and average speed on the approaches. McMahon et al. (1997) showed a significant influence of the number of through lanes on the saturation flow rate. They state that geographical differences are of minor concern as compared to the intersection layout. Wortman et al. (1985) reported on lower response times for downgrade. Rodegerdts et al. (2004) stress the important influence of grade on the clearing behaviour. For entering processes mainly upgrade seems to have a significant influence (Hoffmann and Nielsen 1994). The influence of street side parking is, for instance, mentioned by Köll et al. (2001).

Long (2005) found high variations of start-up lost time and vehicle position relative to the stop line among different sites. They conjecture a connection with stop line position relative to the curb line, trajectories of crossing turning traffic, red turn on red, and type of area among others. The importance of the line of sight was highlighted by Jourdain (1988), at least for intersections with fast approaching traffic. Various influences of intersection characteristics on headways and saturation flow have been tested by Hoffmann and Nielsen (1994).

Due to the multiple influencing factors partly correlating to each other, it is of preeminent importance to analyse the interrelation of the influencing factors derived from the intersection location, geometry, and layouot and the model input parameters by means of statistically sound methods. If only few factors are scrutinised, bias can easily introduced into the model.

4.3.4.2 Signal program

The signal program is defined by a number of factors. The most important ones are

- control regime
  - pretimed vs. traffic actuated
  - stage based vs. signal group based
  - coordinated vs. non-coordinated
- timing parameters
  - cycle time
  - split
  - transition times
- stages
  - number of stages
  - stage sequence (e.g. lead vs. lag turning stages)
  - stage design

The magnitude of possible permutations requires, as has been mentioned before, comprehensive empirical research, to spot the most prominent factors. It can be assumed that timing parameters have greater influence than the stage settings and even more the control regime. Moreover, the stage setting and control regime commonly have a direct influence on the timing parameters. Timing parameters should, hence, be of major concern.

Part of the timing parameters can, furthermore, be the procedure to determine them. The predictability of certain parts of the signal program (namely transition times and intergreen times) for the drivers will have an impact. Uniform yellow times, for instance, will result in different driver behaviour from differing yellow times at adjacent intersections.

4.3.4.3 Traffic condition

Apart from the obvious influence of the vehicle mix, turning ratios, and the saturation degree (more general: demand/supply ratio) on the performance of an intersection, some other factors relating to the traffic condition seem to have an impact on the driver behaviour, too.

The peak hour factor can have a pivotal influence on traffic flow at signalised intersections (GILBERT 1984). LIN and VIJAKUMAR (1988) define vehicle supply as the frequency of vehicles present within 5 s of travel time from the stop line at the end of green, which may deliver different results than simply using vehicle volumes. LIU ET AL. (2007) reported their observations of driver behaviour in response to the yellow signal. Traffic volume was among the influencing factors. Also LONG (2005) states an influence of traffic volumes on driver behaviour. That heavy vehicles are disproportionately more often determining for the clearing process than passenger cars is highlighted by GLEUE (1974).

Consequently, vehicle volumes, distinguished for vehicle types and movements, the saturation degree, and the peak hour factor should always be tested for correlation with the model input parameters.

4.3.4.4 Weather and visibility

Weather has a significant influence on driver behaviour. Particularly the precipitation is the decisive factor. Generally, speeds and accelerations tend to be less in bad weather (cf. e.g. Wortman et al. 1985). Due to changed road surfaces, the influence of weather can be inert. Commonly advert weather situations are excluded from surveys. As long as these situations are the exception and affect the traffic system only rarely this is acceptable. Furthermore, the automatic detection of weather conditions is still precarious. Visibility is influenced by the lighting conditions (due to daylight and street lights) and by weather (precipitation, fog etc.). It can also adversely affected by poor air conditions (dust).

Whether weather and visibility are used as clustering variables depends on the likeliness and predictability of particular conditions and their impacts. Commonly, signal control can neither be reliably adjusted to weather conditions, nor are these situations frequently determining for the peak hour of traffic.

4.3.5 Interdependencies and indirect factors

Many of the mentioned factors are not independent from each other. Above all the driver disposition depends on the situation, and therefore on the general factors. Only for this dependancy, driver disposition can be judged at specific intersections. The interdependencies are highlighted in Figure 16 by dashed lines.

In addition, some factors can be estimated by observing indirectly influencing factors. These indirect factors may not have a causal influence on the driver behaviour, but a sufficient correlation. The preeminent indirect factor is the time. Driver behaviour commonly differs between morning and evening, work days and holidays, summer and winter. It is appropriate to derive other factors (e.g. the share of commuters during the peak hours) from the time.

Time has different scales, all of which have to be considered. Roughly spoken, the smaller the scale, the more important the factor time becomes:

- time of day (hours)
- day of the week (days)
- holidays, bank holidays (days/weeks)
- season (months)

4.3.6 Summary of the model calibration procedure and outlook

The process of generalising the characteristics of intersections with respect to the driver behaviour during the signal change is based on three steps:

1. Identify influencing factors or indirect parameters correlating with them.
2. Cluster intersections according to these factors, or derive correlations between factors and input parameters for the model.
3. Derive mean values and standard errors for all input parameters.

Following these steps, intersections can then be associated with a certain cluster. The input parameters associated with this cluster can be taken from the database. Comprehensive surveys on site are, thus, avoided.

The first step requires extensive data. However, based on the overview of possible influencing factors in Section 4.3.3 and Section 4.3.4, the most crucial and feasible ones are listed below. These factors have to be analysed with respect to influences on the model input parameters.

- area (central business district, urban fringe etc.)
- number and type of approach lanes, intersection width
- speed limit
- signal timing (cycle time, green split)
- coordination
- vehicle volumes according to vehicle types, lane, and position in queue
- saturation degree and peak hour factor
- time (hour, day of the weak)

Additional parameters for more detailed research can be:

- grade
- turning traffic layout and regulation (separate lanes, permitted/protected, lead/lag time, right turn on red)
- stage settings
- time (season, special days)

The latter parameters should be at least constant or similar for surveys only focusing on the first mentioned parameters.

If adverse weather conditions frequently occur, weather and visibility will be an important factor, too. Moreover, the situation at surrounding intersections should be taken into account. Apart from a coordination, the yellow times (same or different) and intergreen time calculation procedure could have an impact on the driver behaviour. The predictability of the signal program for the drivers is partly determined by these factors.

5 Empirical research and exemplative model application

5.1 Introduction

5.1.1 Motivation for empirical research

In the last chapters, the influence of intergreen times on the capacity of signalised intersections was theoretically examined. The different processes during the signal change intervals leading to changes of the effective capacity have been described. A model to evaluate the effective capacity and the improvement potential of the capacity has been introduced.

While this thesis emphasises on the methodological and theoretical background of the capacity analysis with respect to intergreen times, it is important to know about the approximate quantitative importance of the different processes for the intersection capacity, and to know about possible influencing factors. Quantitative data and indications on influencing factors are a good basis to identify further research needs. By the model application, furthermore, the feasibility is proven. Moreover, it has been mentioned before that the analysis of the traffic flow during the change of stages has to be based on observations. It follows that empirical research is an important part of the presented research.

5.1.2 Aims of the empirical research

Based on the motivation for empirical research three aims of surveys as part of this research can be identified:

- obtain data for the application of the model,
- collect information on possible dependencies of the model variables as a first step towards a calibration of the model, and
- get qualitative information on the traffic flow during the signal change.

Following the aims of the empirical research, a suitable survey methodology has to be identified. Three aspects are of interest:

- Which parameters?
- In what precision?
- Of which significance?

While the first question leads to the general survey characteristics, the second limits the choice of suitable devices and methodologies. The last question is aimed at the sample size. All three aspects are dealt with in Section 5.2. This section covers the assessment of possible survey techniques and of the devices required for them, it describes the chosen survey layout, and it describes the realisation. Section 5.3 presents the data obtained. The capacity model is applied by feeding this data into the model. This application to an example intersection is described in Section 5.4. The accuracy of the model application is analysed by using error propagation methodologies.

5.2 Survey preparation and realisation

5.2.1 Survey requirements

5.2.1.1 Required parameters

The parameters needed as model input for the calculation of the effective and maximum capacity are summarised in Table 4. Some parameters are unique for the assessed signal program (P). Others differ among stages (S) or lanes (L). The most diverse parameters are most of the parameters needed for the calculation of the intergreen time difference. They have to be collected for every movement sequence (M). On which level the parameters are valid is denoted in Table 4 by a capital letter. Optional parameters (like speed differences) or parameters only indirectly needed to derive a model variable (like the cumulated headway difference) are separated from the original model parameters.

t_G	green time	S
t_C	cycle time	P
h_s	saturation headway	L
t_{SUL}	start-up lost times	L
t_{cr}	crossing time	L
Δt_{PE}	interaction time	L
$\Delta h(k)$	cumulated headway difference	L
k	number of entering vehicles	L
	with increased headway	L

(a) Parameters for C_{eff}

Δt_c	conflict difference time	M
$\Delta t_{cr,e}$	crossing time of entering vehicle	L
Δt_e	entering time difference	M
Δt_{cr}	crossing time difference	L
Δt_{cl}	clearance time difference	M
t_{saf}	safety margins	M
Δv_e	entering speed difference	M
Δv_{cl}	clearance speed difference	M
Δl_e	entering distance difference	M
Δl_{cl}	clearance distance difference	M
Δl_{veh}	vehicle length difference	L
p_i	probability of movement sequences	M

(b) Parameters for C_{max}

P	signal program	M	movement sequence	S	stage	L	lane

(c) Level on which parameters have to be obtained

Table 4: Parameters required for C_{eff} and C_{max}

To develop the basis for the model calibration, influencing factors have to be observed. As has been shown in Section 4.3, a variety of factors could be relevant. The calibration of the model, therefore, requires extensive surveys. This would go beyond the scope of this thesis. Influencing factors are consequently only mentioned where they had an apparent significance. Intersections and survey times for the surveys have been chosen to represent common situations and without too many factors biasing the survey results.

5.2.1.2 Data precision

Intergreen time calculation is commonly conducted in full seconds or tenths of a second. At least the same precision has to be achieved by the empirical data to get significant results. All parameters used in the model lead to units of time, which are then used to calculate capacities in vehicles per hour. The required precision of speed and distance measurements is determined from the resulting times. The minimum achievable precision for a mean speed of 10 m/s and a travelled distance of 30 m (as common clearance speeds and distances) against the precision of speed and distance measurements is shown in Figure 17.

The figure illustrates, that a distance precision of about one metre and a speed precision of less than one kilometre per hour would be desirable. The survey layout is a compromise between this demand and the feasibility. The desirable precision of measurements is summarised in Table 5.

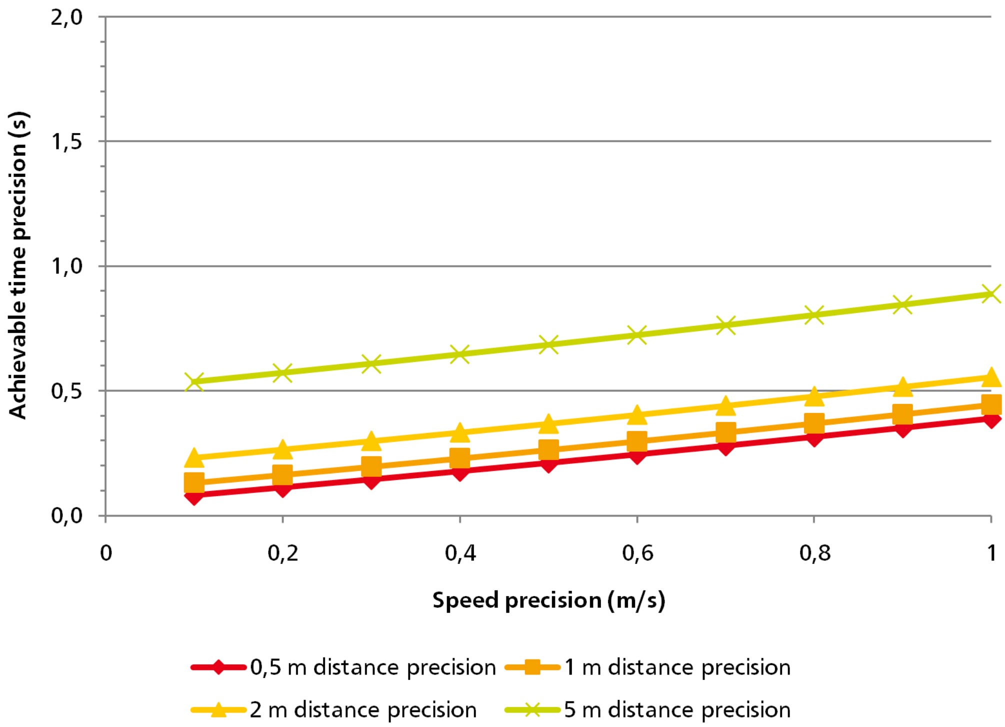


Figure 17: Minimum achievable precision of the travel time of vehicles for $v = 10\,\mathrm{m/s}$ and $l = 30\,\mathrm{m}$ distance travelled

Variable category	Unit	Desired precision
Time	s	0.1
Distance	m	1.0
Speed	km/h	<1.0
	m/s	<0.3

Table 5: Desired precision of measurements

5.2.1.3 Requirements for general observations

Aim of the project is, in addition to the quantification of capacities, to gain further qualitative insight into the traffic flow during the change of stages at signalised intersections. The survey layout has to support this requirement. It should, hence, be possible to analyse the traffic flow on the intersection area qualitatively.

5.2.1.4 Sample size and classification

Due to the many influences on the variables, the focus in this thesis is set on the validation of the methodology, and not on the statistic verification of all possible influences on the model parameters. Still for the survey intersections (including their local conditions) the sample size should be in a statistically relevant magnitude. All conducted measurements have been analysed with respect to the standard error (Section 5.4.4). The standard error δ (Eq. 34) is the relevant statistical variation quantity, because in the present research the averages are of primary interest. The standard error is directly related to the sample size. Low standard errors with respect to the desired accuracy (Section 5.2.1.2) proof a sufficient sample size.

$$\delta = \frac{\sigma}{\sqrt{n}} \tag{34}$$

Where data had to be grouped into classes, the number of classes was determined following the rule of Scott (Eq. 35). When data from different measurements had to be accumulated, the number of classes was averaged and rounded up.

$$k = 3,5\sqrt[3]{\sigma/n} \tag{35}$$

where	k	number of classes	(-)
	σ	standard deviation of sample	([parameter unit])
	n	sample size	(-)

5.2.2 Assessment of survey techniques and development of evaluation methodology

5.2.2.1 Assessment of survey techniques

After a rough pre-assessment of possible survey techniques according to the requirements described before, the following technologies have been further scrutinised:

- light barrier
- optical one-side sensor
- radar
- laser
- pressure sensor
- video

The requirements, namely high accuracy, measurement on intersection approaches, measurement inside of the intersections, economic viability, and simultaneous measurements at different locations, exclude many of the possible technologies.

The most feasible option is using video technology, because several parameters can be efficiently obtained. With a suitable viewing angle, the whole intersection can be captured at once. Because automatic detection methods so far have either too high requirements on the camera position and picture quality, or do not deliver the desired accuracy, the most viable option is a manual evaluation of the videos.

Drawbacks of video observations are insufficient resolution and distortion of the images. They impinge primarily on distance measurements and, because speeds can only be derived from a combination of time and distance measurements, on speed estimations. It was, therefore, decided to gather additional speed data with a more reliable technology. The most accurate and flexible measurements are possible using laser technology. Single vehicles can precisely be aimed at. Frequent measurements are possible, to analyse acceleration behaviour. The only constraint being the movement direction towards (or away from) the laser device to avoid systematic measurement errors.

5.2.2.2 Video observations

Two video observation techniques have been used:

- observation from bird's view
- stop line observations

Deployed devices for bird's view observations

The videos for the bird's view observations were recorded using two IP cameras[10] mounted on a mobile mast approximately 25 m high. The cameras were connected to a computer, thus enabling panning and tilting with direct monitoring of the viewing angle. The videos were recorded in MJPEG format in Advanced Systems Format (ASF) and Audio Video Interleave (AVI) containers respectively. The resolution of the cameras was set to CIF and VGA resolution respectively (i.e. 352x288 pixels and 640x480 pixels respectively). The frame rate averaged 25 to 30 frames per second. The cameras and the trailer with the extension mast can be seen in Figure 18 and Figure 19 on the next page.

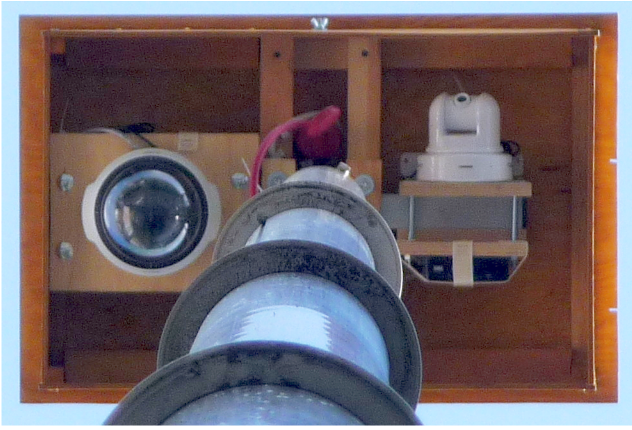


Figure 18: IP Cameras used for video observations (mounted on extension mast)

The two cameras were equipped with wide angle lenses. Depending on the size of the intersection and the position of the mast, at least three approaches and the conflict areas in the intersection could be observed simultaneously. A single frame of a camera illustrating the viewing angle is given in Figure 20 on page 77.

Since the trailer, on which the mast was mounted, requires about 15-20 m^2 area, the choice of the intersections was constricted. The trailer could only be placed on sufficiently wide walkways or adjacent parking sites.

Stop line observations

In addition to these high angle observations, stop line observations were conducted using a normal video camera with hard disc recording[11]. The videos are coded in MPEG-2 in High Definition resolution. The camera was placed near the stop line nearly perpendicular to the observed lane. In this way the stop line, the signal heads, and the vehicles could be observed at the same time. These observations were used to analyse start-up lost times and crossing times.

[10] AXIS PTZ 215 and TRENDnet TV-IP410
[11] Panasonic SDR-H280

Figure 19: Extension mast located next to an intersection

Video evaluation

To get the data, the videos were manually processed using a subtitle generation software[12]. In the subtitles the crossing times of all vehicles at the stop line, the time at the conflict points, the vehicle type, the observed and conflicting stream, and special occurences were recorded. Each lane was evaluated separately.

The subtitles were recorded with a precision of a tenth of a second. The generated subtitle text files can easily be imported into other software, thus enabling the automated processing. Visual Basic modules were programmed to transfer the subtitles into MS Excel and evaluate the contained information. Invalid measurements due to missing frames were discarded.

Process (entering or clearing), stream, vehicle type, and special occurences were contained in the subtitle text. The data was separated into the following elements:

- crossing time of the stop line of all vehicles
- stream and vehicle type of all vehicles
- crossing times of conflict points of the first entering and last clearing vehicle

[12] Subtitle Workshop 2.5 by URUsoft, www.urusoft.net

Figure 20: View of the AXIS IP camera from the extension mast

The stop line evaluation was conducted in the same manner. Conflict points could naturally not be observed, due to the camera position. In addition to the crossing times of vehicles, the beginning and ending times of the green vehicle signal was recorded.

5.2.2.3 Controller data

Due to the high camera angle, the signal heads were not visible on the videos from the high angle observations.[13] The signal changes had to be recorded separately using the signal controller. The controller data could only be recorded with the precision of one second.

To synchronise video and controller data, a signal head was recorded prior to the extension of the mast with the cameras. The visible signal change together with the current time could then be assigned to a signal change in the controller data. However, the achieved accuracy for times recorded with reference to signal changes (e.g. start-up lost times) didn't fulfil the requirements. Start-up lost times and crossing times are therefore based on the direct stop line observations.

5.2.2.4 Speed measurements

Deployed device and survey setting

The entering and clearance speeds of vehicles was recorded separately for selected approaches using a laser device (Figure 21).[14] The device was positioned on the far side of the intersection. The vehicles were mostly measured from the front, thus enabling the detection of the speed of entering vehicles. The

13 In Germany, signal heads are mounted on the respective near side of the intersection, and, thus, are only visible from the according approach.

14 TraffiPatrol XR by ROBOT Visual Systems GmbH, Monheim/Rhein, Germany

entering speed was measured in a fast succession (about every one second). In this way, the speed profile and acceleration behaviour of vehicles could be determined.

Figure 21: Speed measurement with TraffiPatrol XR

While through traffic can be measured in this way with high accuracy, the capturing of turning traffic is restricted. The Doppler principle used by the device only works precisely with the laser beam pointing in the same direction as the movement. The measured speed v_{meas} deviates from the true speed v_{true} by the reciprocal of the cosine of the angle α between movement direction and laser beam direction. The manufacturer recommends a lateral distance of no more than a hundredth of the longitudinal distance (equaling about 0.6 °). The measurement error for greater angles is shown in Table 6.

α (°)	$\frac{v_{\mathrm{meas}}}{v_{\mathrm{true}}}$ (-)	error ($^{\mathrm{m}}/_{\mathrm{s}}$)
1	≈ 1	0.002
2	0.999	0.009
5	0.996	0.057
10	0.985	0.228

Table 6: Speed measurement error depending on the angle α between movement direction and laser beam ($v_{\mathrm{true}} = 15\,^{\mathrm{m}}/_{\mathrm{s}}$)

Even for angles of nearly ten degrees the systematic accuracy conforms to the requirements formulated on page 72. The distance is measured with a precision of ten centimetres, the time with a precision of one second. While distance and speed measurements fulfil the desired accuracy, the time measurement does not. Times (namely entering and clearance times) had, therefore, to be derived from distance and speed measurements. The accuracy stated by the manufacturer (Table 35, Appendix B.1.2) fulfills the requirements sufficiently.

Data evaluation

The data generated from the device was imported into spreadsheet software. Invalid measurements were discarded. The remaining data was separated into entering speeds and clearance speeds. Vehicle types, flow directions, and special occurences were recorded on a measurement protocol and added to the electronic data afterwards.

Under saturated conditions, vehicles clear the intersection as a platoon. In this case the speed of all clearing vehicles is similar. If a vehicle cleared the intersection separately from other vehicles, it was marked and evaluated separately. However, this event is an exception and, thus, primarily of importance for safety evaluations.

Moving starts (Type 3 as described on page 44) only exceptionally occured. No significant conclusions could be drawn on the crossing times of these vehicles and their speed.

5.2.2.5 Interaction times

An interaction time occurs, if the entering vehicle adjusts its entering behaviour due to clearing vehicles in front of it. If a vehicle starts late or accelerates more slowly than it would without any vehicles in the intersection from the last stage, a time difference Δt_{PE} reduces the effective green time.

This time difference cannot be observed directly in situ, because the uninfluenced driver's behaviour is unknown. Whether the observable entering process is influenced by clearing vehicles or not can only indirectly be judged. This analysis requires accurate and comprehensive data on the traffic flow in the intersection. However, the results of the surveys conducted as part of this research did neither deliver the necessary accuracy, nor could a sufficient sample size for such kind of observations be realised.

A methodology to acquire empirical data on the interaction times is presented here as a proposition for further research. Furthermore, based on the surveys the likeliness of interaction times has been estimated, as will be explained in Section 5.3.6.

To estimate interaction times from empirical data, entering times have to be compared for potentially critical situations with situations unlikely involving an interaction. The post-encroachment time can be used to separate the two situations from each other. If a correlation of the entering behaviour with the post-encroachment time can be verified, this correlation gives an indication on the magnitude of interaction times.

The methodology requires, hence, three steps:

1. measurement (or derivation) of post-encroachment time
2. measurement of entering time
3. test for correlation of the two

The post-encroachment time can either be directly measured or calculated from clearance and entering times. The difficulty is that entering time and post-encroachment time have to be measured simultaneously with high accuracy.

To acquire entering times from the video observations, exact time and distance measurements have to be realised. The time measurements have to be related to the signal timing (namely the beginning of green). Exact measurements of the post-encroachment time imply a reference to a conflict point. This conflict point doesn't have to be the conflict point for the calculation of the intergreen times. Drivers may perceive a different point as determining for their behaviour. It has, however, to be a more or less unique point for all observations of one kind of conflict to ensure compatibility.

Another issue is the sample size. Because the probability of interaction times is low (cf. Section 5.3.6), the sample size has to be big as to include sufficient situations where interaction times occur. For a conflict, which occurs with a probability of 50 % and a probability that a interaction time occurs for this conflict of 20 % (both being assumptions tending to an overestimation of the probability), in only ten percent of cycles a manoevre time for the respective conflict can be observed. To acquire 20 observations of interaction times, on average 200 cycles have to be evaluated (5 peak hours for a cycle time of 90 s). Due to variations of the occurence of conflicts and interaction times this value can easily reach significantly higher values.

It is apparent that the observations should focus on conflicts with high probability and low absolute intergreen time differences. These conflicts are more relevant and reduce the necessary sample size. The conflict tree of the survey intersection delivers these values.

5.2.3 Realisation

5.2.3.1 Intersections

At seven urban intersections in the German city of Darmstadt different kinds of data has been gathered. The surveys were conducted in the morning and afternoon peak. An overview of the intersections with the type of coordination (both main directions, only one direction, no coordination), and the survey time (MP: morning peak, AP: afternoon peak) is given in Table 7. Plans of all intersections and a map with their location are given in Appendix B.2.

Intersection identifier	Coordination	MP	AP	Speed	Stop line observation	High angle observation
A 018			✓	✓	✓	
A 019	✓	✓	✓	✓		✓
A 020	(✓)	✓	✓	✓		✓
A 042		✓			✓	
A 046		✓	✓	✓		✓
A 086		✓			✓	
A 098	✓	✓		✓		✓

Table 7: Survey intersections

The intersections were chosen for the following reasons:

- crossroads (standard intersection type)
- high vehicle volumes, preferably on both axes
- installation of equipment possible (wide walkway etc.)

All intersections have a fixed cycle length of 90 s during the peak hours and different kinds of traffic actuation.

5.2.3.2 Interference with traffic flow

For the surveys to be meaningful for general conclusions, it is important that the survey itself does not influence the traffic flow, which includes the driver behaviour. The extension mast, despite – or for – its height, was not easily visible to drivers focusing on the traffic in front of them. An influence on the driver behaviour is not likely, as could be supported by observations on site.

The setup of the speed measurements provided for the influence on the drivers. A position easily visible from the intersection approach was avoided. Drivers who apparently recognised the device quite often reduced their speed – even despite the fact, that speed limit violations of entering vehicles are extremely unlikely. The speed was therefore measured near the stop line, while the device was located in the intersection exit. The speed was measured before drivers perceived the measurement and could react to it. Both, the entering and clearance speeds can, hence, be regarded as reliable.

The greatest difficulties have been observed at high speed approaches (highway entrance to urban area at southern approach of A 020), where speed limit violations could in fact frequently be observed. However, the position of the measurement device was either slightly hidden by trees or distant from the intersection.

5.2.3.3 Simplifications

The comprehensive model requires data for all lanes on all approaches of the evaluated intersection. Due to survey limitations and based on the results, some simplifications have been made. It was decided to generalise parameters that do not differ significantly among lanes, approaches, or even intersections. Accurate speed measurements with the laser device could only be realised on one approach at a time. As will be shown in the next section, the entering speed distributions do not vary decisively among most intersections. Also the clearance speeds follow certain general characteristics.

Furthermore, the stop line observations have been used to obtain crossing times of the entering ($t_{cr,e}$) and clearing vehicles (t_{cr}) only. Starting response times were not recorded, because the first queued vehicle was not always visible. For the effective capacity only crossing times of entering vehicles are required. As for the maximum capacity, the entering process was considered as a moving start (Type 3, cf. p. 44) to correct for the difference between starting response time t_{SR} and crossing time $t_{cr,e}$.

For the model application, data was partly used which had been obtained at different intersections. This simplification reduces the reliability of the quantitative model output.

5.3 Survey results

5.3.1 Introduction

In this section the results of the surveys are given for the different model parameters separately. In Section 5.4 these results are fed into the capacity model for an example intersection. The source of the data is given by the

- intersection identifier (e.g. A 046)
- peak (morning peak, MP, or afternoon peak, AP)
- approach (compass direction; e.g. 'N' for northern approach)
- lane (relative position as right, R, left, L, centre or single lane, C)

Not all survey results are presented here to focus the attention on the pivotal aspects. As has been highlighted before, the empirical data obtained during the surveys has to be regarded as exemplative for the individual intersection where it has been gathered. Parameters surveyed at several intersections with accurate results give indications on the possible influencing factors on the parameters and show the variation of the parameter. Preliminary conclusions can be drawn from this observations. The descriptions in this section are focused on results relevant for the example intersection.

5.3.2 Saturation headway and start-up lost times

Headway data has to be gathered to get the saturation headway h_s, and to derive the start-up lost times t_{SUL}. The headway data was gathered at approaches with low heavy vehicle share and similar turning traffic proportions. The headways were cumulated by vehicle position.

5.3.2.1 Saturation headway

In Figure 22 the headways are displayed according to approach lanes (dots) and averaged over all approaches (line). The ordinate represents the ordinal headway number (i.e. vehicle position of leading vehicle). The last two vehicles in a queue have been discarded to exclude influences of the clearing vehicles, because the green times and, thus, the queue lengths varied significantly. The columns represent the sample sizes of the average headways per position. Values from samples with less than ten observations have been discarded, as have been headways involving bicycles. Each overall average up to headway ten is based on at least 30 observations.

It can be seen that the headway drops, until the fifth vehicle crossed the stop line ($k = 4$). This observation holds true for all approach lanes. The variation from the sixth vehicle on is small. Hence, the headways of the six to last but two vehicles were averaged to give the saturation headway. The thus calculated saturation headway has a coefficient of variation of less than ten percent. The detailed data is given in Table 8. Only data from approach lanes with sufficient sample size (more than ten headways per position) is listed. Row three gives the cumulated headway difference according to Eq. 9 of the first four headways $\Delta h(4)$.

	Parameter	Unit	NR	WR	SR	Overall
1	Saturation headway	(s)	1.9	2.0	1.8	1.9
2	Standard deviation of h_s	(s)	0.5	0.5	0.4	0.5
3	Standard error of h_s	(s)	0.3	0.3	0.2	0.3
4	Cumulated headway difference $\Delta h(4)$	(s)	1.0	1.1	1.0	1.0
5	No of headways (n)	(-)	248	297	124	1022

Table 8: Saturation headways and cumulated headway differences (A 046, AP)

While the saturation headway varies slightly among the observed approaches (regardless of similar traffic composition), the cumulated headway difference remains nearly constant. For the model application (Section 5.4), $\Delta h(k) = 1.0\,\mathrm{s}$ is used, based on the data of more than one thousand headways as given in Table 8. Long (2005) supports this simplification by observing follow up response times of subsequent vehicles in a queue and finding no significant variation among observation sites, vehicle position in queue, vehicle type, and others.

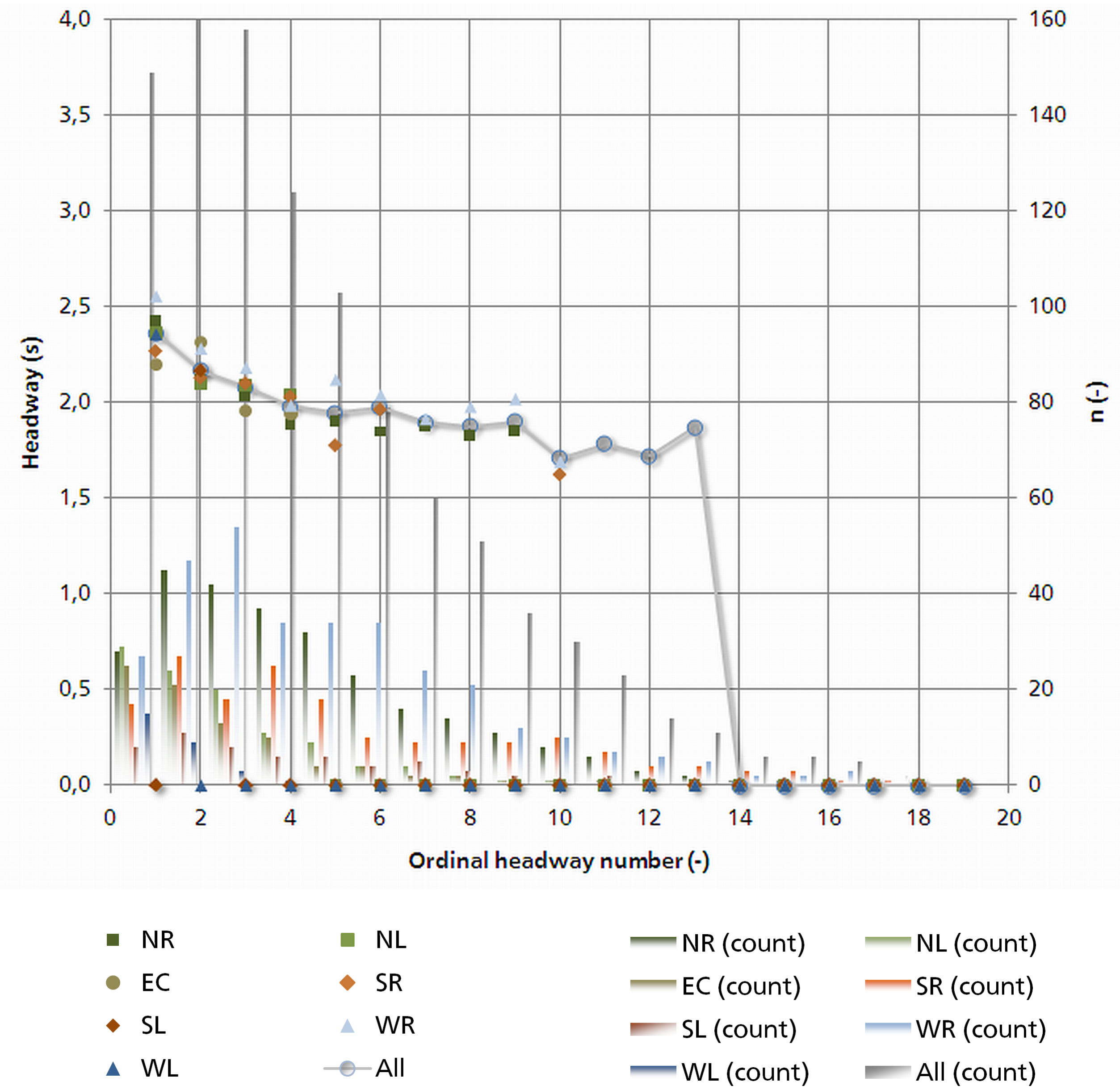


Figure 22: Headways of entering vehicles (A046, AP)

5.3.2.2 Start-up lost time

The start-up lost times t_{SUL} can be derived from the cumulated headway difference $\Delta h(k)$, the crossing time of the entering vehicle $t_{\mathrm{cr,e}}$, and the saturation headway h_s following Eq. 10. If the crossing time of entering vehicles is not directly measured, it can be determined from the starting response time t_{SR} with reference to the green start and the time needed by the first vehicle in the queue to cover the stop line distance l_{SL} according to Eq. 36 (cf. Section 3.2.2).

$$t_{\mathrm{cr,e}} = t_{\mathrm{SR}} + \frac{l_{\mathrm{SL}}}{\overline{v}_{\mathrm{e,eff}}} \tag{36}$$

During the surveys the crossing time of entering vehicles was directly measured, instead of a calculation according to Eq. 36.

Stop line distance

The time to cover the stop line distance can vary significantly. Particularly for large stop line distances ($l_{SL} \gtrsim 2\,\mathrm{m}$) it is advisable to derive the crossing times of entering vehicles from starting response times and entering speed behaviour (Eq. 36, see also Section 5.3.4). At the survey intersections the stop line distance l_{SL} tended to remain below two metres. HOFFMANN and NIELSEN (1994) give an average distance of only one metre with slightly varying results for different intersections. Values in the United States tend to be significantly higher. LONG (2006) gives average values of 3.2 m with a standard deviation of 2.6 m. FOLLMANN and SCHUSTER (1991) highlighted the role of road markings on the crossing time of entering vehicles[15].

In addition to the stop line distance, LONG (2005) outlined other possible factors influencing the start-up lost times, namely differences in positions of the stop line relative to the curb line, differences in blockages by cross-street traffic spilling back into an intersection, and differences in pedestrians straggling across the crosswalk.

During the speed measurements only the distance to the vehicles was measured. The distance measurements of the stopped vehicles right before they started moving can be used to analyse the variation of the stop line distance. The results are shown in Table 9. Overall this distance varies about one metre, which coincides with the results from HOFFMANN and NIELSEN (1994).

Intersection	MP	AP
A 018		0.8
A 019	1.0	1.4
A 020	0.8	
A 046	0.8	0.9
A 098	1.4	

Table 9: Standard deviation of stop line distance (m)

Crossing times of entering vehicles

The crossing times of entering vehicles were measured at three intersections. The results are given in Figure 23. The statistics are shown in Table 10.

	Parameter	Unit	A 018(1)	A 018(2)	A 086
1	Average crossing time $t_{cr,e}$	(s)	1.1	1.4	1.3
2	Standard deviation of $t_{cr,e}$	(s)	1.0	1.2	1.1
3	Standard error of $t_{cr,e}$	(s)	0.1	0.1	0.1
4	Sample size (n)	(-)	142	66	74

Table 10: Crossing times of entering vehicles $t_{cr,e}$

HOFFMANN and NIELSEN (1994) determined crossing times of entering vehicles (starting response times) of 1.3 s for a transition time of one second based on observations from one intersection. TANG (2008) states crossing times of entering vehicles (starting response times) for German intersections of 0.9 s. The results from the surveys support the observations from TANG, while the values from HOFFMANN and NIELSEN are slightly greater. Crossing times obtained in the United States are commonly higher. LI and

[15] While their statement is derived from observations at bottlenecks, it seems sensible to derive a general influence.

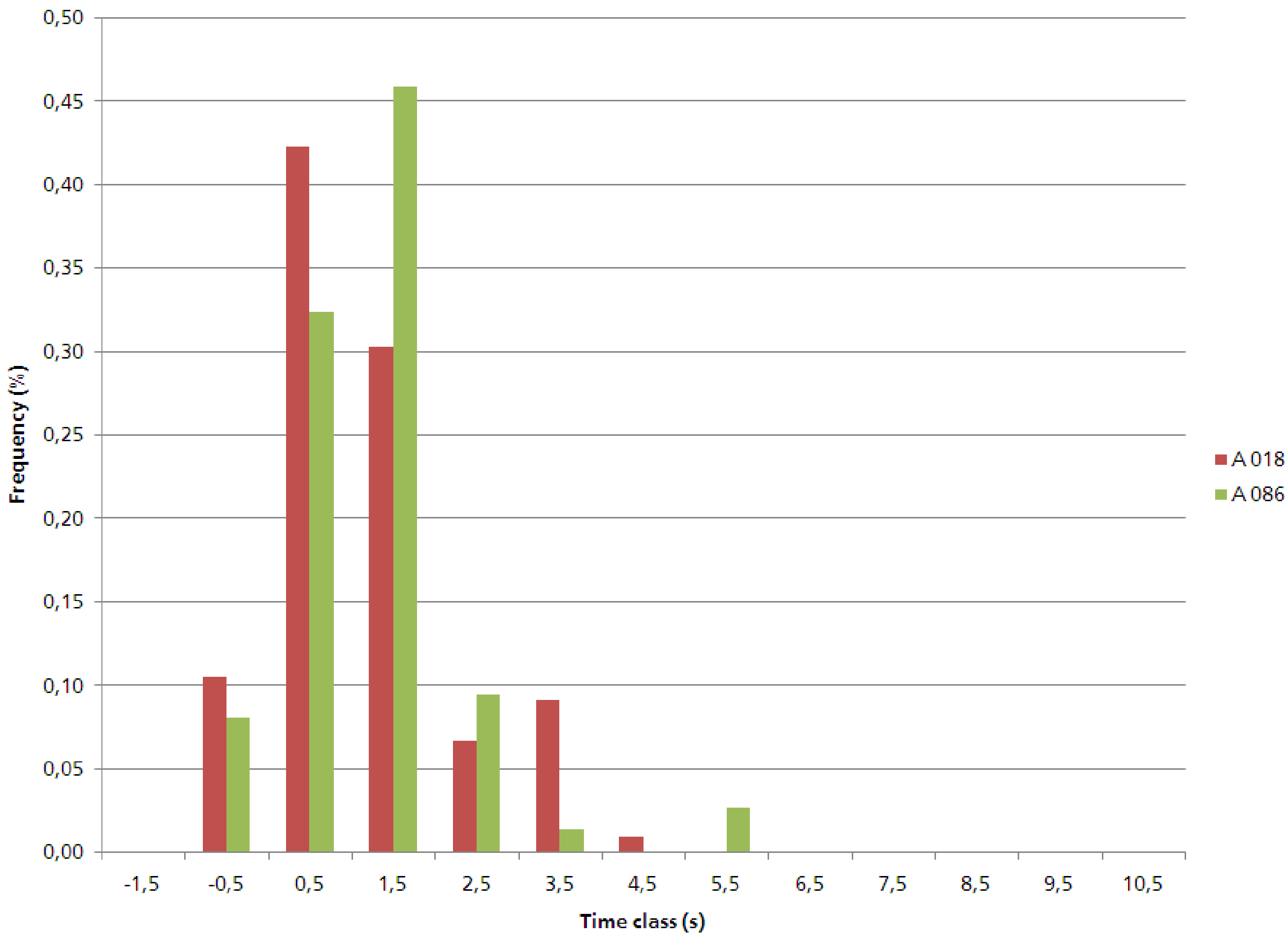


Figure 23: Crossing times of entering vehicles $t_{\mathrm{cr,e}}$ (n=208 at A 018, n=74 at A 086)

Prevedouros (2002), for instance, determined values around 1.6 s. The greater values originate from greater stop line distances and the lack of a transition time between red and green.

The coefficient of variation of the crossing times measured during the surveys ranges around one. Some uncertainty comes along with the crossing times. However, the variation among different intersections under similar traffic conditions (namely saturation) is low. The data from Table 10 is, therefore, averaged giving a general crossing time of entering vehicles of 1.2 s.

With a cumulated headway difference of $\Delta h(k) = 1.0$ s and a crossing time of entering vehicles $t_{\mathrm{cr,e}} = 1.2$ s the start-up lost times can be calculated as a function of h_s. The results for the survey intersections are given in Table 11.

	Parameter	Unit	NR	WR	SR
1	Saturation headway h_s	(s)	1.9	2.0	1.8
2	**Start-up lost time t_{SUL}**	(s)	**0.3**	**0.2**	**0.4**

Table 11: Start-up lost times (A046, AP)

5.3.3 Crossing times of clearing vehicles

The crossing times of clearing vehicles are commonly positive. Crossing times longer than the yellow interval stem from red runners. In Figure 24 the results for intersections A 018 and A 086 are shown. The statistics are shown in Table 12. The dashed lines show cumulative normal distributions fitted to the data using the least squares method. Data from both intersections approximates a normal distribution, but is slightly negatively skewed. A 018 shows a significant amount of red runners; A 086 has lower mean and lower kurtosis.

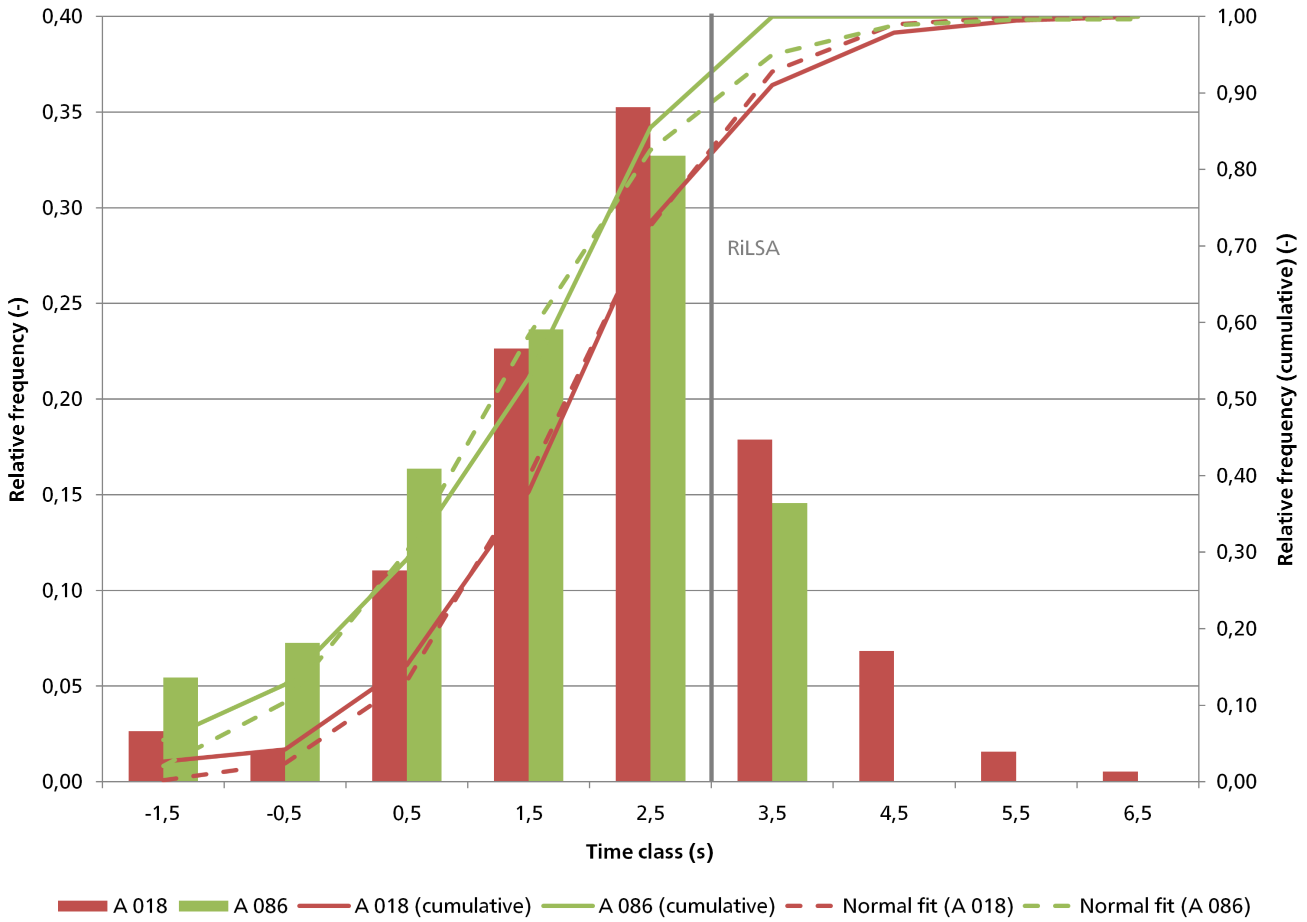


Figure 24: Crossing times of clearing vehicles t_{cr}(n=192 at A 018), n=57 at A 086)

	Parameter	Unit	A 018	A 086
1	Average crossing time	(s)	2.2	1.6
2	Standard deviation of t_{cr}	(s)	1.4	1.6
3	Standard error of t_{cr}	(s)	0.1	0.2
4	Sample size (n)	(-)	192	57

Table 12: Crossing times of clearing vehicles (A 018, AP; A 086, MP)

The sample size for the crossing times is too small to give reliable results on the distribution and the influencing factors. Tang and Nakamura (2007a) indicated a dependency of the frequency of red light violations and the aggressiveness during the yellow interval (observed by the post-encroachment times and speed violations) with vehicle volumes, intergreen times, and red times. Care has to be taken when calibrating the capacity model with respect to crossing times. Due to the unusual amount of red runners at A 018, the crossing times of A 086 are used for the exemplative model application described in Section 5.4.

5.3.4 Speeds

5.3.4.1 Entering speeds

Standard entering processes

At saturated approaches or coordinated approaches with vehicles regularly waiting at the stop line during the onset of green, Entering Type 1 is the standard entering process (cf. Figure 8). This process is a variably accelerated movement. Distance, speed, and acceleration depend on the interval since the start of movement.

For the calculation of intergreen times, the distance from the stop line to the present location of a vehicle is known (or assumed), while the time to cover this distance has to be calculated. The entering movement has, hence, to be described by a function relating time to a distance. The distance, of course, has to be corrected by the systematic and random error Δl_{e}.

Derivation of the entering speed function

Accelerations cannot be measured directly in the present context, therefore a rapid succession of speed measurements was undertaken. Furthermore, the time precision of the used device of one second is insufficient for an accurate estimation of the movement functions. Speed and distance, otherwise, are recorded with satisfactory precision (one kilometre per hour and 10 centimetres respectively).

Therefore, the average movement of the entering vehicles is described based on distance/speed measurements. These measurements were cumulated and a logarithmic function fitted using the least squares method. This approximation describes the speed as a function of the distance. As compared to power functions, a logarithmic function could achieve a better fit. Since such a function is strictly increasing, the approximation is only valid as long as the vehicles accelerate. However, the acceleration process takes commonly significantly longer distances than the entering distances of ordinary signalised intersections. Thus, this approximation holds true for the described problem.

To derive entering times from a speed-distance-function of a variably accelerated movement is only possible by dividing the movement into small steps with assumed constant accelerations. The movement functions for accelerated movements can then be applied to these steps, and the time can be determined incrementally as a function of distance. Details on this procedure are provided in Appendix A.4.

Variance of the acceleration behaviour

Data was collected at five urban intersections. A two parametric logarithmic function (Eq. 37) was fitted to the data sets for each survey individually as described before. The optimum parameters are given in Table 13 together with the number of data sets on which the function fitting is based (sample size). The last column gives the sum of the squared errors. The resulting functions are shown in Figure 25.

$$v(s) = c_1 \ln(c_2 s + 1) \qquad (37)$$

	Intersection	Peak	Sample Size	c_1	c_2	Squared error ($^{km}/_{h^2}$)
1	A 018	AP	118	2.8	0.96	0.75
2	A 019	MP	387	2.6	0.96	2.09
3	A 019	AP	186	2.6	0.96	0.99
4	A 020	MP	118	2.8	0.96	0.75
5	A 020	MP	73	2.8	0.94	0.86
6	A 046	AP	427	2.5	0.95	0.67
7	A 046	MP	279	2.4	0.95	0.10
8	A 098	MP	193	2.8	0.95	0.50

Table 13: Survey details and results of the entering speed measurements

The results show that

- the logarithmic function can be well fitted to the data,
- the second parameter of the function is more or less constant for all survey intersections,
- there is only a weak indication that morning peak (banded rows) and afternoon peak may differ, and
- the data of three intersections coincides quite precisely.

The lower accelerations at intersection A 019 and even more A 046 cannot be explained without further scrutiny. The results, however, give an indication of the range of possible entering behaviour. The results coincide quite accurately with the results obtained by driving measurements conducted by HOFFMANN and NIELSEN (1994). HOFFMANN and NIELSEN showed, furthermore, that the maximum speed of the first vehicle is commonly only reached after more than 150 m, which justifies the use of a strictly increasing function for common entering distances.

Entering time difference

The entering time depends on the entering speed and the entering distance. Because the entering speed is the decisive factor in this context due to its greater variability, its impact on the intergreen time difference is highlighted here.

The German Guidelines for Traffic Signals (RiLSA) assume a moving start (Type 2) with a constant speed of 40 $^{km}/_{h}$ and no crossing time. The entering time calculated in this way can be compared to the effective entering time based on the *equivalent entering speed* as explained in Section 3.3.2 on page 44 f. The *equivalent entering speed* $\overline{v}_{e,eff}$ is defined as the speed with which an entering vehicle would have to travel the entering distance as to arrive at the same time as it arrives with the real accelerated movement.

The equivalent entering speed $\overline{v}_{\mathrm{e,eff}}$ as a function of the entering distance is shown in Figure 26 with the entering speed according to RiLSA indicated in grey. The difference between the effective entering time and the entering time calculated according to FGSV (1992) as a function of the entering distance is illustrated in Figure 27. In the graph shown, the entering distance is assumed constant (no entering distance variation).

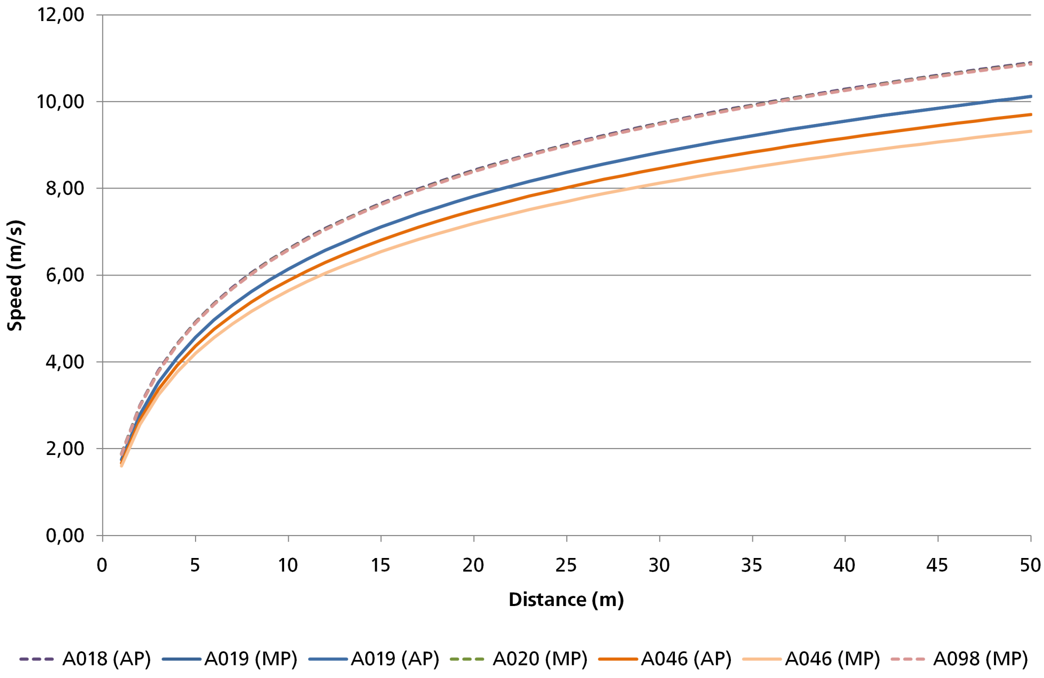


Figure 25: Entering speed functions

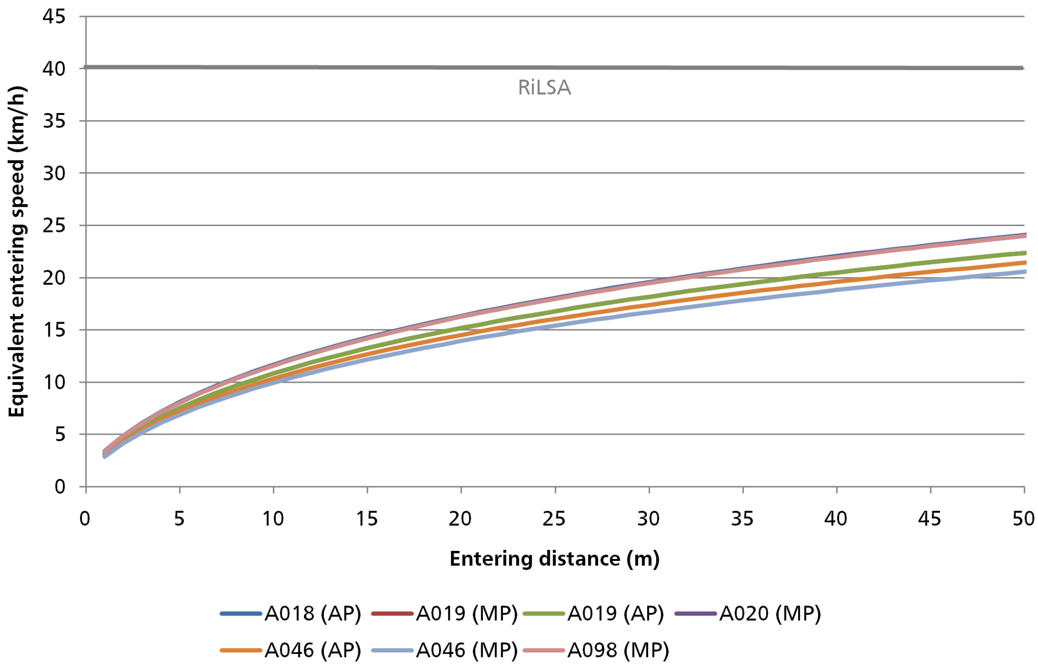


Figure 26: Equivalent entering speeds as a function of the entering distance

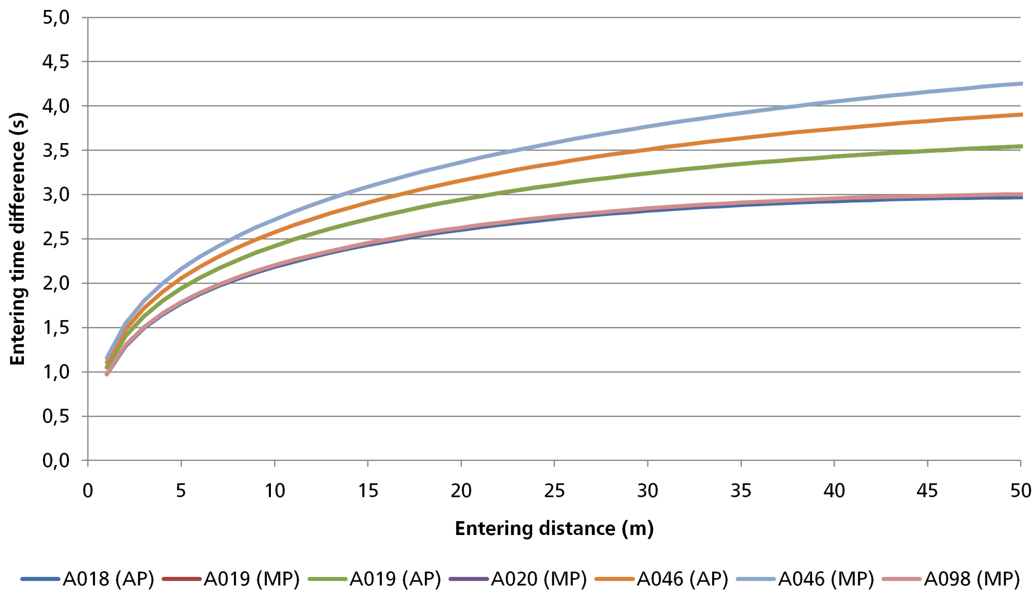


Figure 27: Entering time difference with reference to RiLSA as a function of the entering distance

Moving starts

As has been explained before, moving starts (Type 2 or 3 according to Figure 8 on page 44) are a rare exception (cf. Jakob 1980). The survey supports this statement. The number of moving starts was too small to obtain significant information. Moving starts have, hence, been discarded from the entering speed assessment.

Particularly for coordinated approaches a thorough individual evaluation is recommended. It has to be checked whether moving starts can frequently be expected. Entering crossing times and entering speeds have to be obtained to compute accurate capacity estimates. The results will depend siginificantly on the realisation of the coordination (lead time for turning traffic, offset etc.). This procedure was not followed further, because these observations cannot be generalised.

5.3.4.2 Clearance speeds

During the clearance process, vehicles rarely change their speed significantly. As part of the speed measurements, several clearing vehicles were measured successively. However, a significant change of speed could not be observed. For safety considerations, the rare situations, where clearing vehicles accelerate before arriving at the intersection and decelerating after crossing the stop line have to be considered. For capacity considerations these situations seem to be negligible.

The clearance speed depends noticeably on the approach speed and the saturation degree. In saturated cycles, vehicles clear the intersection as a platoon. The clearance speed, thus, is determined by the slowest vehicles in the platoon. For saturated, non-coordinated approaches with short green times, the maximum speed of the platoon remains well below the speed limit. This coincides with the clearance speeds assumed by the German Guidelines for Traffic Signals (RiLSA). Even for approaches with average green times around 40 s the clearance speed remained lower than the speed limit. An apparent correlation between green time duration and clearance speed could not be observed. This effect could, nevertheless, be superimposed by the influence of coordinated approaches.

Well coordinated approaches lead to significantly higher clearance speeds. Figure 28 shows the clearance speed distributions of the five intersections where speed measurements have been conducted. The details are given in Table 14. The observed approaches of intersections A 019 and A 98 are coordinated and have been summarised, as have the remaining intersections A 018 and A 20. While the speed limit is 50 km/h, free flowing vehicles sometimes reach speeds of up to 70 km/h.

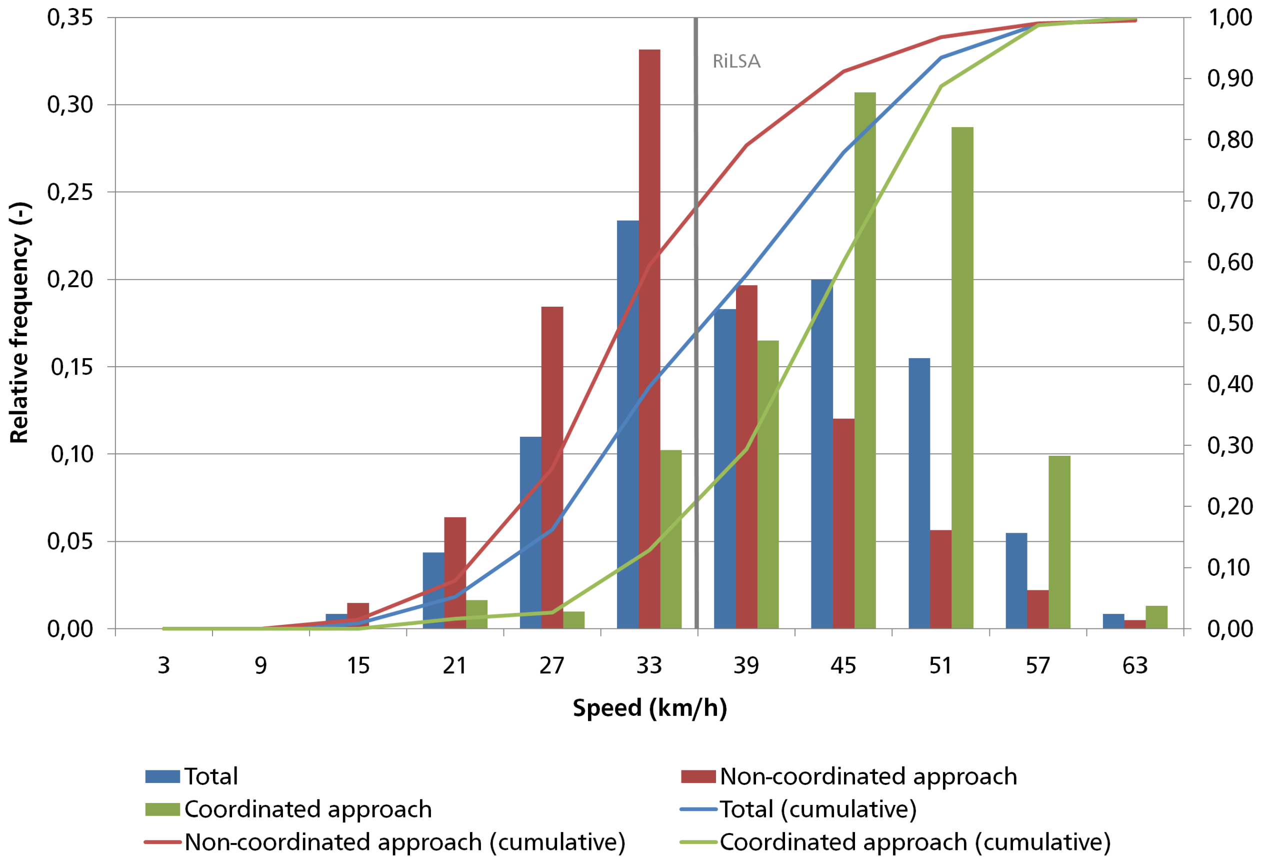


Figure 28: Clearance speed distribution (n=711)

As the average clearance speed varies according to local conditions, the distribution varies, too. For approaches with high pressure on the drivers (high saturation degree, short green times), the distribution tends to be negatively skewed. Higher speeds during yellow as reported by Chang et al. (1985) have not been in evidence. Due to the chosen methodology a systematic assessment, however, could not be realised. For saturated conditions vehicles always clear during yellow. Therefore this dependance is not of particular importance. Surveys conducted by Tang (2008) at coordinated and non coordinated intersections in the same city showed higher clearance speeds (50 km/h averaged over all three survey intersections for through traffic). The clearance speed, hence, seems to vary significantly among intersections.

5.3.5 Clearance and entering distances

The variation of clearance and entering distances depends primarily on the movement. Straight traffic only varies between the lane borders. With a average vehicle width of 1.8 m and a lane width of around 3 m the variation is marginal. Knoflacher and Schopf (1981) obtained the variation of the trajectory of through vehicles and found values less than 0.5 m for residual lane widths of less than 1.5 m. These values coincide well with the results obtained by Jakob (1980). Contrarily, trajectories of turning traffic can easily differ between the outer radius of the turning lane and the shortest distance between stop line and intersection exit, amounting to over five metres. Figure 7 on page 43 illustrates the extreme trajectories of turning traffic for a big intersection (intersection A 046).

	Intersection	Peak	Sample Size	Mean	Min	Max	Standard deviation	CoV	δ
			(-)	(km/h)	(km/h)	(km/h)	(km/h)	(-)	(km/h)
1	A 018	AP	75	38	18	80	11	0.28	1.2
2	A 019	MP	87	46	21	61	8	0.18	0.8
3	A 019	AP	164	45	30	62	7	0.16	0.5
4	A 020	MP	75	38	18	50	11	0.28	1.2
5	A 020	MP	60	36	25	48	5	0.15	0.6
6	A 046	AP	73	33	21	54	7	0.22	0.8
7	A 046	MP	124	35	16	57	9	0.25	0.8
8	A 098	MP	93	47	23	67	9	0.19	0.9
Coordinated			304	46	21	67	8	0.17	0.4
Non-coordinated			407	36	16	80	9	0.24	0.4

Table 14: Results of the clearance speed measurements

The variation of the vehicle length was not further analysed here, due to survey limitations. Hoffmann and Nielsen (1994) ascertained an average vehicle length of 4.5 m resulting in a difference of 1.5 m as compared to the German Guidelines for Traffic Signals (RiLSA).

5.3.6 Likeliness of interaction times

Methodology

Tang and Nakamura (2009) analysed the post-encroachment time at signalised intersections in Germany and Japan. The results show average values of about 6.7 s for German intersections. The minimum values approach 2.0 s as exceptional values, values around 3 s being more frequent. These times are based on turning movements only, which have not been checked for their decisiveness. The sample size is comparably small ($n < 100$). Under these circumstances post-encroachment times of less than two seconds are apparently only accepted by a negligible minority of drivers.

Post-encroachments as a safety measure have been analysed also by van der Horst (1990). He proposed times of less than one second to regard as critical and values of more than 1.5 s as comparably safe. For an analysis of risky behaviour and and assessment methodology of conflicts at signalised intersections refer also to Suzuki et al. (2004b).

To remain on the safe side the determination of the likeliness of interaction times is based on the recent results from Tang. It can be reasoned that the likeliness of an interaction, and consequently the occurence of an interaction time, increases from naught to one between theoretical post-encroachment times t_{PE}' of around 6 s to 2 s. To get an estimate for the occurence of interaction times, the probability of such theoretical post-encroachment times can be used.

The theoretical post-encroachment time can be calculated if the (uninfluenced) driver behaviour of entering and clearing vehicles, namely their crossing time (entering), entering time, crossing time (clearing), and clearance time, is known. Because these values have been observed as average values or can be derived from observed average values, the data from the surveys can be used to calculate the average theoretical post-encroachment time for the example intersection.

The German Guidelines for Traffic Signals (RiLSA) do not take a post-encroachment time into account. The negative intergreen time difference of conflicts $\Delta t_{ig,m}$ gives, thus, a direct measure of the post-encroachment time. Following the conclusions from above, a probability function for the occurence of interaction times given the theoretical post-encroachment time can be estimated.

Because the intergreen time difference is calculated from several random distributed variables, the intergreen time difference can be assumed to be normally distributed with a standard deviation calculated from the square root of summarised variances of the input variables. The intergreen time differences from the example intersection (A 046) are sufficiently normally distributed (90% confidence) as can be seen in Figure 29.

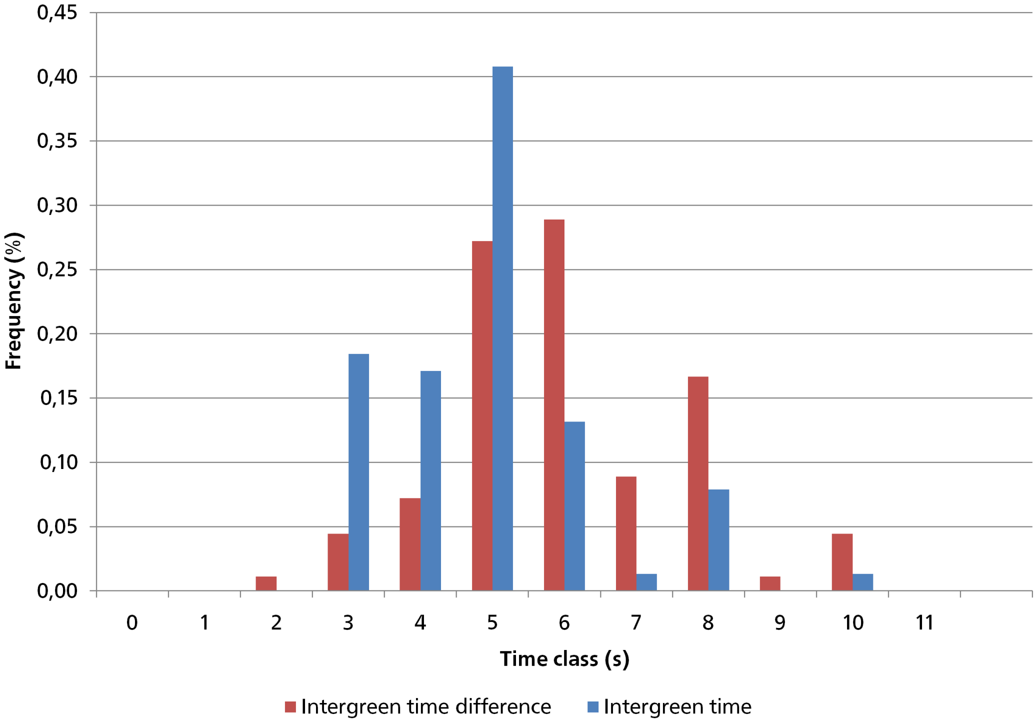


Figure 29: Distribution of intergreen time differences for all movement sequences (A 046, AP)

Estimation for example intersection

Based on this distribution with a mean of -7.3 s, the probability of intergreen time differences of less than six seconds is about 20 %, of intergreen time differences less than four seconds only four percent.

If the probability of the individual conflicts is considered and the interaction time assumed to be the difference between six seconds and the respective negative intergreen time difference (Eq. 38), the average interaction time over all conflicts $\Delta t_{\mathrm{ig,PE,m}}$ is about 0.12 s.

$$t_{\mathrm{PE}} = (6\,\mathrm{s} + \Delta t_{\mathrm{ig,m},i}) \cdot p_i \qquad \forall \quad \Delta t_{\mathrm{ig,m},i} > -6\,\mathrm{s} \tag{38}$$

This result shows that if *all* drivers would not accept a post-encroachment time of less than six seconds (and delay their entering to achieve this minimum time gap), on average additional 0.1 s of interaction time Δt_{PE} would have to be added to the individual intergreen time differences. The overall improvement potential would change less than 50 veh/h for the example intersection, which is significantly less than the achieved accuracy of the measurements (cf. Section 5.4.4). Interaction times are consequently neglected in the exemplative model application.

5.4 Model application

5.4.1 Example intersection

The comprehensive model is applied to an urban intersection in the City of Darmstadt in Germany. The intersection plan is shown in Figure 30. The signal program during the peak hours is a fixed cycle program ($t_C = 90\,\text{s}$) with traffic actuation (public transport prioritisation, green time adjustments, demand stage for turning traffic). Green times and stage sequence vary. Public transport (trams and buses) uses separate lanes in the middle of the main direction.

Simplifications

To give more general results, the stages reserved for public transport only are excluded from the following observations. Moreover, a fixed stage sequence is assumed, avoiding thus additional complexity.[16] The influence of pedestrians at the observed intersection is negligible due to low volumes. Some stages even leave out the parallel pedestrian stream. Signal groups of pedestrians, hence, have been left out here for simplification.

The used stage sequence with signal group identifiers is given in Figure 31. The most frequent time of the public transport stage is indicated between stages V and I. This stage change, hence, has been excluded from the assessment. A direct change from stage V back to stage I is supposed.

The mentioned simplifications lead to an underestimation of the capacity improvement potential due to a lower number of stage sequences. The improvement potential is naturally correlated to the number of stages of the signal program.

Traffic conditions

The intersection has a low heavy vehicle share on all approaches. The afternoon peak has been chosen as the basis for the assessment. Traffic volumes, the share of turning traffic, and the share of specific vehicle types are given in Table 15.

Lane	Through (–)	Left (–)	Right (–)	PC (–)	bike (–)	HV (–)	q (veh/h)
NR	0.83	0.00	0.17	0.99	0.01	0.01	283
NL	0.87	0.13	0.00	1.00	0.00	0.00	171
EC	0.87	0.00	0.13	0.92	0.07	0.01	209
SR	0.80	0.00	0.20	0.94	0.03	0.03	308
SL	0.78	0.22	0.00	0.95	0.00	0.05	182
WR	0.74	0.00	0.26	0.98	0.02	0.01	351
WL	0.00	1.00	0.00	0.99	0.01	0.00	94

Table 15: Vehicle volumes and share of movements at example intersection (A 046, AP)

16 Variale stage sequences could be taken into account by adjusting lost times for specific stage sequences with the probability of their occurence.

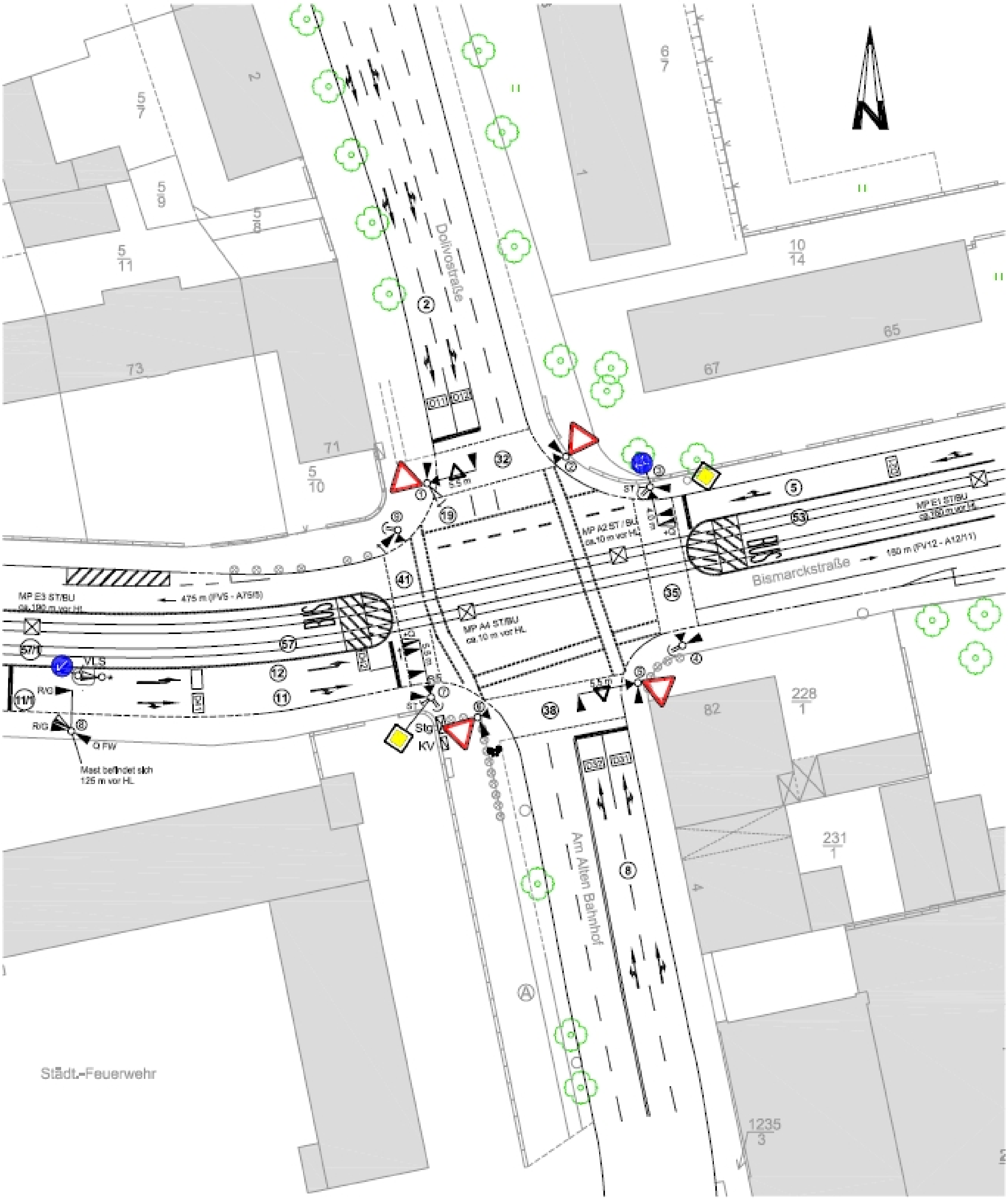


Figure 30: Example intersection (A 046; without scale; source: Stadt Darmstadt 2009)

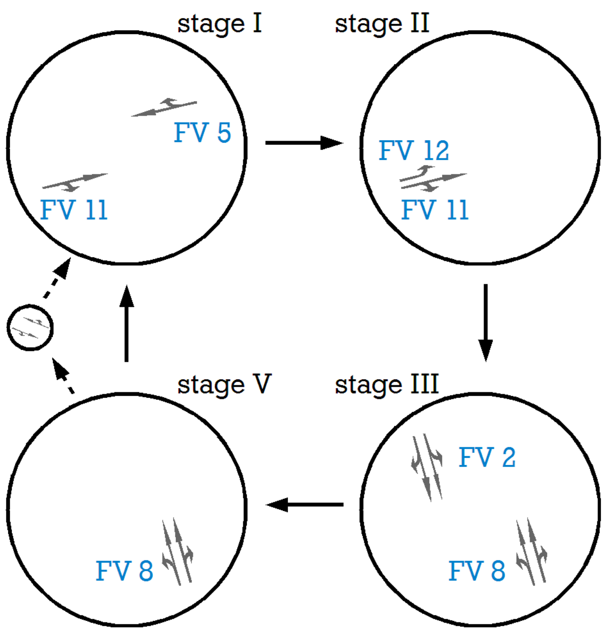


Figure 31: Stage sequence of example intersection (A 046)

5.4.2 Effective capacity

The effective capacity is calculated according to Eq. 33. The parameters needed for all lanes are:

- start-up lost times t_{SUL}
- crossing times of clearing vehicles t_{cr}
- manoeuver times of entering vehicles Δt_{PE}
- signalled green time t_G
- saturation headway h_s

Crossing times of entering and clearing vehicles are determined according to Section 5.3.2 and Section 5.3.3. As has been shown, the crossing time of entering vehicles $t_{\mathrm{cr,e}}$ determined during the surveys seems to be sufficiently independent from the local situation. An average value of $t_{\mathrm{cr,e}} = 1.2\,\mathrm{s}$ is used.

The same applies to the cumulated headway difference $\Delta h(k)$, which is taken as 1.0 s with $k = 4$. The headways at the different approaches have been shown in Figure 22. Since the turning vehicle share and the share of heavy vehicles is very low (cf. Table 15), it seems to be justified to assume equal cumulated headway differences.

Crossing times of clearing vehicles generated from the video observations at the example intersection did not deliver the required accuracy. Therefore they are taken from intersection A 086 as an intersection with similar conditions as A 046. Since the crossing times vary significantly (cf. Table 12), here appreciable uncertainty is introduced.

For the left turning lane on the western approach, the queue length was commonly less than five vehicles. No saturation headway could be determined here. A value of 1.9 s is assumed. For queues shorter than five vehicles, the cumulated headway will not reach the full value. It is, hence, ajusted to account for this situation by 0.2 s (assumed average queue length of two vehicles; $\Delta h(4) - \Delta h(2) = 0.2\,s$).

With the respective saturation headways, the green time differences and the effective capacity can be calculated. The different parameters are summarised in Table 16. The interaction times are left out for the reasons described before. The actual signalled green times are variable due to the traffic actuation. Here average values are chosen. The difference between calculated and effective capacity is derived from the green time extensions, the saturation headways, and the cycle time of $t_C = 90\,s$.

Lane	$\mathbf{h_s}$ (s)	$\mathbf{t_{SUL}}$ (s)	$\mathbf{t_{cr}}$ (s)	$\mathbf{{}'t_{gG}}$ (s)	$\mathbf{t_G}$ (s)	$\mathbf{C_{eff,i}}$ (veh/h)	$\mathbf{C_{calc,i} - C_{eff,i}}$ (veh/h)
NR	1.9	0.3	1.6	1.3	20	448	27
NL	1.9	0.3	1.6	1.3	20	448	27
E	1.8	0.2	1.6	1.4	26	608	31
SR	1.8	0.4	1.6	1.2	25	582	27
SL	1.8	0.4	1.6	1.2	25	582	27
WR	2.0	0.3	1.6	1.3	40	826	26
WL	1.9	0.1	1.6	1.5	12	284	32
All						**3778**	197

Table 16: Calculation of C_{eff} at A 046 (afternoon peak)

The *effective capacity* can be compared to the theoretical capacity according to FGSV (2001). The adjustment factors for the saturation flow under standard conditions, $q_{s,St} = 2000\,{}^{veh}/_{h}$, and the resulting capacities are given in Table 17. Details on the calculation can be found in Appendix A.1.1.

Lane group	**Adjustment factors**	$\mathbf{q_S}$ (veh/h)	**N** (-)	$\mathbf{\frac{t_G}{t_C}}$ (-)	**C** (HBS) (veh/h)
N	lane width	1870	2	0.22	831
E		2000	1	0.29	578
S	radius, heavy vehicles	1935	2	0.28	1075
WR	lane width	1800	1	0.44	800
WL	lane width	1800	1	0.13	240
All					**3524**

Table 17: Saturation headways and capacity according to FGSV (2001)

The effective capacity can also be compared to capacity estimates based on TRB (2000). However, the calibration of the model to American traffic conditions leads to much lower capacities. In Table 18 saturation flows per lane and the capacity following a simplified application of the HCM method are given. Adjustments of lane width and lane utilisation are neglected, the former due to the different conditions in the United States and Germany, the latter due to the lack of relevance for general conclusions. The unadjusted saturation flow rate from HBS ($q_{S,St} = 2000\,{}^{veh}/_{h}$) has been used. Details on the calculation can be found in Appendix A.1.2.

The *effective capacity* of the intersection of about 3800 veh/h differs by about 300 veh/h from the capacity calculated according to FGSV (2001). TRB (2000) leads to an even lower capacity of only about 3450 veh/h. The error attached to the calculation of the efffective capacity is analysed in Section 5.4.4.

Lane group	HV (%)	P_{LT} (-)	P_{RT} (-)	f_{HV} (-)	f_{LT} (-)	f_{RT} (-)	N (-)	$\frac{t_g}{t_C}$ (-)	q_S ($^{veh}/_{h}$)	C (HCM) ($^{veh}/_{h}$)
N	0.01	0.13	0.17	0.99	0.84	0.97	2	0.24	1628	771
E	0.01	–	0.13	0.99	1.00	0.98	1	0.30	1946	592
S	0.04	0.22	0.20	0.96	0.83	0.97	2	0.29	1551	903
WR	0.01	–	0.26	0.99	1.00	0.97	1	0.46	1912	877
WL	0.00	1.00	–	1.00	0.95	1.00	1	0.15	1900	285
										3429

Table 18: Adjustment factors and capacity according to HCM

Capacity reductions caused by intergreen times

By relating the intergreen times to the ending stage, the intergreen time can be converted into capacity reductions. The effective capacity reductions will be lower due to the effective green time being longer than the signalled green time (positive green time difference). The capacity reductions due to intergreen times at the survey intersection for the three stage changes mentioned amount to $\Delta C_{t_{ig}} \approx 700\,^{veh}/_{h}$ without consideration of green time differences and $\Delta C_{t_{ig}-\Delta t_{gG}} \approx 550\,^{veh}/_{h}$ with consideration of green time differences (Table 19).

Ending stage	Lanes	t_{ig} (s)	h_s (s)	Δt_{gG} (s)	$\Delta C_{t_{ig}}$ ($^{veh}/_{h}$)	$\Delta C_{t_{ig}-\Delta t_{gG}}$ ($^{veh}/_{h}$)
I	E	5	1.8	1.4	111	80
II	WR	8	2.0	1.3	160	134
	WL	8	1.9	1.5	168	137
III	NR	6	1.9	1.3	127	99
	NL	6	1.9	1.3	127	99
Overall					**693**	**549**
Averaged	(per lane)		1.9	1.4		**($\Delta = 40$)**

Table 19: Capacity reductions due to intergreen times (A 046, AP)

It should be mentioned that the capacities thus calculated are only reached for saturated conditions. The difference to the actual vehicle volumes is based on the traffic actuated program and the simplifications made.

5.4.3 Maximum capacity

5.4.3.1 Exemplative quantitative results

Basis for the calculation of the maximum capacity is the conflict tree (cf. Section 3.3.4). For every conflict the speed, distance, and time differences (cf. Section 3.3.2) have to be calculated. The overall intergreen time difference depends on the stage sequence. For variable stage sequences, the probability of the different possible stage sequences would have to be considered. Here – as has been mentioned before – a fixed sequence is assumed to improve clarity. Furthermore, the actual traffic actuated signal program showes only marginal variations, the result is therefore a good estimate for the actual situation.

Intergreen time differences

The intergreen time differences $\Delta t_{ig,m,i}$ for the individual movement sequences are determined according to Eq. 20 together with Eq. 21 and Eq. 22. The parameters needed are:

- the difference of the crossing times of entering vehicles $\Delta t_{cr,e,i}$ (cf. Section 5.3.2)
- the difference of the entering time $\Delta t_{e,i}$ (calculated from differing entering distances and entering speeds)
- the difference of the crossing time $\Delta t_{cr,i}$ (cf. Section 5.3.3; the same effective crossing times of through vehicles and turning vehicles have been assumed)
- the difference of the clearance time $\Delta t_{cl,i}$ (calculated from differing clearance distances and clearance speeds)

The entering speed has been calculated based on the speed measurements at the example intersection (cf. Section 5.3.4, Table 13). The clearance speed is derived from the measurements from non-coordinated approaches (Table 14). Since it equals the clearance speed in the Guidelines, no difference occurs. Systematic distance differences are taken into account as described in Section 3.3.2.2. The variation error has been estimated from the local situation. Only the distances in connection with left turning vehicles are corrected. The variation of vehicle length has been neglected.

The intergreen times and the intergreen time differences for all signal group combinations are given in Table 20. The complete conflict tree of the survey intersection is shown in Appendix B.3.

Stage		**Signal group**		$t_{ig,s,i}$	$\Delta t_{ig,g,i}$
Clearing	Entering	Clearing	Entering		
(-)	(-)	(-)	(-)	(s)	(s)
1	2	5	11	0	0.0
1	2	5	12	5	-4.8
1	2	11	12	0	0.0
2	3	11	2	5	-5.8
2	3	11	8	8	-7.3
2	3	12	2	6	-4.9
2	3	12	8	8	-7.1
3	5	2	8	6	-5.5
5	1	8	5	8	-7.5
5	1	8	11	5	-5.8

Table 20: Intergreen times and intergreen time differences for example intersection

Capacity improvement potential

With the cumulated intergreen differences for signal groups $\Delta t_{ig,g,i}$, the green time extensions $\Delta t_{G,i}$ can be obtained. An optimisation with the Simplex Algorithm delivers extensions as shown in Table 21. With the saturation headways and the number of lanes, the capacity improvement potential can be calculated according to Eq. 32. The results are listed in the last column of Table 21.

The capacity improvement potential ΔC_{max} (based on times calculated to an accuracy of the tenth of a second) amounts to more than 1400 veh/h. This is the maximum improvement potential for the survey intersection under the prerequisite of complete information and deterministic driver behaviour.

Approach	Signal group	Δt_G (s)	No of lanes (-)	h_s (s)	$\Delta C_{max,i}$ ($^{veh}/_{h}$)
N	FV 2	6.7	2	1.9	282
E	FV 5	0.0	1	1.8	
S	FV 8	8.2+8.6+5.8=22.6	2	1.8	1004
W	FV 11	1.7	1	2.0	34
W	FV 12	0.5+4.9=5.4	1	1.9	114
					1434

Table 21: Green time extensions and resulting capacity improvements for the example intersection

Consideration of saturation headways at the conflict points

The *maximum capacity improvement potential* is calculated under neglect of post-encroachment times. Even under optimal conditions, the capacity could not further be improved with respect to intergreen times. In reality post-encroachment will have to be considered, reducing the capacity (cf. achievable capacity, Section 4.2.5). The post-encroachment time needed for safety reasons is not in the focus of this research. However, when comparing the signal change interval to a continuous stream, it is apparent, that the capacity calculated without consideration of post-encroachment times will lead to higher values (at least at the decisive conflict point) than the capacity of the continuous stream. In a continuous stream the minimum headway is the saturation headway. It is, therefore, of interest to consider the saturation headway at the conflict point.

The saturation headway consists of the time needed for the vehicle to cover its own vehicle length and the net time gap to the following vehicle. The former time is already considered at the conflict area. The post-encroachment time has to equal the mentioned net time gap to fulfill the requirement explained above. If an average vehicle length of five metres is assumed and the clearance speed is set to $v_{cl} = 10\,^{m}/_{s}$, the saturation headway has to be reduced by half a second to deliver the post-encroachment time.

The post-encroachment time reduces the intergreen time differences and, thus, the green time extensions. The capacity improvement potential with consideration of saturation headways at the conflict points can be calculated with the reduced green time extensions. The green time extensions of Table 21 are consequently recalculated with intergreen time differences (cf. Appendix B.3) reduced by the post-encroachment time of $t_{PE} = h_{s,i} - 0.5$ s. The resulting capacity improvement potential (Table 22) is still more than 80 % of the maximum one.

Approach	Signal group	Δt_G' (s)	$\Delta C_{max,i}$' ($^{veh}/_{h}$)
N	FV 2	6.7-1.4	223
E	FV 5	0.0	
S	FV 8	8.2+8.6+5.8-3·1.3	831
W	FV 11	1.7	34
W	FV 12	0.5+4.9-1.4	105
			1193

Table 22: Green time extensions and resulting capacity improvements for the example intersection (saturation headway at conflict points)

5.4.3.2 Analysis of the result

Minimum intergreen times

For the example intersection the results show that integreen times could be nearly reduced to naught under the assumptions on which the maximum improvement potential is based (cf. Table 20). The decisive intergreen time differences for the signal group combinations range between 80 % and 120 % of the decisive intergreen times calculated according to the German Guidelines for Traffic Signals (RiLSA) with the average being nearly 100 %. The parameters leading to this extreme potential reduction of intergreen times are highlighted in the following paragraph.

Relative importance of parameters

The relative importance of the different parameters for the improvement potential at the survey intersection is given in Table 23 and Figure 32. For the dermination of the different parameters refer to Section 3.3 on page 36.

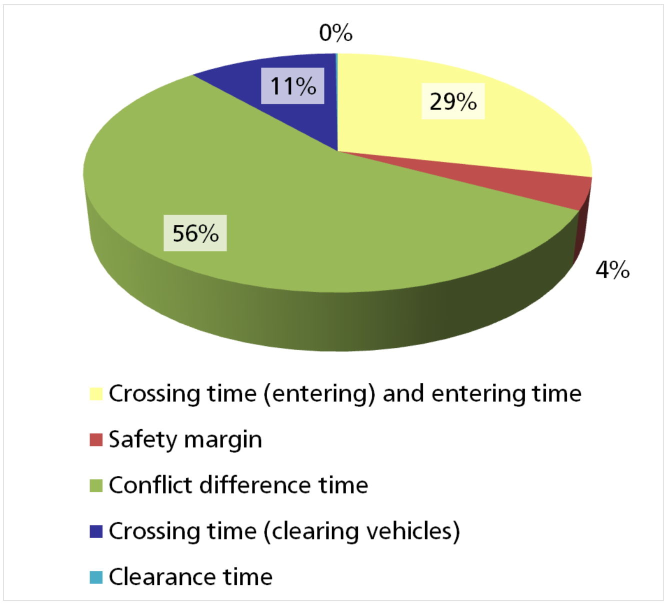


Figure 32: Relative influence of different factors on the overall intergreen time difference

Factor		Relative weight
Conflict difference time	$\Delta t_{c,i}$	56%
Starting response and entering time	$\Delta t_{SR} + \Delta t_{e,i}$	29%
Crossing time	$\Delta t_{cr,i}$	11%
Safety margin	t_{saf}	4%
Clearance time	$\Delta t_{cl,i}$	0%

Table 23: Relative influence of different factors on the overall intergreen time difference

To illustrate the role of the determining conflict, in Table 24 the determining conflicts for the example intersection are given together with the probability of their occurence. The conflicts with particularly long intergreen times are highlighted. It can be seen, that always either bicycles or turning traffic is involved, both of which have commonly only low probability of occurence. Specifically the bicycle traffic leads to long intergreen times, even if only very few cyclists are present at the particular intersection.

t_{ig}	clearing direction	clearing vehicle	entering direction	p_i
5	right	MV	left	0.13
5	through	bike	left	0.07
10	through	bike	right	0.01
5	through	MV	left	0.09
8	through	bike	right	0.00
8	through	bike	right	0.00
6	left	bike	left	0.00
8	left	bike	through	0.00
6	right	MV	left	0.04
6	through	MV	left	0.21

Table 24: Determining conflicts for the example intersection

This, of course, only applies to large intersections, where bicycle traffic leads to longer intergreen times than motorised vehicles.[17] The influence of certain movement sequences on the capacity should therefore always be considered together with the likeliness of the underlying movements.

Conclusions for the number of stages

Commonly the number of stages is kept as low as possible to improve the capacity as long as no safety concerns have to be raised. As could be seen, the intergreen times don't have to be of major importance in this context. If moving starts can be excluded and unfavourable conflicts are prohibited, the intergreen times could be significantly reduced. Taking the effective capacity into account, the capacity will be only insignificantly reduced by the intergreen times for additional stages.

However, the main reason for capacity reductions by additional stages has to be seen in the number of lanes receiving green time simultaneously. Because the total capacity is the sum of the lane capacities and the lane capacities will be reduced on average by additional stages, the capacity will be lower the more stages are used. This holds only true, of course, as long as the capacities of permitted streams is higher than the capacity reductions by additional stages.

For the example intersection, for instance, stages three and five could be reduced to one stage. In this way signal group FV 2 would receive additional green time. The capacity would be higher. However, the capacity for the left turning traffic of FV 8, which would be permitted only in this case, could be too low to serve the demand.

Thus, the capacity impact of additional stages has to be evaluated individually. It depends mainly on the capacity of permitted streams, number of lanes and saturation headways, and commonly to a minor degree, depending on the intergreen time calculation method, on the intergreen times.

5.4.4 Uncertainty of model results

5.4.4.1 Introduction

The variation of the model parameters depends on the variation of the population (true variation of input variables), on the measurement error, and on the error introduced through simplifications. The former two are not distinguished here, because both the variation of the underlying population and

[17] Following FGSV (1992), bicycles become determining for clearance distances longer than 17.4 m.

the measurement error increase the uncertainty of the model results in the same way. The measurement methodology was assessed separately to detect deficiencies. The results of the uncertainty analysis underline parameters of pivotal importance for the accuracy of the model.

The measurement error depends on the data acquisition methodology. Technical details on the used devices are given in Appendix B.1. The methodology of the speed measurements has been explained in Section 5.2.2.4. Most other results are obtained from video observations. The accuracy of video evaluations depends on the accuracy of the video, the accuracy of the evaluation software, and partly on the reaction time and diligence of the person evaluating the video.

The accuracy of the video is determined by the vantage point, the resolution, the video format, and the quality of the recording. While MJPEG videos could be evaluated framewise, leading to a maximum precision of 0.03 s (30 frames per second for the AXIS camera), MPEG-2 videos (Panasonic camera) could only be evaluated in realtime. Details on the video formats and the evaluation procedure are given in Appendix B.1.3. While a framewise evaluation is only limited by the video itself and the steadiniess of the framerate, a realtime evaluation introduces errors due to the reaction times of the evaluating person. Comparative tests with different persons proved that this error ranged well below 0.1 s on average (Todt 2009). Moreover, reaction times lead to a shift towards one direction (which are cancelled out due to the calculation of time differences only).

The influence of the variation of the variable populations and the measurement error on the results of the capacity model can be judged by using the laws of error propagation. The results are outlined in Section 5.4.4.2 with the mathematical background given in Appendix A.5. The role of simplifications and assumptions is touched upon in Section 5.4.4.3. Conclusions on the overall accuracy of the results obtained by the model application are drawn in Section 5.4.4.4.

5.4.4.2 Accuracy of model output

To be able to judge the outcome of the model, the uncertainties involved in the calculation of the capacities have to be considered. Since the model uses means, the variation of the mean, i.e. the standard error $\delta = \frac{\sigma}{\sqrt{n}}$, is of interest (with σ being the standard deviation of the observed variable and n being the sample size). The standard error for effective capacity and maximum capacity improvement is derived in the following sections.

Determination of error of the effective capacity

The error involved in the model application with reference to the effective capacity is summarised in Table 25. It can be seen that the error has a similar magnitude than the capacity differences between effective and theoretical capacity. However, most of this error stems from the uncertainties involved in measuring the saturation headways which influences the error for the green time differences, too. The green time differences can be forecasted to an accuracy of about one second. It is apparent that for an accurate determination of the effective capacity headway data of high quality is crucial. To reduce the standard error, big sample sizes are paramount.

Entering speed variation

Table 26 gives the means and standard errors for all distance classes, to which the data from intersection A 046 was cumulated. Speed functions were fitted to the mean values, the upper bound (defined by the mean plus the standard error), and the lower bound (mean minus standard error). Based on these three speed functions, the entering times have been calculated. The results for the distance classes are given together with the resulting standard error for the entering time in the last columns of Table 26.

Lane	δ_{t_G} (s)	δ_{h_s} (s)	$\delta_{t_{SUL}}$ (s)	$\delta_{t_{cr}}$ (s)	$\delta_{'t_{PE}}$ (s)	$\delta_{'t_{gG}}$ (s)	$\delta_{C'_{t_{gG}}}$ (veh/h)	$\delta_{C_{t_G}}$ (veh/h)	$\delta_{C_{h_s}}$ (veh/h)	$\delta_{C_{eff,i}}$ (veh/h)	$\delta_{C_{eff}}$ (veh/h)
NR	2	0.3	1.2	0.2	0.1	1.2	26	43	85	99	
NL	2	0.3	1.2	0.2	0.1	1.2	26	43	85	99	
WR	2	0.3	1.1	0.2	0.1	1.1	22	41	88	100	
WL	2	0.3	1.1	0.2	0.1	1.1	23	42	93	105	
EC	2	0.3	1.2	0.2	0.1	1.2	26	44	115	126	
SR	2	0.2	0.8	0.2	0.1	0.9	19	45	125	134	
SL	2	0.2	0.8	0.2	0.1	0.9	19	45	125	134	
All											304

Table 25: Variation of C_{eff} at A 046 (afternoon peak)

The upper limit of the standard error of the entering time of 0.2 s has been applied in the further error propagation calculations.

Distance class	Measured speed		Entering time difference Δt_e			
	Mean	Standard Error	Upper	Mean	Lower	**Standard Error**
(m)	(km/h)	(km/h)	(s)	(s)	(s)	**(s)**
1.0	4.4	0.4	1.3	1.2	1.1	**0.1**
6.0	17.7	0.4	2.0	1.9	1.7	**0.1**
10.0	20.9	0.5	2.1	1.9	1.7	**0.2**
14.0	22.8	0.5	2.0	1.8	1.6	**0.2**
18.0	26.0	0.5	1.9	1.7	1.5	**0.2**
22.5	27.2	0.6	1.7	1.4	1.2	**0.2**
26.0	29.3	0.7	1.5	1.3	1.0	**0.2**
30.0	30.7	0.7	1.3	1.0	0.8	**0.2**
33.5	33.3	1.0	1.1	0.8	0.6	**0.2**

Table 26: Variation of entering time difference (A 046, afternoon peak) with $v_e = 40\,\text{km/h}$ and $\Delta l_e = 0\,\text{m}$

Determination of error of the capacity improvement potential

The error involved in the capacity improvement potential has to be derived from the green time extensions Δt_G which depend on the intergreen time differences of the underlying movement sequences. Since the number of movement sequences per lane n_m varies, only average values can be generally computed. Here, the average number of movement sequences per lane of seven is used. The error of the maximum capacity improvement potential depends on the number of green time extensions $n_{\Delta t_G}$. At the example intersection five green time extensions can be realised.

The conflict difference time and the safety margins can be exactly determined ($\delta_{\Delta t_c} = 0\,\text{s}$ and $\delta_{\Delta t_{saf}} = 0\,\text{s}$). The error for the clearance times is uniformly determined from the speed measurements at non-coordinated approaches (Table 14) and an assumed distance error of 0.5 m.

$$\delta_{t_{cl}} = \sqrt{\delta^2_{t_{cl}\Delta l_{cl}} + \delta^2_{t_{cl}\Delta v_{cl}}} = \sqrt{0.05\,\text{s}^2 + 0.31\,\text{s}^2} = 0.3\,\text{s}$$

The overall error for the capacity improvement potential is given in Table 27. It is less than 10 % of the improvement potential. No salient negative influencing parameter can be discerned. The error results from all model parameters quite evenly.

Signal Group	$\delta'_{t_{SR}}$ (s)	δ'_{t_e} (s)	$\delta'_{t_{cr}}$ (s)	$\delta'_{t_{cl}}$ (s)	δ_{h_s} (s)	$\delta'_{t_{ig,m,i}}$ (s)	δ'_{t_G} (s)	n_l (-)	$\delta'_{C'_{t_G}}$ veh/h	$\delta'_{C_{h_s}}$ veh/h	$n_{\Delta t_G}$ (-)	$\delta'_{C_{max,i}}$ veh/h
FV 2 (N)	0.1	0.2	0.2	0.3	0.3	0.4	1.1	2	48	6	1	49
FV 5 (E)	0.1	0.2	0.2	0.3	0.3	0.4	1.1	1	23	3	0	0
FV 8 (S)	0.1	0.2	0.2	0.3	0.3	0.4	1.1	2	50	7	1	50
FV 11 (WR)	0.1	0.2	0.2	0.3	0.3	0.4	1.1	1	23	3	0	0
FV 12 (WL)	0.1	0.2	0.2	0.3	0.3	0.4	1.1	1	24	3	3	71
All												**100**

Table 27: Variation of ΔC_{max} at A 046 (afternoon peak)

5.4.4.3 Error due to assumptions and simplifications

Simplifications due to lack of model calibration

The error propagation presented in the last section gives the direct influence of measurement errors on the overall result of the model. Inaccuracies resulting from the simplifications described in Section 5.2.3.3 cannot be judged mathematically. These inaccuracies can only be estimated by further extensive surveys. By analysing the influencing factors on the different model parameters (cf. Section 4.3), it can be shown whether or not the assumptions met are valid.

Assumptions on arrival distribution

For the calculation of the probability of movement sequences the random arrival of vehicles was postulated. The probability was based on the total share of certain movements on a lane (cf. Section 3.3.5.3 on page 50). The small sample size makes a reliable test against this postulate difficult. In the peak hours only 80 cycles occur ($t_C = 90$ s), of which some have to be discarded. On average about 60 cycles could be evaluated. The number of first entering vehicles on each lane belonging to a certain movement was compared to the expected number following the total share of this movement on the lane. The differences have been evaluated using Pearson's χ^2-Test.

At a confidence level of 85 % only the eastern lane (E) and northern left (NL) lane follow the assumption, while on the other lanes the number of through traffic is higher than can be explained by random variation (Table 28). However, the likeliness of an error of second type is high if the hypothesis of random arrivals is discarded. Furthermore, through traffic commonly leads to higher intergreen time differences. No reason for the bias of the arrival distribution can be discerned. A random error is likely. Thus, the assumption of random arrivals seems nevertheless to be sensible.

The values in Table 28 are calculated with Eq. 39 and Eq. 40.

$$E_i = p_i n \tag{39}$$

$$\chi^2 = \sum \frac{(O_i - E_i)^2}{E_i} \tag{40}$$

Lane	Share of through traffic p_i (-)	Observed through traffic O (veh)	Expected through traffic E (veh)	Confidence interval p_{χ^2} (-)
NR	0.83	63	58	0.10
NL	0.87	60	60	0.97
WR	0.74	62	52	0.01
E	0.87	37	37	0.87
SR	0.80	39	36	0.26
SL	0.78	29	31	0.45

Table 28: Test for random distribution of first entering vehicles

where			
	p_i	probability of movement sequence i	(–)
	n	total number of vehicles in first position observed on lane	(veh)
	E_i	expected number of vehicles in first position of queue belonging to movement i	(veh)
	O_i	observed number of vehicles in first position of queue belonging to movement i	(veh)
	χ^2	value approaching χ^2 distribution	(veh)
	p_{χ^2}	probability of hypothesis being true without error of first type (confidence interval)	(–)

5.4.4.4 Overall accuracy of model results

As has been justified in Section 5.1.2, the main focus of the survey was on the accuracy of data for single parameters. The standard errors involved in the data collection proof that this aim was achieved. Most time errors remain well below one second. Most of the variation can be attributed to the variation of the population itself. Both the effective capacity and the capacity improvement potential could be determined to an accuracy of less than ten percent of the respective results (about 300 veh/h or 8 % of the effective capacity, 100 veh/h or 7 % of the total improvement potential). This is an adequate result considering the many sources of error cumulating in the overall result.

The inaccuracy of the effective capacity is primarily caused by the measurement of the saturation headway. This error could be significantly reduced by measuring the crossing times of platoons instead of single vehicles. Information on the sequence of headways, which is needed to determine the time until saturation is reached, will, however, not be available thus.

The evaluation method is appropriate to gather high quality data, if the video evaluation is conducted diligently. Attention should be paid, nevertheless, on the assumptions met for the application of the capacity model to single intersections. These assumptions have to be verified by a model calibration as explained in Chapter 4.

6 Conclusions

6.1 Introduction

The presented research had the aim to analyse the influence of intergreen times on the capacity of signalised intersections in detail. The capacity is defined as the sum of all vehicles on all approach lanes able to cross the intersection under optimal conditions. A distinction was made between the base capacity, i.e. the capacity not influenced by gap acceptance behaviour inside of the intersection due to permitted turning traffic, and the (final) capacity. Gap acceptance behaviour was not further analysed.

The basic fundamentals of capacity reductions by intergreen times are summarised in Section 6.2. The detailed capacity analysis beyond these fundamentals was achieved in two steps.

1. Based on the intergreen times as they are, the traffic flow during the signal change intervals was scrutinised to obtain the effective capacity. The effective capacity is influenced by start-up lost times, crossing times of clearing vehicles, interaction times, and the saturation flow together with the signal program (green times and cycle time). The result of this part of the research is summarised in Section 6.3.

2. The second step went beyond the existing situation and focused on the improvement potential of the capacity. This improvement potential was derived from the generic aim of intergreen times to avoid conflicts. This aim is achieved, if clearing and entering vehicles don't use the same conflict areas at the same time. If the trajectories of all vehicles would be known in advance, the intergreen times could be adjusted to a minimum.

   The difference between the thus determined intergreen times and the actually used ones was termed *intergreen time difference*. By assigning these intergreen differences to signal groups, the green times of these signal groups can be extended. These extensions lead to capacity improvements. The detailed conclusions from the application of this procedure are presented in Section 6.4.

The main focus of this research was laid on a sound methodological background and the development of models to generate effective and maximum capacities. Nevertheless, to gain further insight into the signal change intervals, to prove the feasibility of the models, and to get preliminary quantitative results, surveys have been conducted. The survey methodology was developed out of the requirements of the capacity model. This methodology was applied at several signalised intersections in the German city of Darmstadt. For all model parameters data was gathered and processed separately. This data served as input into the capacity model. In this way quantitative statements on the role of intergreen times for the capacity of signalised intersections could be made. These statements are summarised in the following sections.

The application of the capacity model is greatly simplified by calibrating it to specific situations. The surveys served, moreover, the analysis of possible influencing factors on the model input parameters as a first step towards this calibration. The calibration, however, calls for more extensive surveys than could be achieved as part of the present research. Recommendations for the layout of this kind of research together with a critical assessment of the applied survey methodology are presented in Section 6.5.

The conclusions close with a concise summary of the lessons learned from the presented research (Section 6.6).

6.2 Reasons for capacity reductions by intergreen times

6.2.1 Introduction

The example shown in Section 5.4 illustrates that intergreen times could in many situations theoretically be avoided. However, the assumptions leading to this statement (cf. Section 3.1.2.2) are so far not realistic. Furthermore, in most cases even based on these assumptions intergreen times are required. A number of factors can be pointed out that determine whether intergreen times are positive or negative (e.g. ratio of entering distances to clearance distances, ratio of entering speeds to clearance speeds, crossing times). More important than the intergreen times itself is the resulting sum of effective green times as compared to the sum of effective red times at an intersection, because part of the intergreen time – the crossing time of clearing vehicles – is part of the effective green time at the same time.

This section expands upon the mentioned factors, namely the dimension and location of conflict areas, headways at the stop line and the conflict point, and the effect of different entering speeds and entering distances. It is highlighted, under which circumstances intergreen times lead to capacity reductions or, in rare cases, capacity improvements. In this context the role of stage settings and the number of stages for the capacity is also explained.

6.2.2 Dimension and location of conflict areas

Dimension of the conflict area

The difference between the clearance distance with and without consideration of the true conflict area dimension (cf. Section 3.3.2.2 on page 40) will always be greater than the same difference for the entering distance for movements using different exit lanes. The clearing vehicle always has to clear at least part of the conflict area, before the entering vehicle may arrive at it. Figure 5 on page 41 illustrates this fact. The bigger the conflict area is, the more the capacity will be reduced. The size of the conflict area is particularly important at one-lane/two-way facilities (e.g. due to road works). The size of the conflict areas is determined by the angle of intersection and the lane widths. It is apparent that under normal circumstances the dimension of the conflict area leads to a prolongation of the individual intergreen times of tenths of a second only.

Change of decisive conflicts

As has been explained in Section 3.3.5, during each change of stages only one out of commonly several conflicts is decisive for the intergreen time calculation. If only one conflict area would exist, it would be decisive for all stage changes and the sum of entering distances and clearance distances would be equal. The decisive conflict area in reality, however, changes from each stage to the next. Some conflict areas, thus, may be determining only for one specific signal group sequence. This leads to an imbalance of the sum of clearance distances as compared to the sum of entering distances and consequently to longer intergreen times.

This can be illustrated by a simplified intersection without turning traffic (Figure 33). The clearing vehicles are depicted with red triangles, the entering with green triangles. Four conflict areas exist for this simple situation (blue frames). The two decisive ones for the shown change of stages under the prerequisites of an intergreen time calculation neglecting entering processes from a full stop (e.g. RiLSA) are filled in blue. When imagining the next stage change for this two stage program (clearing streams become entering streams and vice versa), the remaining two conflict areas become decisive, because they lead to the longer intergreen times. It can be seen, that the sum of clearance distances is greater than the sum of entering distances for this simple example. The balance of clearance and entering distances will be shifted towards the clearance distances. The value of the difference between the sum of clearance distances and entering distances is determined by the distance between the decisive conflict areas Δl_{CP}. This distance leads to capacity reductions.

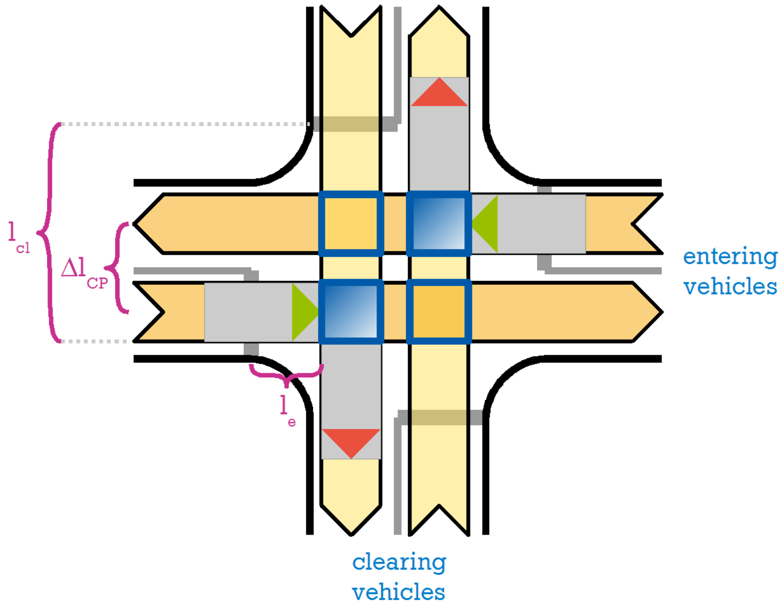


Figure 33: Changing of decisive conflict areas

The stage settings and the control regime have a significant influence on these capacity reductions. The more signal groups are conflated into one stage, the more conflicts exist, usually leading to a greater imbalance of clearance and entering distances. If the control regime is based on stages, the decisive conflict results from the conflicts of all signal groups. With a signal group based control, clearance and entering distances will be more balanced. This can be illustrated by the simple example in Figure 33. The situation described before applys to a stage based control with two stages only. For a four stage program, the ratio of clearance to entering distances depends on the stage sequence. If the movements receive green time clockwise, always the conflict areas nearest to the respective stop line of the clearing stream is decisive. In this way the clearance distances can be minimised and the entering distances maximised, leading to shorter intergreen times per change of stages than with a four stage program. But even with a stage sequence less favourable, the intergreen times will never be longer than for the two stage program.

The change of the decisive conflict area is the reason for conflict areas remaining unoccupied during certain intervals of the cycle. The conclusion is that conflict areas for signal programs with few stages should be as close together as possible to increase the capacity.

6.2.3 Headways at the stop line and at the conflict point

Cumulated headway differences

Each entering process leads to cumulated headway differences (cf. Section 3.2.2 on page 32). These headway differences with reference to the saturation headway reduce the capacity. Each change of stages, thus, leads to a capacity reduction due to the acceleration process of the entering vehicles. Only in case of moving starts, where no headway differences occur, this capacity reduction can be avoided. In case of moving starts, however, the crossing time of entering vehicles has to be considered, which may lead to capacity reductions.

Post-encroachment time and saturation headway

The minimum intergreen time was calculated based on the assumption that no post-encroachment time occurs. Assumed that during each stage only one vehicle per lane would cross the intersection, no net headways would occur. Only the time needed by the clearing vehicle to cover its own vehicle length would result in a gross headway. This would lead to higher capacities than with longer stages where gaps between vehicles driving in the same direction will occur. However, if this assumption can be realised, the headways between vehicles belonging to the same movement could be avoided, too. This capacity improvement is, therefore, only a perceived one, resulting from the compromise between a simple and yet realistic model.

Nevertheless, if the minimum headways between the front bumpers of clearing and entering vehicles not leading to interaction times (cf. Section 3.2.4) should be shorter than the saturation headway, a capacity improvement may in fact be realised by stage changes. The effect, however, will be marginal, if it can be realised at all.

6.2.4 Entering speed and entering distance

For equal entering and clearance distances and equal entering and clearance speeds the entering time and the clearance time cancel each other out. If the entering speed is less than the clearance speed, the entering time will be greater. The difference between entering time and clearance time will increase with longer entering distances. Intergreen times will consequently be shorter for lower entering speeds and longer entering distances.

Entering speed and capacity

As long as moving starts are the exception, clearance speeds will commonly be greater than equivalent entering speeds (cf. Section 3.3.2.3). Entering vehicles will consequently need more time to cover the same distance than clearing vehicles. Lower entering speeds, however, will also lead to greater headways at the stop line. The cumulated headway difference will increase. The reduction of entering speeds will, hence, lead, on the one hand, to a greater maximum capacity, on the other hand it will decrease the effective capacity.

Entering distance and capacity

If the entering speed is less than the clearance speed, entering distances are inverse proportional to intergreen time. The longer the entering distance will be, the shorter the intergreen time will be. This will lead to a capacity improvement. The reason for this improvement is the platoon behaviour illustrated in Figure 34.

The platoon waiting at the intersection for green accelerates from a full stop. The last vehicle of the platoon will accelerate to the clearance speed with which it crosses the intersection. If the crossing time of the first vehicle and the last vehicle is determined at different cross-sections, the difference between the two will be lower the further the cross-section is displaced from the stop line, as long as the first entering vehicle accelerates. The time during which this cross-section is occupied by the platoon t_{occ} is, thus, shorter, the greater the distance between the stop line and the cross-section is.

If the cross-section intersects with the conflict point, the consequences for the capacity become apparent. The conflict point is occupied for a shorter time if it is further away from the stop line ($t_{\mathrm{occ}}(\mathrm{CP}_1)$ vs. $t_{\mathrm{occ}}(\mathrm{CP}_2)$). The shorter the conflict point is occupied by one movement, the earlier it can be occupied by the subsequent one. In this way capacity improvements can be achieved by increasing the distance between the stop line and the conflict point.

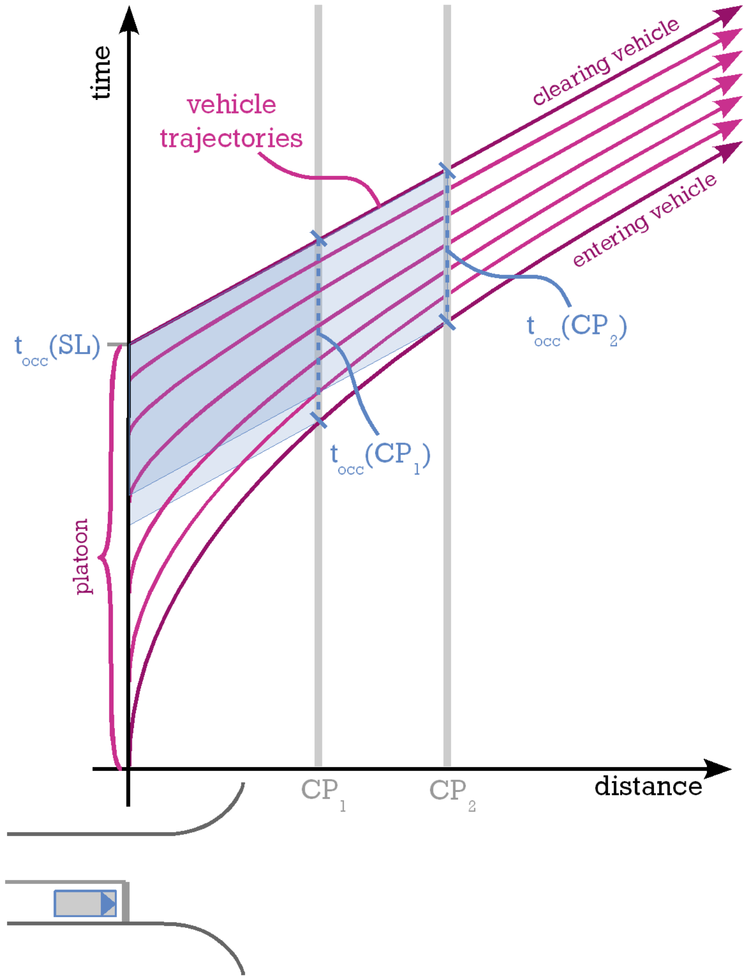


Figure 34: Illustration of occupancy time of conflict points

The described effect can be observed only, if the first entering vehicle accelerates between the stop line and the conflict point. The distance between stop line and conflict point, however, has not only an effect on the entering times, but depends on a number of factors mainly connected to the intersection layout. Nevertheless, displaced stop lines may not lead to capacity reductions, but can be even advantageous for the intersection capacity.

6.2.5 Stage settings

The stage settings have a major impact on the capacity of signalised intersections. Concerning the base capacity without consideration of gap acceptance, the major impact is the number of stages. The higher the number of stages the less lanes receive green at the same time resulting in lower lane capacities and consequently in lower intersection capacity. It could be shown that the intergreen times play only a minor role in this context. However, permitted streams have to be taken into account. The capacity can be significantly lower than the base capacity due to permitted turning traffic.

The stage settings have also an impact on the movement sequences and resulting conflicts. The major part of the capacity improvement potential with reference to intergreen times stems from conflict difference times, i.e. intergreen times not required due to the low probability of certain conflicts. While these intergreen times have to be considered due to safety reasons, it may be worthwhile to prevent very rare conflicts with long intergreen times totally. Which conflicts should be considered in this sense can be derived from the conflict tree described in Section 3.3.5.3.

6.3 Effective capacity of signalised intersections

6.3.1 Conclusions with reference to the state-of-the-art

While the effective capacity of signalised intersections has been in the focus of several research projects in the past, the results of the research presented here proved that still not all questions have been answered. While traffic follows the same principles all over the world, namely in industrialised countries with more or less homogeneous vehicle mix, calculation procedures for capacities vary.

The U.S. Highway Capacity Manual (TRB 2000) already uses effective green times which account for most capacity impacts of intergreen times. Only interaction times are neglected. The German Highway Capacity Manual (FGSV 2001), on the other hand, does not define start-up lost times, crossing times, interaction times, and, consequently, effective green times at all. But particularly in Germany, due to short start-up lost times, the effective green time varies significantly from the signalled green time. The specific aspects of intergreen times, following the capacity estimation according to FGSV, can only be indirectly factored into the saturation flow. This procedure is worth reconsideration.

6.3.2 Conclusions from the empirical data

Most of the research conducted on effective capacity was done in the United States. It is no surprise that the results vary from European, namely German, research. Three major differences concerning the situation at signalised intersections can be distinguished:

- the application of transition times between red and green (yellow-and-red)
- the regulations concerning yellow times (permissive or restrictive yellow rule)
- the position of signal heads

The effective capacity can be reduced by start-up lost times and interaction times while it increases due to vehicles crossing the stop line after the end of the green interval (crossing times of clearing vehicles).

Start-up lost times

The transition time between red and green (one second yellow-and-red) in Germany leads to nearly negligible start-up lost times. Cumulated headway differences of the first entering vehicles are nearly compensated by very short crossing times of entering vehicles (including starting response times and the time to reach the stop line). The transition time of one second is more than enough for the majority of drivers to perceive the impending signal change and react to it. Due to small stop line distances (the distance between the stopping position of the first vehicle in the queue and the stop line), even the crossing time of the first entering vehicle as the determining factor for start-up lost times remains below one second.

As compared to start-up lost times of two seconds mentioned in TRB (2000), this value shows the improvement potential by transition times and short stop line distances. The application of a transition time between red and green is highly recommended. While the use of count down signals is restricted in context with traffic actuation, a pretimed interval of one second is perfectly sufficient. Research showed even a lack of influence of a longer transition interval.

Crossing times of clearing vehicles

The situation for clearing vehicles is different. While average crossing times are always positive, they vary significantly among intersections. This variation leads to a low predictability. Unfortunately, the surveys conducted for this research could only give some indications on the main influencing factors. Apparently, the pressure on the drivers expressed by their disposition, the green split, and the traffic demand play the decisive roles.

Despite the variation of the data it is apparent, that crossing times lead to significant (around two seconds on average) green time differences, i.e. the differences between signalled and effective green. To neglect the crossing times of clearing vehicles leads to an underestimation of the capacity.

Interaction times

Not considered for the determination of effective green times so far are interaction times. Interaction times occur if the first vehicle in the queue delays the entering process (by either starting late or accelerating more moderately). These interaction times depend on the quality of the intergreen time calculation. Following the common rationale of intergreen times, interaction times should be forestalled.

The main importance of interaction times has to be seen in the context of permitted turning traffic. Vehicles waiting in the intersection for gaps in the opposing traffic commonly clear late and lead to a delay of entering vehicles. This situation was excluded from the present research. The likeliness of interaction times for other conflicts was theoretically analysed. The results show only low significance. However, despite the difficulties of determining the quantitative role of interaction times since they cannot be directly observed, a methodology for surveys providing an estimation of the impacts was developed.

6.3.3 Overall conclusions for the effective capacity

The effective capacity of signalised intersections is significantly higher than a capacity, calculated without consideration of effective green times, but based on saturation flow and signalled green times only (as in the German Highway Capacity Manual, HBS). At the example intersection the difference amounted to about 7 %. To calculate an *effective saturation flow*, as is implicitly proposed in HBS, appears to be misleading. The procedure proposed by the HCM is more straight forward and describes explicitly the processes leading to green time differences. The procedure in HBS should be revised accordingly.

The main reason are crossing times of clearing vehicles, i.e. the interval between the end of the signalled green time and the crossing of the last vehicle. Start-up lost times are at least compensated by these crossing times. Short start-up lost times can be achieved by indicating the impending signal change by a transition time (e.g. red-and-yellow), and fostering low stop line distances. If intergreen times are too short, drivers of entering vehicles may adjust their behaviour which leads to interaction times. Commonly these interaction times can be neglected.

6.4 Optimisation potential and recommendations

6.4.1 Relevance of the capacity improvement potential

The main part of this thesis covers the systematic derivation of a model to compute the maximum optimisation potential of the capacity of signalised intersections with respect to intergreen times. While this optimisation potential does not account for safety issues, it nevertheless gives an upper limit of what *could* be achieved by an optimisation of intergreen times.

For three reasons the developed methodology bears high significance for the performance evaluation of signalised intersections:

- Despite the high efforts spent on improving calculation procedures for intergreen times, many doubts have been raised on the quality of the presently prevailing methods (from a safety perspective!).
- Research areas could be identified which are suitable for an improvement without safety concerns.
- A capacity assessment is required to guide the research towards areas worth scrutiny from both a safety *and* capacity point of view.

These three points are further highlighted in the following sections.

6.4.2 Perceived and verified safety connected to intergreen

As has been pointed out before, traffic flow and signal control follow the same principles all over the world. Nevertheless, no unique calculation method for intergreen times could be established so far. Partly this results, of course, from the different driver behaviour, vehicle mix and so forth in different countries. Nevertheless, the intergreen time calculation procedures vary more than this will account for. Even on fundamental principles no unanimity could be achieved so far.

From the literature review five important unanswered questions can be deducted. These questions apply to the definition of intergreen times, the objective and acceptance of intergreen times, and the safety of intergreen times and its assessment.

- What are intergreen times?

  Inter-green times are intervals between the end of green of one signal group and the beginning of green of a conflicting signal group. But does “green” refer to *signalled green* or to *effective green*? While in Germany the term *intergreen time (Zwischenzeit)* is well established and refers to signalled green, in the United States the term *signal change interval* is more common. Yellow time and all-red (or red clearance) time are explicitly distinguished. One reason can be found in the yellow interval calculation, which cannot be separated from the intergreen determination in the United States. Consequently the different objectives and characteristics of yellow time and crossing time are not distinguished. In Germany, in opposition, a uniform yellow interval is used.

- How is the responsibility for the safety at signalised intersections shared between drivers and traffic engineers?

  Research showed that drivers are well capable to react to critical situations. The more responsibility is taken from the drivers by high safety margins the more they tend to loose this capability. Consequently, the question has to be raised, whether intergreen times have to ensure the complete clearance of all conflict areas before entering vehicles can arrive, or whether it is sufficient to make drivers aware of conflicting vehicles still in the intersection. The former objective is commonly set for passenger cars, while the latter is sometimes applied, for instance, to tramways or lorries.

- Are intergreen times accepted by drivers?

  So far intergreen times are calculated with a kinematic model underlying. This model implies deterministic driver behaviour. However, it is apparent that drivers react not only to the specific signals for them, but also on the individual situation as has been explained before. Signals have to be perceived as reasonable by drivers; otherwise compliance will deteriorate. The common perception that long intergreen times are safer disregards this fact.

- How can the safety of intergreen times be assessed?

  Traffic is only partly predictable. Parameters needed for the intergreen time calculation vary randomly. Safety margins, hence, are necessary to ensure safe intergreen times for driver behaviour deviating from the mean. While different procedures to compute safety margins have been proposed, so far the safety margins are only insignificantly correlated to the variability of the input parameters. The probability of red running, for instance, can be higher than the probability of a moving start at an individual intersection as could be observed during surveys. Nevertheless, the former is not considered (commonly justified by the illegal driver behaviour), while the latter is considered. Moreover, short term safety improvements may deteriorate over the years due to driver adoption to new signal settings. Longitudinal long term studies, however, are rare.

- What safety level is desirable?

  Since the driver behaviour varies randomly, the deviation from the average behaviour, that should be accounted for, has to be defined. This starts with the definition of speeds to consider (85-percentile? speed limit?) and ends with the decision on the exclusion of illegal behaviour. The latter point is commonly agreed upon, but still most engineers tend to relieve drivers from their *legally binding* responsibility to drive defensively and react to the situation. The answer to the question for a desirable safety level, however, bears no consequences as long as we do not know how to compute the safety level (see *"How can the safety of intergreen times be assessed?"*).

All these points proof that it is inappropriate to justify all capacity losses due to intergreen times with an improved safety. The U.S. Signal Timing Manual (Koonce et al. 2008) even underlines the necessity to balance safety and capacity (with reference to red clearance intervals). Many issues have still to be addressed. The prioritisation of further research should be based not only on safety concerns, but should bear capacity impacts in mind. The methodology developed for this thesis provides the means for the capacity part of an overall assessment.

6.4.3 Recommendations for optimisation

Intergreen times can be optimised without safety concerns for three reasons:

- Some extensions of the intergreen times are not justified by the traffic flow.
- Conflicts which lead to long intergreen times (e.g. turning traffic) can be prevented.
- The variability of input parameters for the intergreen time calculation can be decreased, which reduces the magnitude of required safety margins.

6.4.3.1 Superfluous intergreen time extensions

Every extension of the intergreen times, generally speaking, leads to a reduction of intersection capacity. Naturally a lower limit exists. An exceeding of this limit leads to interaction times. Nevertheless, every extension of the intergreen times which does not lead to an improved safety should be avoided to increase capacity. "Safety" margins, for instance, which are not based on a verifiable safety improvement, fall into this category. The arbitrary rounding of results in Germany belongs to these "safety" margins.

Of high relevance for the capacity is the entering time. Particularly at non-coordinated approaches effective entering times easily reach values of five seconds. Due to the high relevance of entering times for the risk of right angle collisions, care has to be taken when reducing intergreen times by the entering times. However, at saturated intersections without coordination, moving starts can apparently be safely excluded. By reliable traffic detection the presence of stopped vehicles can even be assured. And particularly for saturated conditions the capacity is of importance. The feasibility of a distinction between coordinated and non-coordinated approaches is proven in Austria, where moving starts are considered at coordinated approaches only. The risks and opportunities of vehicle to infrastructure communication technologies should be analysed in the future. On the one hand, these systems can increase the likeliness of moving starts, on the other hand, they can enable the signal controller to predict the vehicle arrivals more precisely. How these consequences can be taken into account in the intergreen time calculation should be subject of future research.

The intergreen time calculation according to ITE (1999) does not account for entering times at all. The German Guidelines for Traffic Signals (RiLSA) always assume a moving start. Intergreen times can be reduced by sometimes several seconds, if entering times are considered and instead of a moving start an acceleration from a full stop is assumed. At the example intersection scrutinised in this research, about a third of the improvement potential (more than 400 veh/h) stemmed from entering and starting response times.

With the increasing dissemination of vehicle-to-infrastructure communication systems, the importance of moving starts will increase. This communication can also be used to predict arrival times and speeds of entering vehicles more precisely.

It is worthwhile to have a closer look at the conditions under which moving starts may be excluded. Entering times could, thus, be considered in their most likely duration. Traffic actuated intergreen times appear to be suitable to further improve the safety of such a measure. If vehicles are present at all approach lanes, no moving start can occur.

6.4.3.2 Conflict assessment

The procedure to derive green time extensions from individual intergreen time differences introduces a maximum of transparency into the capacity assessment of intergreen times. The presented conflict tree not only illustrates a detailed intergreen time calculation procedure, it, furthermore, highlights the impact of individual conflicts under consideration of their likeliness. This likeliness of conflicts together with the intergreen time differences leads to a detailed catalogue of unfavourable conflicts.

The likeliness is derived from the share of certain streams and vehicle types on the examined lanes. The less likely a certain movement sequence is, the less importance the respective movement bears. If, for instance, turning traffic leads to particularly long intergreen times which become determining for a certain signal change, this turning traffic may be prohibited to improve intersection performance. Such a decision has to be based, of course, not only on the intergreen time assessment.

The proposed procedure can also be used to compare different intersection layouts, stage settings, and stage sequences under consideration of the improvement potential. The consequences of layout decisions and their relative importance can be seen from an early stage of the design process. Decisive conflict areas can be determined, which is the basis for assessing a more compact intersection layout (cf. Section 6.2 for the importance of compact intersections for the capacity).

6.4.3.3 Variability of input parameters

Even if it is not the case today, safety margins should be based on the variability of input parameters. This is the fundamental principle of risk analysis, irrespective of capacity considerations. Only in this way, a defined safety level can be realised as precisely as possible.

Safety margins can be smaller, if outliers are less likely and the parameters vary less around a mean. The variation depends not only on the true variation of the population of the respective variable, but also on the predictability, i.e. the variation of the known sample of this variable. Two measures are, hence, suitable to improve capacity performance of signalised intersections without compromising safety in this context:

- Reduce the (true) variability of the input parameters for the intergreen time calculation, and reduce the probability of outliers.

- Reduce the error introduced by insufficient sample size or improper survey methodology for the determination of input parameters.

The latter point is also related to influencing factors on driver behaviour. If the average and variation of certain parameters is strongly correlated to certain factors (e.g. the speed limit), intergreen times should take this correlation into account. In this way, certain parameters may be adjusted to the local situation (as has been realised in the U.S. concerning grade). Relevance of this aspect can be seen both in the clearing and entering behaviour of drivers at signalised intersections.

Clearing behaviour

The application of both mentioned points can be seen in crossing times and clearance speeds. Both vary significantly among intersections, but their distributions show high excess at a single intersection. The surveys conducted as part of this research could not deliver a comprehensive assessment of possible influences on these distributions, but indicated a correlation with traffic demand and green time split.

Entering behaviour

By using a transition time between red and green the crossing times of entering vehicles become more predictable, because drivers orient themselves on these signals in the first place instead of signals of crossing traffic or such. For further remarks on entering times refer to Section 6.4.3.1 above.

6.4.4 Conclusions for intersection layout and signalisation

To identify the optimisation potential of intergreen times, it is of primary interest to disclose safety gaps. However, if several issues with safety relevance are raised[18], not only safety, but other aspects – capacity being one of them – should be considered as part of an overall assessment to prioritise further research or possible measures. Without knowledge of the capacity impacts of intergreen times this is not possible. Here some areas are highlighted which offer the potential for capacity improvements, but have to be evaluated from a safety perspective as well.

[18] A number of safety related general questions have been raised in Section 6.4.2 on page 114.

Grade

While in the United States the influence of grade on the acceleration and deceleration behaviour has been introduced in the formulas for yellow time and, consequently, intergreen time in 1982 with good results, in many countries, Germany being one of them, this influence has not been addressed explicitly so far, neither in the yellow time, nor in intergreen times.

Yellow time

The determination of yellow time has long since been debated by traffic professionals. Significant research could show that the stopping behaviour of drivers is mainly influenced by the distance to the stop line. Prevailingly the yellow time duration is, however, determined by a kinematic model based on speeds and decelerations only. Furthermore, the advantages and disadvantages of uniform versus individual yellow times have not been researched comprehensively. While individual yellow times offer the chance for local adjustments and, thus, an optimisation, countries like Germany profit from uniform intervals due to the customisation of drivers to these intervals.

Regardless of these two issues, Köll et al. (2004a) stated that the yellow interval is commonly underestimated by drivers. The consequence are clearance lost times (or more precisely: shorter crossing times than feasible). Too long yellow times may, furthermore, lead to a higher variability of crossing times with the effects highlighted on page 117.

The latter observation underlines the importance of a distinction between crossing time and yellow time. While the crossing time is closely related to the yellow interval, it is nevertheless not linearly correlated to it and depends on a number of factors, the yellow time being only one of them. Crossing times are not only vital for the calculation of the capacity (green time extension), they also mark the beginning of the clearance interval. Here a major difference can be seen between the signal change philosophy prevailing in the U.S. and the one in Germany. The intergreen equation fostered by the ITE equates crossing time and yellow time. In Germany, yellow times are based on the same principles as in the U.S. (kinematic model), but they are laid down as uniform values in the German Guidelines for Traffic Signals (RiLSA), depending only on the local speed limit in three distinct steps (50, 60, or 70 km/h). Crossing times, thus, can deviate from the yellow interval duration. Namely for turning traffic and bicycles shorter crossing times are assumed.

By distinguishing between crossing time and yellow time, the distinct purpose of the two intervals is underlined. Yellow times are supposed to influence the driver behaviour. In the first place they should focus on safety aspects, only in the second place supporting a legal driving behaviour (both of which is, of course, connected to each other). Their main purpose is to make the driver behaviour predictable – for the other drivers (avoidance of rear end collisions) and for the engineer to determine sensible intergreen times. The crossing time, oppositely, is not a set value, but an observable one, describing the driver behaviour and as such is a direct part of the intergreen time.

Bicycles at signalised intersections

Bicycle traffic is an important part of the transport system, because it offers many advantages in different areas (environmental friendly, requires few space, affordable for all travellers etc.). Otherwise cyclists together with pedestrians are the most vulnerable travellers. They are also very sensitive to detours and not very sensitive to traffic rules. Together these characteristics lead to high requirements for the design of bicycle traffic facilities.

In the context of signalised intersections and intergreen times bicycles can play a major role. While at large intersections, bicycles can become the determining vehicle type due to their low clearance speed, many cyclists do not behave as expected. Observations showed that the variability of cyclists' behaviour is high. Cyclists following the signals for cars and respecting road markings appear to be faster at the same time. Slow cyclists, on the other hand, frequently use the walkway and follow the pedestrian signals.

Because the consideration of cyclists can lead to long intergreen times following the German Guidelines for Traffic Signals (RiLSA), it has to be questioned whether the assumptions made in the guidelines are correct. Furthermore, it has to be considered whether specific facilities for bicycles at signalised intersections can help to improve the overall intersection capacity. Part of these considerations should be separate signals for bicycles. Only in this way the intergreen times can be adjusted specifically for bicycles as has been suggested by, for instance, Taylor (1993) and Wachtel et al. (1995).

Intersection size

If entering times are not considered or entering speeds are assumed to be greater than clearance speeds, compact intersections lead to shorter intergreen times and consequently to higher capacity. However, in reality moving starts are the exception. Vehicles accelerating from a full stop result in equivalent entering speeds lower than the average clearance speeds (cf. Section 5.3.4). If this fact is taken into account, larger intersections are advantageous, since with a smaller ratio of clearance times to entering times the resulting intergreen times are shorter. The consequences and reasons for the shorter intergreen times are expanded upon in Section 6.2.4.

6.5 Recommendations for further empirical studies

6.5.1 Recommendations for the survey methodology

The survey methodology applied during this research was developed based on the requirements of the capacity model. To obtain accurate and precise data, a number of parameters have to be measured simultaneously at all approaches of an intersection. The only feasible option for such kind of analysis are video observations. Other options would require the extensive installation of measurement devices.

To retrieve high quality data from videos involves a number of technical issues. Particularly the time accuracy and ensueing problems are difficult to tackle to date. The surveys revealed that not only the general survey layout has to be devised thoroughly, but some thought has to be directed at the technical details like video format, synchronisation, and angle of view. For the surveys conducted as part of this research it was decided to emphasise on high accuracy and precision by gathering high quality data separately at different intersections subsequently. To relate this data to one example intersection and one period of time, as has been done, implies several assumptions which could introduce some error to the results for the example intersection. The overall quantitative statements on this particular intersection, hence, were not of primary concern when choosing this procedure. Aim was to obtain data for single parameters with high reliability.

To discern the influencing factors on the model parameters requires more extensive surveys than have been feasible during the present research. The results, therefore, give only a general impression on the quantative range of capacity impacts. The main focus was put on the verification of the methodology and qualitative assessments. This could be adequately achieved. While several details, particularly of the video evaluation, still have to be improved for future surveys, the general layout prooved to be suitable for the problems at hand.

6.5.2 Recommendations for the focus of further research

6.5.2.1 Capacity related research

Future research should focus, on the one hand, on the verification of the rough quantitative results obtained from the surveys. On the other hand, more extensive surveys should be conducted to calibrate the model as has been described in Chapter 4.

Surveys should focus primarily on parameters showing high variation (among intersections and at single intersections) and being of salient importance for the capacity model. Namely crossing times and clearance speeds are among them. They should be tested for influencing factors like traffic demand and green split. Moreover, entering times and starting response times should be analysed in more detail, as the stop line distance could turn out to be a factor easily to be influenced or predicted, while the entering behaviour showed some aberration not explained so far.

Furthermore, the speed of turning traffic so far has been disregarded. The results should be generalised to encompass all different kinds of movement in an intersection. This in depth research could include the analysis of the role of heavy vehicles as a third distinct vehicle type in addition to bicycles and passenger cars following the German Guidelines for Traffic Signals (RiLSA). The presented research focussed on *base capacities* and excluded the impact of permitted traffic and pedestrians from the model. This gap should be closed by future research.

The application of the model to additional intersections will help to derive some more general conclusions on the importance of particular stage settings and stage sequences. The generation of conflict trees for different intersections and signal programs could reveal patterns relevant for the capacity assessment.

6.5.2.2 Safety related research

The recommendations described above have been separated into improvement potential not ensued by safety consequences, and measures which will most likely have impacts on the safety. The latter measures, hence, should be further assessed also from a safety perspective.

The most prominent research need, however, has to be seen in the improvement and unification of the intergreen calculation procedures. As has been pointed out by traffic professionals before, the prevailing methods are not based on a sound safety model considering the random character of traffic flow. Consequently, the achieved safety by intergreen times is not only gratuitously varying, but capacity is squandered without safety improvements. A sound safety model would enable decision makers to balance safety and capacity transparently and purposefully, and still would let them adjust the safety requirements according to their own experience and requirements.

6.6 Summary

The presented research offers a comprehensive insight into the processes involved during signal change intervals. This insight was used to develop a capacity model that not only offers the means to derive effective capacities at signalised intersections under consideration of all aspects of intersection layout, the signal program, and the random character of traffic flow, but also gives an estimate of the improvement potential of intergreen times from a capacity point of view.

Intergreen times can be improved without fearing a safety decline. Not all capacity reductions caused by intergreen times lead to safety improvements, because intergreen times to date are not based on a sound safety model. Namely the consideration of entering times and the detailed assessment of conflicts and their likeliness leads to significant capacity improvements.

Intersection layout and signal program design should always take the impacts on intergreen times into account. It was underlined that a low variation of the driver behaviour and a reliable forecast of it requires smaller safety margins resulting in higher capacity. Driver behaviour and its predictability can be influenced by a comprehensive analysis of influencing factors and a sensible intersection layout and signal program design. This improvement potential has been detailed with reference to prevailing manuals and standards. The advantages of a transition time between red and green, the distinction between yellow time and crossing time, and the inconsistencies in the definition of capacity in the German Highway Capacity Manual (HBS) are highlighted.

The application of the capacity model based on exemplative empirical data showed that intergreen times in theory can nearly be avoided completely. The improvement potential for a medium size intersection amounts to nearly 1500 veh/h with an effective capacity of 3800 veh/h.

Further research should focus on more substantial surveys for the calibration of the model. Particularly crossing times, clearance times, and the entering behaviour should be analysed with reference to possible influencing factors like saturation degree, green time split, movement, and coordination. Furthermore, the role of vehicle-to-infrastructure communication technologies bears many opportunities for the optimisation of traffic signal control. These opportunities (and the risks) should be scrutinised.

While the results of the presented research are based on several assumptions, they, nevertheless, underline the high potential involved in improved intergreen times.

List of Abbreviations

Used	Unit	RiLSA	HCM	English	German
General					
l	m			length/distance	Länge, Strecke
n				number	Anzahl
q	$^{veh}/_{h}$	q	V	vehicle volumes	Verkehrsstärke
t	s			time/interval	Zeit(dauer)
C	$^{veh}/_{h}$	C	c	capacity	Kapazität
Basic parameters					
b	$(-)$	b	$\frac{v}{s}$	flow ratio	Verkehrsflussverhältnis
C_{calc}	$^{veh}/_{h}$			calculated intersection capacity	berechnete Kapazität des Knotenpunkts
C_{eff}	$^{veh}/_{h}$			effective intersection capacity	effektive Kapazität des Knotenpunkts
$C_{\mathrm{l,eff}}$	$^{veh}/_{h}$			effective lane capacity	effektive Kapazität eines Fahrstreifens
$C_{\max}$	$^{veh}/_{h}$			maximum intersection capacity	maximale Kapazität des Knotenpunkts
h_{s}	s			saturation headway	Zeitbedarfswert bei Sättigung
l_{cl}	$^{veh}/_{h}$	s_0		clearance distance	Grundräumweg
l_{e}	$^{veh}/_{h}$	s_{e}		entering distance	Einfahrweg
l_{SL}	$^{veh}/_{h}$			stop line distance	Abstand der Haltlinie
l_{veh}	$^{veh}/_{h}$	l_{Fz}	L	vehicle length	Fahrzeuglänge
$l_{\mathrm{w,cl}}$	$^{veh}/_{h}$			width of lane of the clearing vehicle	Fahrstreifenbreite des räumenden Fahrzeugs
$l_{\mathrm{w,e}}$	$^{veh}/_{h}$			width of lane of the entering vehicle	Fahrstreifenbreite des einfahrenden Fahrzeugs
p	$(-)$			probability of movement sequence	Wahrscheinlichkeit einer Bewegungsfolge
q_{S}	$^{veh}/_{h}$	q_{S}	s	saturation flow rate	Sättigungsverkehrsstärke
$\mathrm{vt}_{\mathrm{cl}}$	$(-)$			vehicle type of clearing vehicle	Fahrzeugtyp des räumenden Fahrzeugs
t_{C}	s	t_{U}	C	cycle time	Umlaufzeit
t_{cr}	s	$t_{\ddot{\mathrm{U}}}$		crossing time (of clearing vehicle)	Überfahrzeit (des räumenden Fahrzeugs)
$t_{\mathrm{cr,e}}$	s			crossing time of entering vehicle	Überfahrzeit des einfahrenden Fahrzeugs
t_{cl}	s	t_{r}		clearance time	Räumzeit
t_{e}	s	t_{e}		entering time	Einfahrzeit
t_{g}	s			effective green time	effektive Freigabezeit
t_{G}	s	t_{F}		signalled green time	signalisierte Freigabezeit
t_{ig}	s	t_{Z}		intergreen time	Zwischenzeit
$t_{\mathrm{ig,m}}$	s			intergreen time for conflict	Zwischenzeit eines Konflikts

Used	Unit	RiLSA	HCM	English	German
$t_{ig,g}$	s			intergreen time for signal group sequence	Zwischenzeit einer Signalgruppenfolge
$t_{ig,s}$	s			intergreen time for stage sequence	Zwischenzeit einer Phasenfolge
$t_{ig,l}$	s			intergreen time for lane combination	Zwischenzeit einer Fahrstreifenkombination
$t_{ig,min}$	s			minimum intergreen time	minimale Zwischenzeit
t_{obs}	s			observation time	Beobachtungszeitraum
t_{PE}	s			post-encroachment time	Nettozeitlücke am Konfliktpunkt
t_{saf}	s			safety margin	Sicherzeitszuschlag
t_{SR}	s			starting response time	Anfahrreaktionszeit
t_{SUL}	s			start-up lost time	(Anfahrzeitverlust)
v_{cl}	$^{m}/_{s}$	v_r		clearance speed	Räumgeschwindigkeit
$\overline{v}_{cl,eff}$	$^{m}/_{s}$			equivalent clearance speed	equivalente Räumgeschwindigkeit
v_e	$^{m}/_{s}$	v_e		entering speed	Einfahrgeschwindigkeit
$\overline{v}_{e,eff}$	$^{m}/_{s}$			equivalent entering speed	equivalente Einfahrgeschwindigkeit
Difference parameters between effective and assumed values					
ΔC_{max}	$^{veh}/_{h}$			maximum capacity improvement potential	maximales Verbesserungspotenzial der Kapazität
Δl_{cl}	m			total clearance distance difference	Gesamträumwegdifferenz
Δl_{e}	m			total entering distance difference	Gesamteinfahrwegdifferenz
$\Delta l_{cl,sys}$	m			systematic clearance distance difference	systematische Räumwegdifferenz
$\Delta l_{e,sys}$	m			systematic entering distance difference	systematische Einfahrwegdifferenz
Δl_{veh}	m			vehicle length difference	Fahrzeuglängendifferenz
Δt_{c}	s			conflict difference time	Konfliktdifferenzzeit
Δt_{ig}	s			intergreen time difference	Zwischenzeitdifferenz
Δt_{gG}	s			green time difference	Freigabezeitdifferenz
Δt_{G}	s			green time extension	Freigabezeitverlängerung
Δt_{PE}	s			interaction time	Interaktionszeit
Δt_{saf}	s			safety margin difference	Sicherzeitszuschlagdifferenz
Δt_{SUL}	s			start-up lost time difference	(Anfahrzeitverlustdifferenz)
Δt_{cl}	s			clearance time difference	Räumzeitdifferenz
Δt_{cr}	s			crossing time difference of clearing vehicles	Überfahrzeitdifferenz räumender Fahrzeuge
$\Delta t_{cr,e}$	s			crossing time difference of entering vehicles	Überfahrzeitdifferenz einfahrender Fahrzeuge
Δt_{e}	s			entering time difference	Einfahrzeitdifferenz
Δv_{cl}	$^{m}/_{s}$			clearance speed difference	Räumgeschwindigkeitsdifferenz
Δv_{e}	$^{m}/_{s}$			entering speed difference	Einfahrgeschwindigkeitsdifferenz

Used	Unit	RiLSA	HCM	English	German
$\Delta h(k)$	s			cumulated headway difference	kumulierte Zeitlückendifferenz

List of Figures

List of Tables

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Appendices

A Details on calculation procedures

A.1 Definitions of capacity in Germany and the United States

A.1.1 German Highway Capacity Manual (HBS)

The capacity is defined in FGSV (2001) as the "maximum number of traffic elements per time unit, which can be served by a signalised intersection given the geometric and traffic related circumstances."

The capacity for an intersection is the sum of the capacities for all lanes (Equation 6-51):

$$C_K = \sum_{i=1}^{n} C_i$$

where	C_K	intersection capacity	(veh/h)
	n	number of lanes	(–)
	C_i	lane capacity	(veh/h)

The capacity of lanes is determined using the saturation flow rate and the available green time, corrected by the impacts of special constellations (Table 30).

Adjustment	Factor	Determination	Condition
Heavy vehicle share	f_{SV}	$1 - 0.0083e^{0.21 \cdot HV}$	$2\,\% \leq HV \leq 15\,\%$
Lane width	f_b	$1 + \frac{2(W-3)}{5}$	approximation, $3\,\text{m} \geq W \geq 2.6\,\text{m}$
Turning radius	f_R	1.0	$R > 15\,\text{m}$
		0.9	$10\,\text{m} > R > 10\,\text{m}$
		0.85	$R \leq 10\,\text{m}$
Grade	f_s	$0.85 \leq f_s \leq 1.15$	$-5\,\% \leq s \leq +5\,\%$

Table 30: Adjustment factors for saturation flow rate according to FGSV (2001) (abridged)

The saturation flow rate for the traffic streams at the example intersection (cf. Section 5.4.2) are given in Table 31. The capacities of lane groups (Table 17) is calculated from the harmonic mean of the saturation flow rates of the streams (Eq. 41).

$$q_{\text{SM}} = \frac{1}{\sum \frac{a_i}{q_{S,i}}} \qquad a_i = \frac{q_i}{\sum q_i} \tag{41}$$

with	q_{SM}	saturation flow of mixed lane	($^{veh}/_{h}$)
	a_i	share of stream i on lane	(-)
	q_i	vehicle volume of stream i on lane	($^{veh}/_{h}$)
	$q_{S,i}$	saturation flow rate of stream i	($^{veh}/_{h}$)
	$\sum q_i$	vehicle volume on lane	($^{veh}/_{h}$)

Lane	Direction of flow	$q_{S,St}$	HV	f_{HV}	Second adjustment	f_2	q_S
		($^{pc}/_{h}$)	(%)	(-)		(-)	($^{veh}/_{h}$)
NR	R	2000	1	1.00	lane width	0.90	1800
NR	T	2000	1	1.00	lane width	0.90	1800
NL	T	2000	1	1.00		1.00	2000
NL	L	2000	1	1.00		1.00	2000
E	R	2000	1	1.00		1.00	2000
E	T	2000	1	1.00		1.00	2000
SR	R	2000	4	0.98	radius	0.90	1765
SR/SL	T	2000	4	0.98		1.00	1962
SL	L	2000	4	0.98		1.00	1962
WR	R	2000	1	1.00	lane width	0.90	1800
WR	T	2000	1	1.00	lane width	0.90	1800
WL	L	2000	0	1.00	lane width	0.90	1800

Table 31: Saturation flow rate of traffic streams at A 046 according to HBS

A.1.2 U.S. Highway Capacity Manual (HCM)

To simplify the calculation methods, TRB (2000) provides the combination of lanes to *lane groups*. The capacity for each lane group is defined as “the maximum hourly rate at which vehicles can reasonably be expected to pass through the intersection under prevailing traffic, roadway, and signalization conditions. . . . [C]apacity is stated in (veh/h)."

The capacity of a lane group is defined in Equation 16-6:

$$c_i = s_i \frac{g_i}{C}$$

where	c_i	capacity of lane group i	(veh/h)
	s_i	saturation flow rate for lane group i	(veh/h)
	g_i	effective green time for lane group i	(s)
	C	cycle length	(s)

The capacity of lane groups is determined using the saturation flow rate and the effective green time, corrected by the impacts of special constellations. The ones used in Section 5.4.2 are listed in Table 32. Factors for permitted left turning and right turning vehicles are calculated according to Appendix C in TRB (2000), based on volumes and green times of turning traffic and opposing traffic. The calculated adjustment factors and capacities according to HCM are given in Table 18 on page 98.

Adjustment	Factor	Determination	Remarks
Heavy vehicle share	f_{HV}	$\frac{100}{100+HV}$	
Left turns	f_{LT}		separate calculation for nonprotected stages
Right turns	f_{RT}	0.85	exclusive lane
			separate calculation for nonprotected stages

Table 32: Adjustment factors for saturation flow rate according to TRB (2000) (abridged)

A.2 Test for distribution of streams among entering vehicles

For the calculation of the probability of certain movement sequences (Section 3.3.5, p. 50), it was assumed that the vehicles in the front of the queue waiting for the green signal are distributed like all vehicles on a lane. Pearson's χ^2-Test was used to test this assumption. Table 33 gives the share of through traffic for all vehicles on the respective lane (3), the total number of vehicles in the first position of the queue (4)[19], the observed (5) and expected (6, Eq. 39) number of through vehicles in front of the queue, and the resulting χ^2 value according to Eq. 40. The χ^2 value is compared with the χ^2 distribution with one degree of freedom (two streams, total number of vehicles constrained).

Lane	Streams	Share Through (–)	Vehicles in front of queue Total (veh)	Observed through (veh)	Expected through (veh)	χ^2 (-)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
NR	TR	0.83	70	63	58	2.7
NL	TL	0.87	69	60	60	0.0
EC	TR	0.87	42	37	37	0.0
SR	TR	0.80	45	39	36	1.3
SL	TL	0.78	40	29	31	0.6
WR	TR	0.74	70	62	52	7.4

Table 33: Observed and expected through vehicles in first position at example intersection A 046

A.3 Calculation of entering time and clearance time differences

The difference of effective and assumed entering/clearance times (Section 3.3.5.2) depends on the effective distances and speeds obtained for the calculation of these times (Eq. 42 and Eq. 43).

$$\Delta t_{\mathrm{e}} = t_{\mathrm{e,eff}} - t_{\mathrm{e}} = \frac{l_{\mathrm{e}} + \Delta l_{\mathrm{e}}}{v_{\mathrm{e}} + \Delta v_{\mathrm{e}}} - \frac{l_{\mathrm{e}}}{v_{\mathrm{e}}} = \frac{v_{\mathrm{e}}\Delta l_{\mathrm{e}} - \Delta v_{\mathrm{e}} l_{\mathrm{e}}}{v_{\mathrm{e}}^2 + v_{\mathrm{e}}\Delta v_{\mathrm{e}}} \tag{42}$$

$$\Delta t_{\mathrm{cl}} = t_{\mathrm{cl,eff}} - t_{\mathrm{cl}} = \frac{l_{\mathrm{cl}} + l_{\mathrm{veh}} + \Delta l_{\mathrm{cl}} + \Delta l_{\mathrm{veh}}}{v_{\mathrm{cl}} + \Delta v_{\mathrm{cl}}} = \frac{v_{\mathrm{cl}}(\Delta l_{\mathrm{cl}} + \Delta l_{\mathrm{veh}}) - \Delta v_{\mathrm{cl}}(l_{\mathrm{cl}} + l_{\mathrm{veh}})}{v_{\mathrm{cl}}^2 + v_{\mathrm{cl}}\Delta v_{\mathrm{cl}}} \tag{43}$$

[19] This number equals the number of evaluated cycles, and represents, hence, the sample size.

Derivation of time as a function of distance

The entering speed is approximated as a function of distance as in Eq. 37.

To calculate the entering times from this function, the movement is split into small distance steps of length Δs, on which a constant acceleration is assumed. The equations for an accelerated movement can be applied to each of these steps (Eq. 44 and Eq. 45).

$$s_i = \frac{1}{2}a_i(t_i - t_{i-1})^2 + v_{i-1}(t_i - t_{i-1}) + s_{i-1} \tag{44}$$

$$v_i = a_i(t_i - t_{i-1}) + v_{i-1} \tag{45}$$

These equations are solved for $t_i - t_{i-1} = \Delta t_i$.

$$\Delta t_i = \frac{-v_{i-1} \pm \sqrt{v_{i-1}^2 - 2a_i \Delta s}}{a_i}$$

$$\Delta t_i = \frac{v_i - v_{i-1}}{a_i}$$

Isolating a_i gives

$$a_i = \frac{v_{i-1}^2 - v_i^2}{2\Delta s} \tag{46}$$

which is inserted in Eq. 45.

$$\Delta t_i = \frac{2\Delta s}{v_i + v_{i-1}} \tag{47}$$

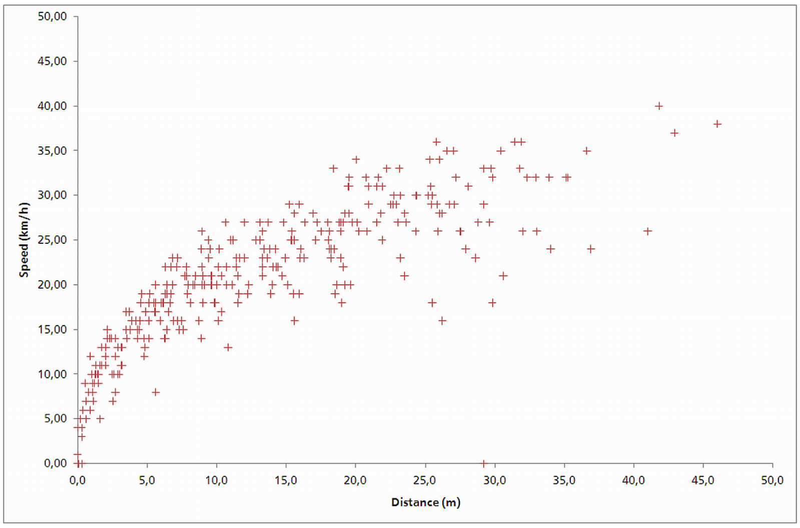
With $t_0 = 0$, $s_0 = 0$, $v_0 = 0$, $s_i = i\Delta s$ and the logarithmic function for the speed as of Eq. 37 the individual t_i can be calculated by summing up incrementally.

$$t_i = \sum \Delta t_i = \sum \left(\frac{2\Delta s}{c_1 \ln(c_2 s_i + 1) + c_1 \ln(c_2 s_{i-1} + 1)} \right) = \frac{2\Delta s}{c_1} \sum \left(\frac{1}{\ln(c_2 \Delta s(2i-1) + 2)} \right) \tag{48}$$

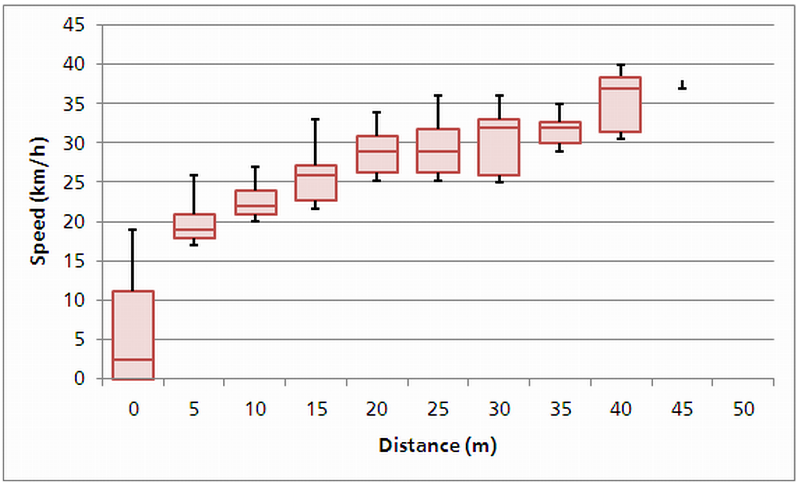
Because the first few metres of acceleration can only be determined with high uncertainty, the distance steps Δs should not be chosen too small. The smaller they are, the greater becomes the influence of the first steps on the overall result. $\Delta s = 1\,\mathrm{s}$ was chosen as a compromise between precision and error.

Fitting the speed-distance-function

The speed-distance-measurements are grouped into bins of equal width. The logarithmic function following the form in Eq. 37 was then fitted to these classes by means of the least-squares method. Figure 35 shows the different steps.



(a) Scatter diagram



(b) Box and whisker plot

Figure 35: Speed-distance measurements

A.5 Mathematical background for the calculation of uncertainties in the model output

The uncertainty of a function consisting of several random variables $f(x_1, x_2, \ldots, x_n)$ can generally be obtained by calculating the contributions of each variable to the overall error δ_{fx_i} and summing up these contributions in quadrature (Eq. 49).

$$\delta_f = \sqrt{\sum \delta_{fx_i}^2} \tag{49}$$

The contributions are the partial derivates of the function times the error of the respective variable (Eq. 50).

$$\delta_{fx_i} = \left| \frac{\partial f}{\partial x_i} \right| \delta_{x_i} \tag{50}$$

For additions or subtractions, the resulting error simplifies to Eq. 51.

$$\delta_f = \sqrt{\sum \delta_{x_i}^2} \tag{51}$$

Annotations to error propagation calculation

Cumulated headway difference The cumulated headway difference $\Delta h(k)$ according to Eq. 9 is influenced by the uncertainty of the saturation headway and the headways of the first entering vehicles. The function is a simple addition.

Start-up lost times The start-up lost time t_{SUL} according to Eq. 10 is the sum of individual variables.

Green time difference The green time difference Δt_{gG} according to Eq. 12 is the sum of individual variables.

Entering time The entering time calculation is based on a number of assumptions. To give an indiciation of the uncertainty involved, speed functions are fitted to the upper and lower bounds of the speed classes measured. The bounds are defined by the standard error. The entering times can then be calculated with the bound functions to give the variation (see below).

Clearance time The clearance time depends on the clearance distance and its variation, and the clearance speed and its variation. Since clearance distances and their variation have only analytically been derived, the standard error is assumed to be 0.5 m. The clearance distance only weekly influences the resulting error of the clearance time. The values given below are upper limits calculated for clearance distances of 40 m with a distance difference of ten percent (4 m). To take account of the measurement error, the error of the clearance speeds was calculated from both the standard error of the survey results $\delta_{\Delta v_{\mathrm{cl}},m}$ and the stated accuracy of the measurement device $\delta_{\Delta v_{\mathrm{cl}},m}$ stated by the manufacturer ($\mp 2\,\mathrm{km/h}$).

Capacities The capacity functions require the application of Eq. 50 to all variables involved.

Equations for the error of the effective capacity

$$\delta_{\Delta h(k)} = \sqrt{(k\delta_{h_s})^2 + \sum {h_i}^2} \tag{52}$$

$$\delta_{\Delta t_{\mathrm{SUL}}} = \sqrt{\delta^2_{\Delta t_{\mathrm{SR}}} + \delta^2_{h_s} + \delta^2_{\Delta h(k)}} \tag{53}$$

$$\delta_{\Delta t_{\mathrm{gG}}} = \sqrt{\delta^2_{\Delta t_{\mathrm{SUL}}} + \delta^2_{\Delta t_{\mathrm{cr}}} + \delta^2_{\Delta t_{\mathrm{PE}}}} \tag{54}$$

$$\delta_{C_{\mathrm{eff},i}} = \sqrt{\delta^2_{Ch_s} + \delta^2_{Ct_G} + \delta^2_{C\Delta t_{\mathrm{gG}}}} \tag{55}$$

$$\delta_{C_{\mathrm{eff}}} = \sqrt{\sum \delta^2_{C_{\mathrm{eff},i}}} \tag{56}$$

$$\tag{57}$$

Contributions to C_{eff}:

$$\delta_{Ch_s} = \frac{t_{\mathrm{obs}}(t_G + \Delta t_{\mathrm{gG}})}{h_s^2 t_{\mathrm{C}}} \delta_{h_s} \tag{58}$$

$$\delta_{Ct_G} = \frac{t_{\mathrm{obs}}}{h_s t_{\mathrm{C}}} \delta_{t_G} \tag{59}$$

$$\delta_{C\Delta t_{\mathrm{gG}}} = \frac{t_{\mathrm{obs}}}{h_s t_{\mathrm{C}}} \delta_{\Delta t_{\mathrm{gG}}} \tag{60}$$

$$\tag{61}$$

Equations for the error of the maximum capacity

$$\delta_{t_{\mathrm{cl}}} = \sqrt{\delta^2_{t_{\mathrm{cl}}\Delta l_{\mathrm{cl}}} + \delta^2_{t_{\mathrm{cl}}\Delta v_{\mathrm{cl}}}} \tag{62}$$

$$\delta_{\Delta t_{\mathrm{ig,m}}} = \sqrt{\delta^2_{\Delta t_{\mathrm{c}}} + \delta^2_{\Delta t_{\mathrm{SR}}} + \delta^2_{\Delta t_{\mathrm{e}}} + \delta^2_{\Delta t_{\mathrm{cr}}} + \delta^2_{\Delta t_{\mathrm{cl}}} + \delta^2_{t_{\mathrm{saf}}}} \tag{63}$$

$$\delta_{\Delta t_{\mathrm{G}}} = \sqrt{n_m \cdot \delta^2_{\Delta t_{\mathrm{ig,m}}}} = \sqrt{7 \cdot \delta^2_{\Delta t_{\mathrm{ig,m}}}} \tag{64}$$

$$\delta_{\Delta C_{\mathrm{max},i}} = \sqrt{\delta^2_{\Delta C \Delta t_{\mathrm{G},i} i} + \delta^2_{\Delta C h_{\mathrm{s},i}}} \tag{65}$$

$$\delta_{\Delta C_{\mathrm{max}}} = \sqrt{\sum \delta^2_{\Delta C_{\mathrm{max},i}}} \tag{66}$$

Contributions to Δt_{cl} and ΔC_{max}:

$$\delta_{t_{\mathrm{cl}}\Delta l_{\mathrm{cl}}} = \frac{v_{\mathrm{cl}}}{v^2_{\mathrm{cl}} + \Delta v_{\mathrm{cl}} v_{\mathrm{cl}}} \delta_{\Delta l_{\mathrm{cl}}} \tag{67}$$

$$\delta_{t_{\mathrm{cl}}\Delta v_{\mathrm{cl}}} = \frac{\Delta l_{\mathrm{cl}} + l_{\mathrm{cl}}}{(v_{\mathrm{cl}} + \Delta v_{\mathrm{cl}})^2} \delta_{\Delta v_{\mathrm{cl}}} \tag{68}$$

$$\delta_{\Delta v_{\mathrm{cl}}} = \sqrt{\delta^2_{\Delta v_{\mathrm{cl}},m} + \delta^2_{\Delta v_{\mathrm{cl}},d}} \tag{69}$$

$$\delta_{\Delta C \Delta t_{\mathrm{G}}} = \frac{n_l \cdot 3600}{h_s \cdot t_{\mathrm{C}}} \delta_{\Delta t_{\mathrm{G}}} \tag{70}$$

$$\delta_{\Delta C h_s} = \frac{n_l \cdot 3600}{h_s^2 \cdot t_{\mathrm{C}}} \delta_{h_s} \tag{71}$$

B Details on the conducted surveys

B.1 Technical Details

B.1.1 Cameras

Two IP cameras (AXIS and TRENDnet) and one camcorder have been used to record videos of intersections. The specifications are given in Table 34.

Manufacturer Model	AXIS PTZ 215	TRENDnet TV-IP 410	Panasonic SDR-H 280
Chip	1/4” CCD	1/4” CMOS	3x1/6” CCD
Max. Resolution	4-CIF	VGA	1.9 Mill. eff. pixel (16:9)
Used resolution	CIF	VGA	1080p, LP
Focus	AF/Manual	Fix	AF/Manual
Focal length	3.8-46 mm	4.0 mm	3.0-40.0 mm
Used focal length	3.8 mm	4.0 mm	3.0 mm
Aperture	1.6-2.7	1.8	1.8-2.8
Pan	$\mp 170°$	$\mp 168°$	–
Tilt	180°	-15 to $+90°$	–
Video format	MJPEG/MPEG-4	MJPEG	MPEG-2

Table 34: Technical specifications of cameras

B.1.2 Speed measuring device

A laser speed measuring device, mounted on a tripod, was used to obtain speeds of entering and clearing vehicles. The technical specifications are given in Table 35. A photograph showing the operation can be found on 78 (Figure 21).

Manufacturer Model	Robot Visual Systems GmbH TraffiPatrol XR
Measurement range (speed)	0 km/h to 500 km/h
Measurement range (distance)	10 m to 1500 m
Measurement accuracy (speed)	$\mp$2 km/h below 100 km/h
Measurement accuracy (distance)	$\mp$0.2 m
Measurement duration	0.3 s
Laser divergence	2 mrad

Table 35: Technical specifications of speed measurement device

B.1.3 Remarks on the video evaluation

The videos were recorded in MJPEG format (AXIS and TRENDnet) and MPEG-2 (Panasonic) respectively. MJPEG compresses every single frame of the video independantly of other frames. In this way, every frame can be easily accessed. MPEG-2 divides the video in *groups of pictures* (GOP). Every GOP starts with a intra-coded frame (keyframe, I-frame), which is compressed similar to MJPEG frames. The remaining frames of a GOP are predictive-coded frames (either unidirectional, P-frame, or bidirectional, B-frame), which means that only differences between frames are saved. While this compression leads to small file sizes, only I-frames can be directly accessed. P- and B-frames require a reconstruction of all frames since the last I-frame.

The videos have been evaluated using free software for the generation of subtitles (Subtitle Workshop 2.5 by URUsoft, www.urusoft.net). This simple software does not support the reconstruction of single frames. Navigation is possible between keyframes only. Consequently, MPEG-2 videos have to be evaluated in real time (interruptions are possible though). While the AXIS and TRENDnet videos can be evaluated framewise (which allows for higher accuracy and the correction of user mistakes), the Panasonic videos had to be evaluated in real time.

To assess the achievable accuracy, part of a video has been evaluated for headways five times by four different persons. A comparison of the results shows that only 2 % of the vehicles were not or only significantly late or early registered. Most of these errors have been marked by the evaluating person. The standard deviation of the headways remained below 0.05 s (Todt 2009). The overall quality of the video evaluation can, hence, be stated as good.

B.2 Survey locations

Surveys were conducted at seven intersections. Depending on the intersection layout, the intersections are suitable for observations with the extension mast (Figure 19), for stop line observations, and for speed measurements. The intersections with the different measurements are listed in Table 7 on page 80. Plans of the intersections are given in Figure 38 to Figure 44 on the following pages. The position of the extension mast, the stop line camera, and the speed measurement device are marked (cf. Figure 37).

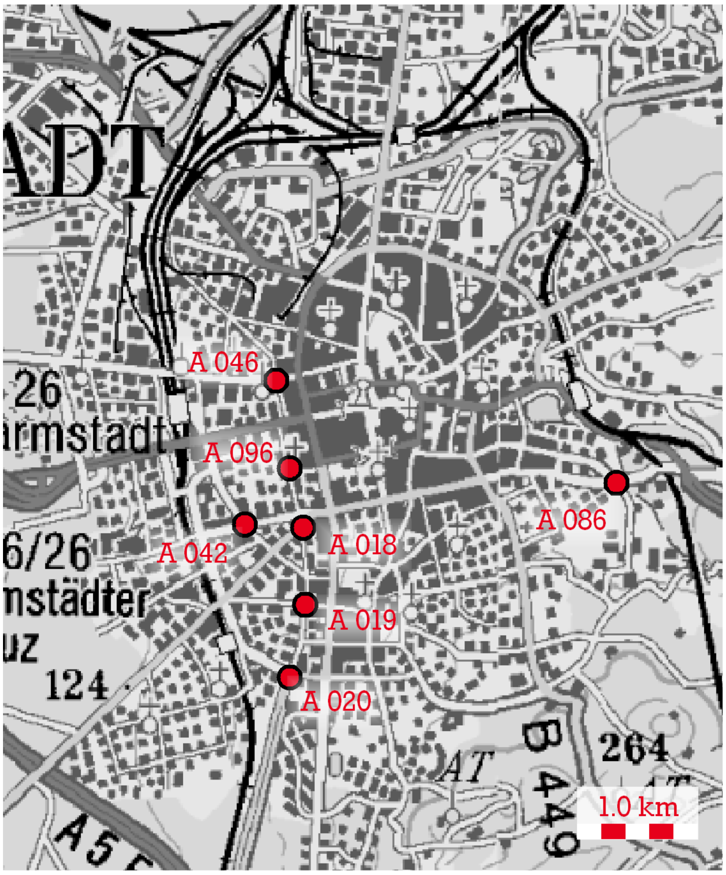


Figure 36: City map of Darmstadt with survey intersections

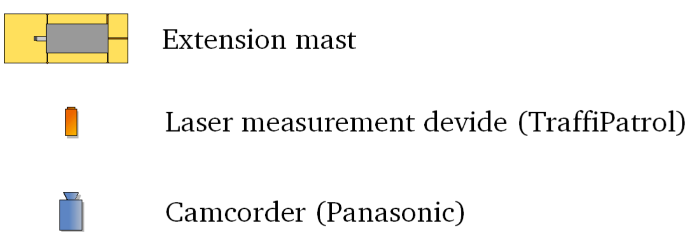


Figure 37: Legend to the intersection plans (Figure 38 to Figure 44)

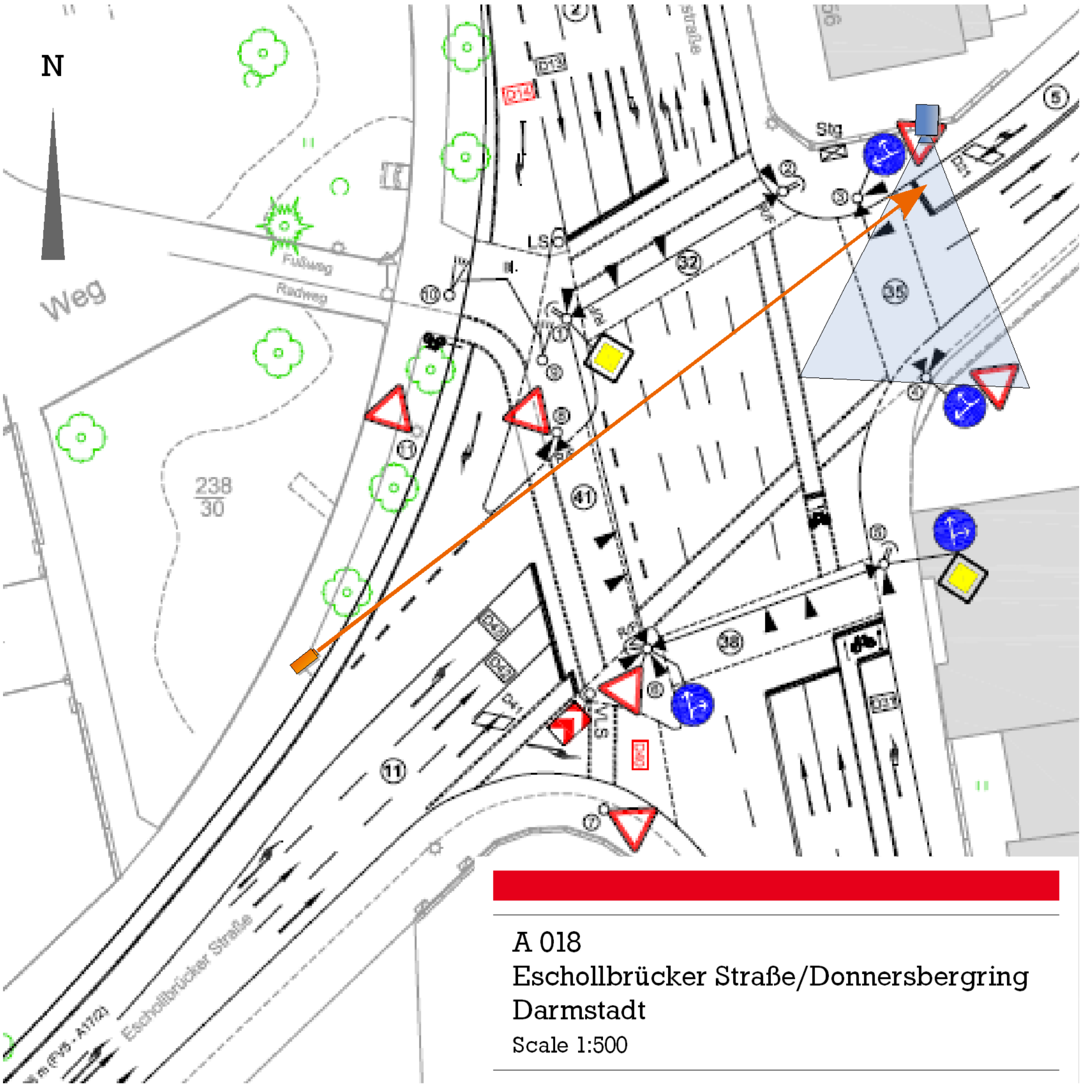


Figure 38: Intersection and survey layout at A 018

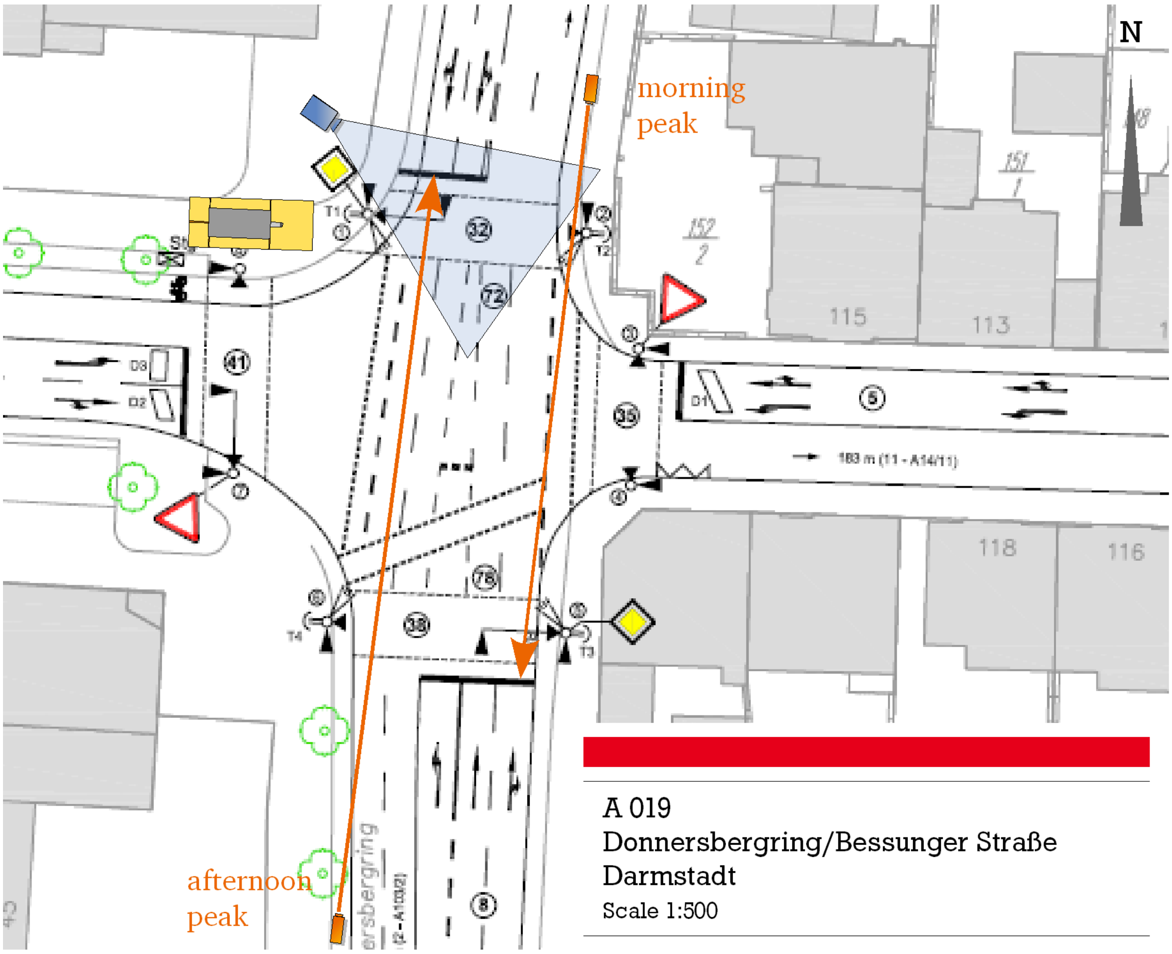


Figure 39: Intersection and survey layout at A 019

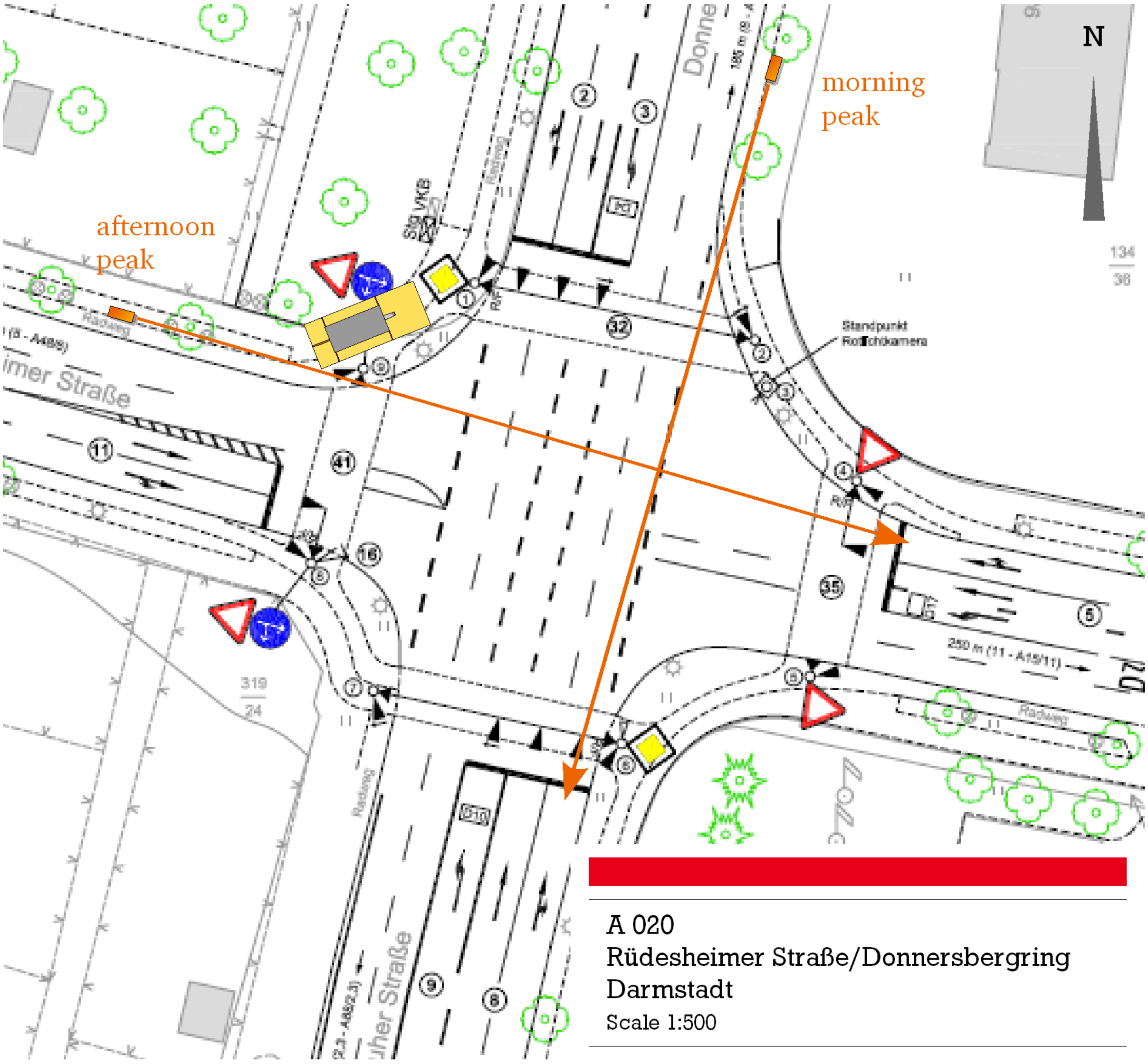


Figure 40: Intersection and survey layout at A 020

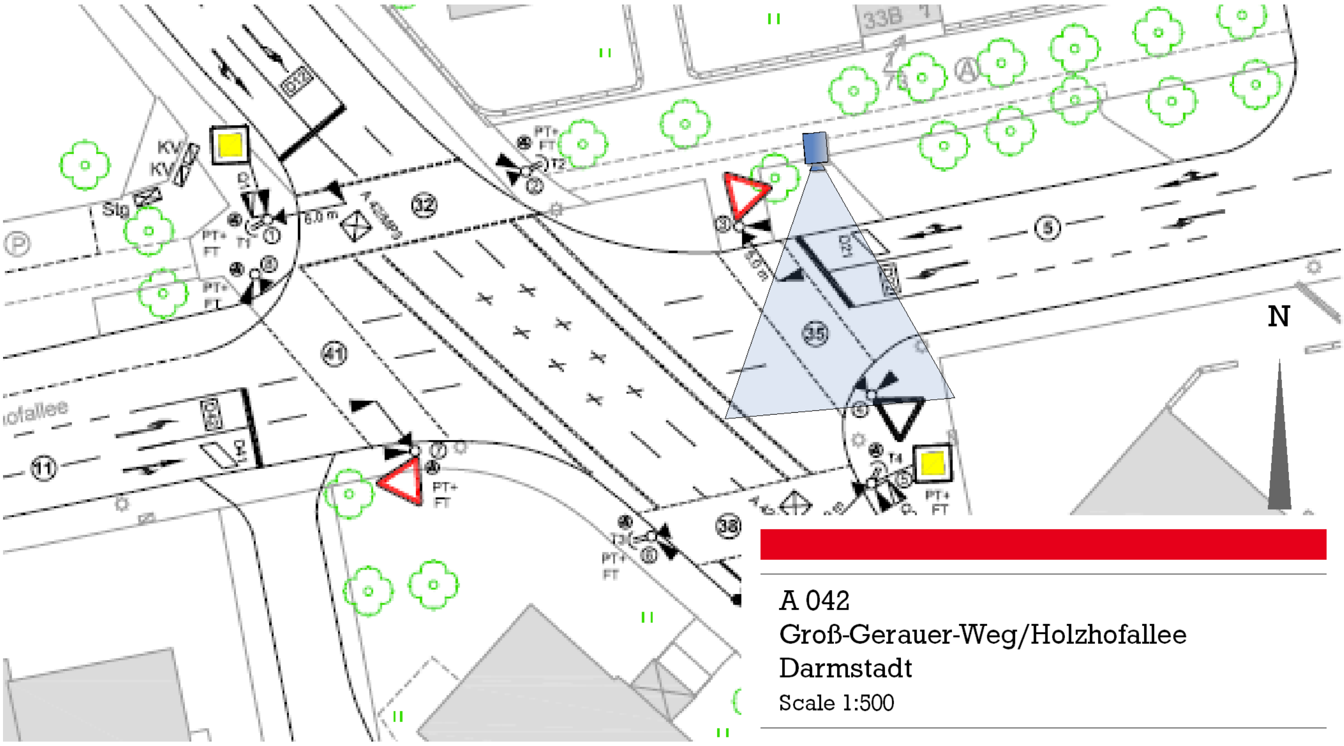


Figure 41: Intersection and survey layout at A 042

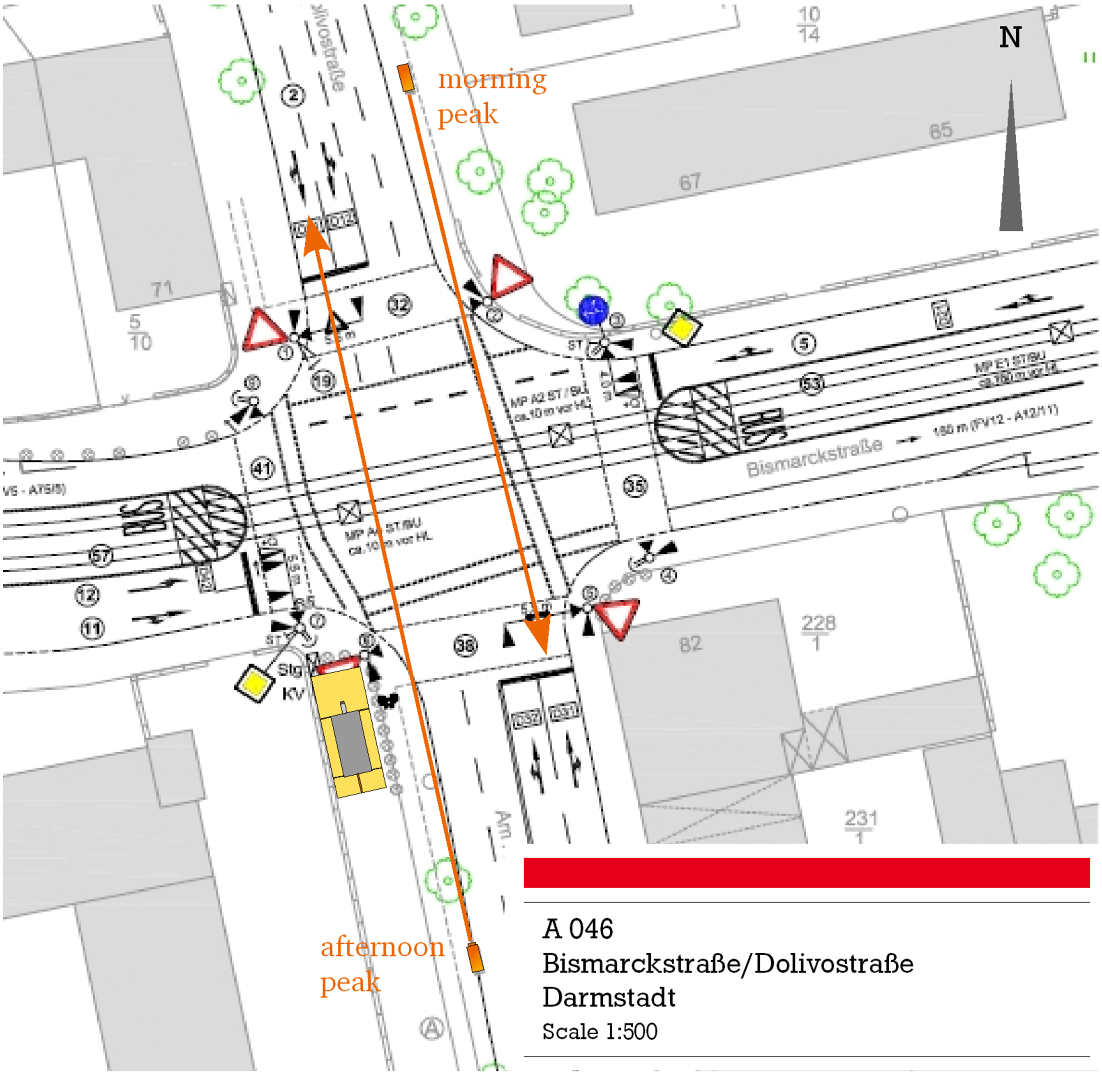


Figure 42: Intersection and survey layout at A 046

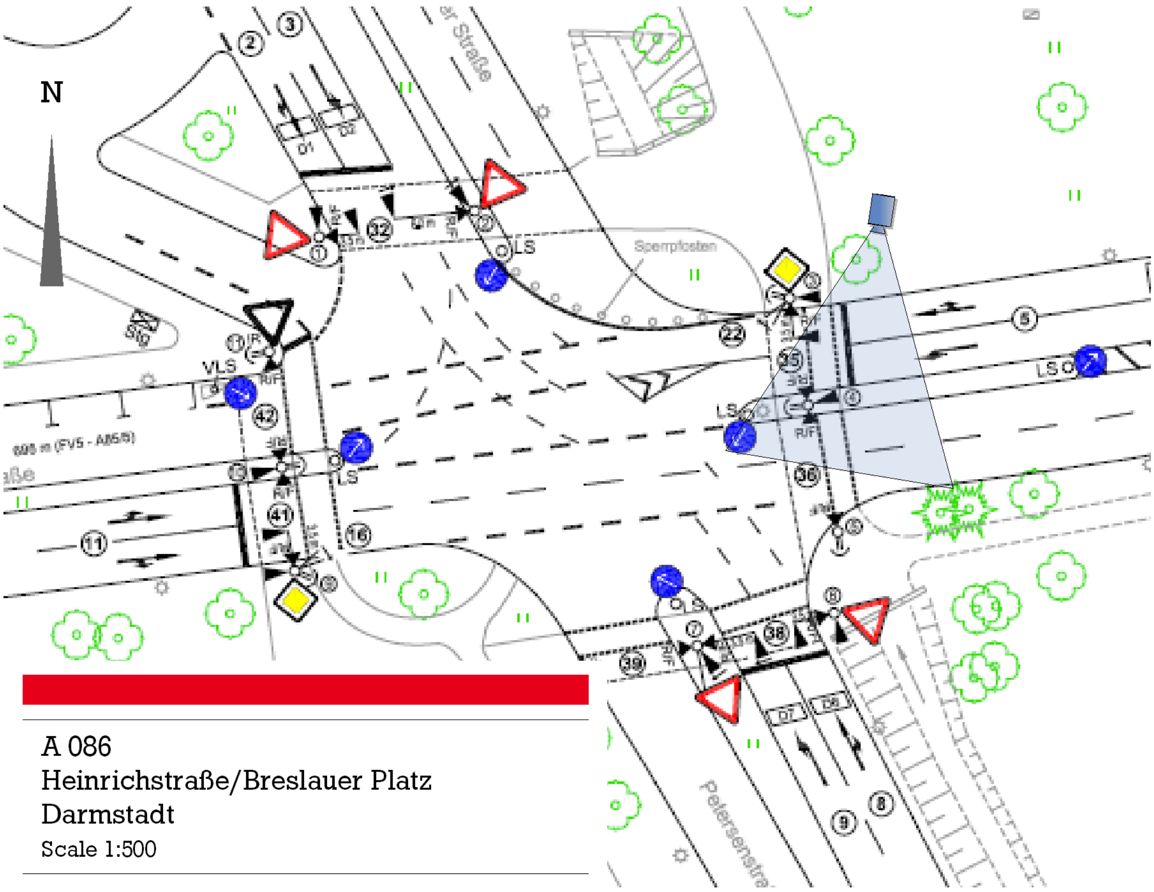


Figure 43: Intersection and survey layout at A 086

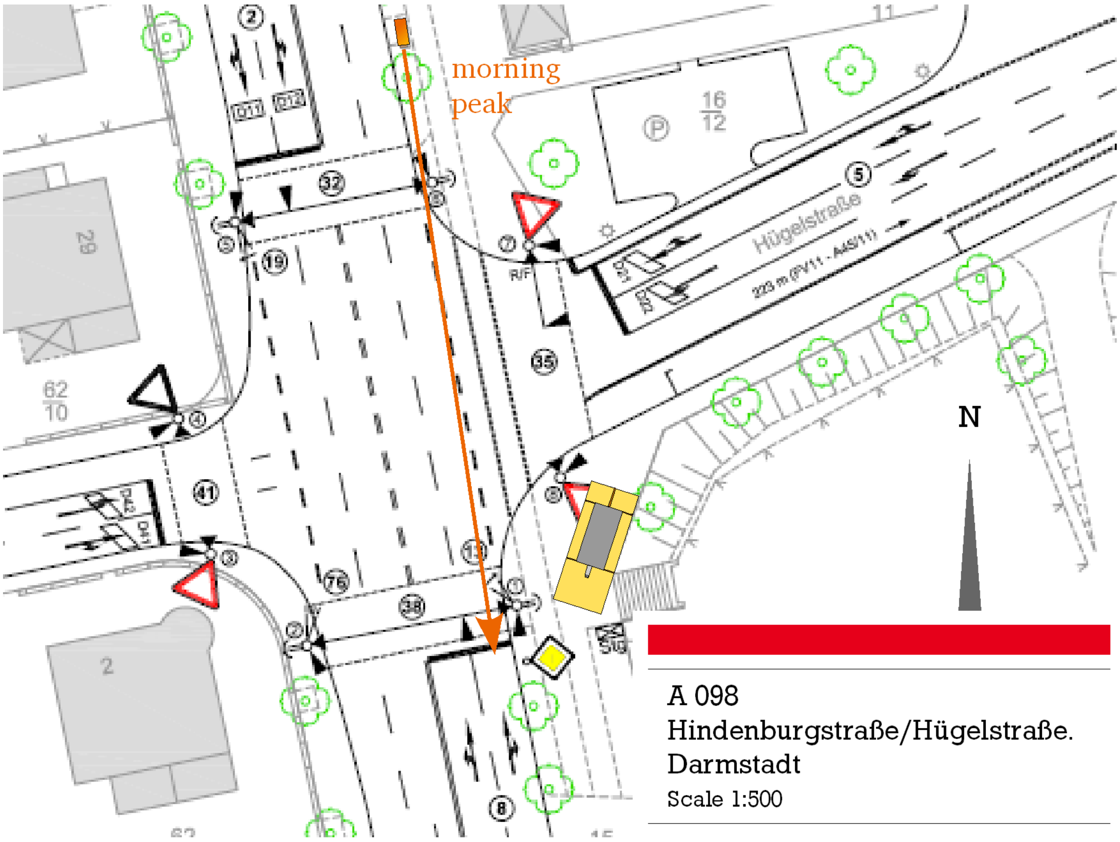


Figure 44: Intersection and survey layout at A 098

B.3 Complete conflict tree

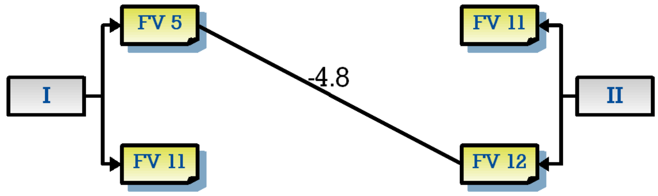
Stage		Signal group		Lane		Stream		b/c	$t_{ig,s,i}$	p	$´t_c$	t_{saf}	$´t_{cr,e} + ´t_e$	$´t_{cr}$	$´t_{cl}$	$´t_{ig,m,i}$	$´t_{ig,l,i}$	$´t_{ig,s,i}$
c	e	c	e	c	e	c	e	c										
(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(s)	(–)	(s)	(s)	(s)	(s)	(s)	(s)	(s)	(s)
1	2	5	11	EC	WR	4	10	b	0	0.00	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	4	10	c	0	0.03	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	4	11	b	0	0.00	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	4	11	c	0	0.10	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	5	10	b	0	0.02	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	5	10	c	0	0.20	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	5	11	b	0	0.05	0	0	0.0	0.0	0.0			
1	2	5	11	EC	WR	5	11	c	0	0.60	0	0	0.0	0.0	0.0		0.0	0.0
1	2	5	12	EC	WL	4	12	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
1	2	5	12	EC	WL	4	12	c	5	0.13	0	-0.2	-3.6	-0.4	0.2	-3.9		
1	2	5	12	EC	WL	5	12	b	5	0.07	0	0	-2.6	0.0	0.5	-2.1		
1	2	5	12	EC	WL	5	12	c	5	0.80	-1	-0.4	-2.6	-1.4	0.2	-5.2	-4.8	-4.8
1	2	11	12	WR	WL	10	12	b	0	0.00	0	0	0.0	0.0	0.0			
1	2	11	12	WR	WL	10	12	c	0	0.25	0	0	0.0	0.0	0.0			
1	2	11	12	WR	WL	11	12	b	0	0.01	0	0	0.0	0.0	0.0			
1	2	11	12	WR	WL	11	12	c	0	0.74	0	0	0.0	0.0	0.0		0.0	0.0
2	3	11	2	WR	NR	10	1	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NR	10	1	c	5	0.05	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NR	10	2	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NR	10	2	c	5	0.20	0	-1	-3.3	-0.4	0.0	-4.7		
2	3	11	2	WR	NR	11	1	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NR	11	1	c	5	0.14	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NR	11	2	b	5	0.01	-2	-1	-3.0	0.0	0.0	-6.0		
2	3	11	2	WR	NR	11	2	c	5	0.60	-2	-0.4	-3.0	-1.4	0.0	-6.8	-6.0	
2	3	11	2	WR	NL	10	2	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NL	10	2	c	5	0.22	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NL	10	3	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NL	10	3	c	5	0.03	-5	0	0.0	0.0	0.0	-5.0		
2	3	11	2	WR	NL	11	2	b	5	0.01	-2	-0.2	-3.0	0.0	0.0	-5.2		
2	3	11	2	WR	NL	11	2	c	5	0.64	-2	0	-3.0	-1.4	0.0	-6.4		

Stage		Signal group		Lane		Stream		b/c	$t_{ig,s,i}$	p	$´t_c$	t_{saf}	$´t_{cr,e} + ´t_e$	$´t_{cr}$	$´t_{cl}$	$´t_{ig,m,i}$	$´t_{ig,l,i}$	$´t_{ig,s,i}$
c	e	c	e	c	e	c	e	c										
(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(s)	(–)	(s)	(s)	(s)	(s)	(s)	(s)	(s)	(s)
2	3	11	2	WR	NL	11	3	b	5	0.00	-5	0	-1.0	0.0	0.3	-5.8		
2	3	11	2	WR	NL	11	3	c	5	0.09	0	-0.7	-2.2	-1.4	0.1	-4.2	-5.8	-5.8
2	3	11	8	WR	SR	10	7	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SR	10	7	c	8	0.05	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SR	10	8	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SR	10	8	c	8	0.20	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SR	11	7	b	8	0.00	0	-0.7	-2.8	0.0	0.0	-3.5		
2	3	11	8	WR	SR	11	7	c	8	0.14	-2	-0.6	-2.9	-1.4	0.0	-6.9		
2	3	11	8	WR	SR	11	8	b	8	0.01	-2	-0.5	-2.4	0.0	0.0	-4.9		
2	3	11	8	WR	SR	11	8	c	8	0.59	-3	-0.3	-2.4	-1.4	0.0	-7.1	-7.3	
2	3	11	8	WR	SL	10	8	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SL	10	8	c	8	0.19	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SL	10	9	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SL	10	9	c	8	0.06	-8	0	0.0	0.0	0.0	-8.0		
2	3	11	8	WR	SL	11	8	b	8	0.01	-3	-0.3	-2.3	0.0	0.0	-5.6		
2	3	11	8	WR	SL	11	8	c	8	0.56	-3	-0.6	-2.3	-1.4	0.0	-7.3		
2	3	11	8	WR	SL	11	9	b	8	0.00	-3	-0.5	-2.2	0.0	-0.3	-6.0		
2	3	11	8	WR	SL	11	9	c	8	0.18	-3	-0.7	-2.2	-1.4	-0.1	-7.4	-7.5	-7.3
2	3	12	2	WL	NR	12	1	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
2	3	12	2	WL	NR	12	1	c	6	0.19	-6	0	0.0	0.0	0.0	-6.0		
2	3	12	2	WL	NR	12	2	b	6	0.00	-3	-0.3	-2.7	0.0	0.0	-6.0		
2	3	12	2	WL	NR	12	2	c	6	0.81	-2	-1	-2.7	-0.4	0.0	-6.1	-6.0	
2	3	12	2	WL	NL	12	2	b	6	0.00	-2	-0.2	-2.5	0.0	0.0	-4.7		
2	3	12	2	WL	NL	12	2	c	6	0.87	-2	-0.2	-2.5	-0.4	0.0	-5.1		
2	3	12	2	WL	NL	12	3	b	6	0.00	0	-0.7	-2.1	0.0	0.3	-2.6		
2	3	12	2	WL	NL	12	3	c	6	0.13	-1	-0.2	-2.1	-0.4	0.1	-3.6	-4.9	-4.9
2	3	12	8	WL	SR	12	7	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
2	3	12	8	WL	SR	12	7	c	8	0.20	-8	0	0.0	0.0	0.0	-8.0		
2	3	12	8	WL	SR	12	8	b	8	0.00	0	-0.4	-3.0	0.0	-1.1	-4.5		
2	3	12	8	WL	SR	12	8	c	8	0.80	-3	0	-3.0	-0.4	-0.4	-6.8	-7.1	

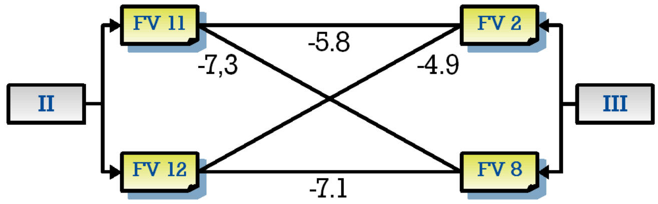
Stage		Signal group		Lane		Stream		b/c	$t_{ig,s,i}$	p	$\acute{}t_c$	t_{saf}	$\acute{}t_{cr,e} + \acute{}t_e$	$\acute{}t_{cr}$	$\acute{}t_{cl}$	$\acute{}t_{ig,m,i}$	$\acute{}t_{ig,l,i}$	$\acute{}t_{ig,s,i}$
c	e	c	e	c	e	c	e	c										
(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(s)	(–)	(s)	(s)	(s)	(s)	(s)	(s)	(s)	(s)
2	3	12	8	WL	SL	12	8	b	8	0.00	-3	-0.5	-2.8	0.0	-0.6	-6.9		
2	3	12	8	WL	SL	12	8	c	8	0.76	-4	-0.1	-2.8	-0.4	-0.2	-7.5		
2	3	12	8	WL	SL	12	9	b	8	0.00	-4	-1	-2.9	0.0	-0.2	-8.1		
2	3	12	8	WL	SL	12	9	c	8	0.24	-4	-0.8	-2.9	-0.4	-0.1	-8.2	-7.7	-7.1
3	5	2	8	NR	SR	1	7	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	1	7	c	6	0.04	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	1	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	1	8	c	6	0.15	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	2	7	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	2	7	c	6	0.16	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	2	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SR	2	8	c	6	0.65	-6	0	0.0	0.0	0.0	-6.0	-6.0	
3	5	2	8	NR	SL	1	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SL	1	8	c	6	0.14	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SL	1	9	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SL	1	9	c	6	0.04	0	-0.1	-2.7	-0.4	0.1	-3.1		
3	5	2	8	NR	SL	2	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SL	2	8	c	6	0.62	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SL	2	9	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NR	SL	2	9	c	6	0.19	-1	-0.3	-1.7	-1.4	0.1	-4.3	-5.5	
3	5	2	8	NL	SR	2	7	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	2	7	c	6	0.17	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	2	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	2	8	c	6	0.70	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	3	7	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	3	7	c	6	0.02	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	3	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SR	3	8	c	6	0.10	-6	0	0.0	0.0	0.0	-6.0	-6.0	
3	5	2	8	NL	SL	2	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SL	2	8	c	6	0.66	-6	0	0.0	0.0	0.0	-6.0		

Stage		Signal group		Lane		Stream		b/c	$t_{ig,s,i}$	p	$´t_c$	t_{saf}	$´t_{cr,e} + ´t_e$	$´t_{cr}$	$´t_{cl}$	$´t_{ig,m,\mathbf{i}}$	$´t_{ig,l,\mathbf{i}}$	$´t_{ig,s,\mathbf{i}}$
c	e	c	e	c	e	c	e	c										
(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(s)	(–)	(s)	(s)	(s)	(s)	(s)	(s)	(s)	(s)
3	5	2	8	NL	SL	2	9	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SL	2	9	c	6	0.21	-2	-0.2	-0.7	-1.4	0.1	-4.2		
3	5	2	8	NL	SL	3	8	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SL	3	8	c	6	0.10	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SL	3	9	b	6	0.00	-6	0	0.0	0.0	0.0	-6.0		
3	5	2	8	NL	SL	3	9	c	6	0.03	-6	0	0.0	0.0	0.0	-6.0	-5.6	-5.5
5	1	8	5	SR	EC	7	4	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SR	EC	7	4	c	8	0.02	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SR	EC	7	5	b	8	0.01	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SR	EC	7	5	c	8	0.16	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SR	EC	8	4	b	8	0.00	0	-0.9	-2.8	0.0	0.0	-3.7		
5	1	8	5	SR	EC	8	4	c	8	0.10	0	-0.1	-2.8	-1.4	0.0	-4.3		
5	1	8	5	SR	EC	8	5	b	8	0.03	-2	-0.5	-2.7	0.0	0.0	-5.2		
5	1	8	5	SR	EC	8	5	c	8	0.67	-3	-0.8	-2.8	-1.4	0.0	-8.0	-7.5	
5	1	8	5	SL	EC	8	4	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SL	EC	8	4	c	8	0.10	-3	0	-3.4	-1.4	0.0	-7.8		
5	1	8	5	SL	EC	8	5	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SL	EC	8	5	c	8	0.66	-3	-0.8	-2.8	-1.4	0.0	-8.0		
5	1	8	5	SL	EC	9	4	b	8	0.00	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SL	EC	9	4	c	8	0.03	-8	0	0.0	0.0	0.0	-8.0		
5	1	8	5	SL	EC	9	5	b	8	0.00	0	-0.8	-3.4	0.0	0.0	-4.2		
5	1	8	5	SL	EC	9	5	c	8	0.21	-3	-0.2	-3.4	-0.4	0.0	-7.0	-7.8	-7.5
5	1	8	11	SR	WR	7	10	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SR	WR	7	10	c	5	0.05	-5	0	-2.2	0.0	0.0	-7.2		
5	1	8	11	SR	WR	7	11	b	5	0.01	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SR	WR	7	11	c	5	0.14	0	-0.9	-2.8	-0.4	0.0	-4.1		
5	1	8	11	SR	WR	8	10	b	5	0.01	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SR	WR	8	10	c	5	0.19	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SR	WR	8	11	b	5	0.02	-2	-0.8	-3.4	0.0	0.0	-6.2		
5	1	8	11	SR	WR	8	11	c	5	0.58	-2	-0.1	-3.4	-1.4	0.0	-6.9	-6.1	

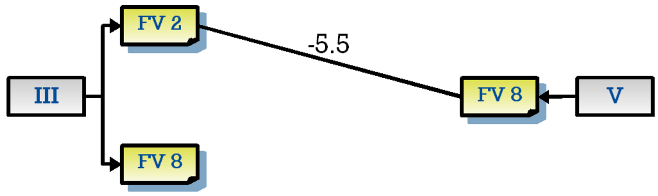
Stage		**Signal group**		**Lane**		**Stream**		**b/c**	$\mathbf{t_{ig,s,i}}$	**p**	$\acute{\mathbf{t}}_{c}$	$\mathbf{t_{saf}}$	$\acute{\mathbf{t}}_{cr,e} + \acute{\mathbf{t}}_{e}$	$\acute{\mathbf{t}}_{cr}$	$\acute{\mathbf{t}}_{cl}$	$\acute{\mathbf{t}}_{ig,m,i}$	$\acute{\mathbf{t}}_{ig,l,i}$	$\acute{\mathbf{t}}_{ig,s,i}$
c	e	c	e	c	e	c	e	c										
(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(–)	(s)	(–)	(s)	(s)	(s)	(s)	(s)	(s)	(s)	(s)
5	1	8	11	SL	WR	8	10	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SL	WR	8	10	c	5	0.19	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SL	WR	8	11	b	5	0.00	-2	-0.8	-2.9	0.0	0.0	-5.7		
5	1	8	11	SL	WR	8	11	c	5	0.57	-2	-0.1	-2.9	-1.4	0.0	-6.4		
5	1	8	11	SL	WR	9	10	b	5	0.00	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SL	WR	9	10	c	5	0.06	-5	0	0.0	0.0	0.0	-5.0		
5	1	8	11	SL	WR	9	11	b	5	0.00	-2	-0.4	-2.7	0.0	0.0	-5.1		
5	1	8	11	SL	WR	9	11	c	5	0.18	-1	-0.9	-2.7	-0.4	0.0	-5.0	-5.8	-5.8



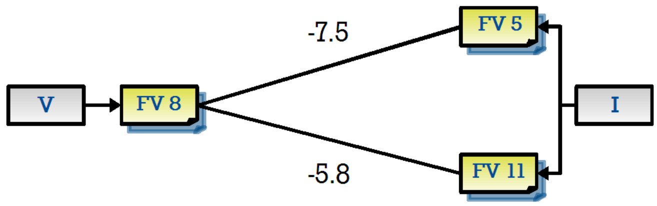
(a) Stage change I-II



(b) Stage change II-III



(c) Stage change III-V



(d) Stage change V-I

Figure 45: Conflict trees condensed to signal group level of example intersection (A 046, AP)

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