Traffic Signals in Motorcycle Dependent Cities

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Abstract

Traffic signal control was first invented in 1868 in Great Britain. Then, it has quickly spread across many other countries, what are now usually developed countries. Therefore, during a long history of development, traffic signals usually have been dealing with traffic in which four-wheel vehicles play an important role in motorised traffic.

However, as a result of an unequal development among countries, many Motorcycle Dependent Cities (MDCs) nowadays still exist mostly in the developing countries such as: Hanoi, Ho Chi Minh City (Vietnam), New Delhi (India), Taipei (Taiwan), Bangkok (Thailand), etc. In these cities, motorcycles play an important role in motorised traffic.

Because motorised traffic dominated by motorcycles was born later than car traffic, the complete application of traffic signals from developed countries, which usually deals with four-wheel vehicles, to these specific traffic conditions of two-wheel vehicles is usually ineffective. In practice, it has been causing many traffic problems at traffic signals.

To solve problems at traffic signals in MDCs, this study analysed the applicability of the German standard “Guidelines for Traffic Signals” (RiLSA, edition 2009) to establish a draft of Guidelines for Traffic Signals in MDCs. In order to achieve the goals and objectives, some contents of RiLSA needed to be modified, in which the minor modifications are directly written in the draft of Guidelines for Traffic Signals in MDCs. This study, therefore, focuses on the major modifications of the intersection layout engineering design, the signal program design, and the traffic signal control strategies. However, the four criteria: (i) traffic safety, (ii) traffic flow quality, (iii) environmental impacts, and (iv) economics are always considered when modifying any content of RiLSA, in which the first two criteria play an important role in MDCs. And overall, the traffic regulation at traffic signals has to obey the following priority in order: (i) pedestrians, (ii) cyclists, (iii) public transport, and (iv) motorised traffic.

For the intersection layout engineering design, it is necessary to take not only motorised traffic but also public transport, pedestrian traffic, and cycle traffic into account as a whole. However, the layouts for cycle traffic and pedestrian traffic can be applied from RiLSA. Therefore, some modifications on the intersection layouts are implemented for motorised traffic and public transport, in which the major aim is firstly to give priority to public transport, and secondly to give priority to motorcycles in motorised traffic.

For the signal program elements, unlike some available researches that consider only the equivalent factor converting motorcycles into passenger car units, this study has dealt with a series of the signal program elements such as cycle time, green time, amber time, intergreen time, etc., in which a new concept of saturation flow was used. This concept results from the homogeneous motorcycle traffic saturation flow and the homogeneous car traffic saturation flow, and of course this saturation flow depends on the motorcycle traffic volume as well as car traffic volume on the approach. It means that the saturation flow is not fixed as the homogeneous car or motorcycle saturation flow, but it varies depending on the proportion of motorcycles and cars in the traffic flow. From this new concept of the saturation flow, the formulas calculating the cycle time and the green time were formed. For the last two signal program elements, the amber time and the intergreen time were calculated based on the German method in which the positions of the stop-lines, the speeds as well as the deceleration rate of vehicles are taken into account.

For the traffic signal control strategies, according to RiLSA, there are three macroscopic control levels and three microscopic control levels, in which the microscopic control levels are activated from the macroscopic control levels. However, in MDCs, counting the number of vehicles requires too much effort or it seems to be impossible with conventional technologies. Therefore, only the macroscopic control level “time-dependent signal program selection (A1)” can be applied to MDCs. From this macroscopic control level, two microscopic control levels (fixed-time signal program (B1), and signal program adaption including green time adjustment (B2), phase swapping (B3), demand phase (B4), and time-offset adjustment (B5)) are activated in MDCs because the fixed-time signal program is always easily implemented, and the signal program adaption does not require efforts in counting the number of vehicles. Instead, it only requires detecting vehicles, and this is possible for two-wheel vehicles. The last microscopic control level, the signal program formation (B6), should not be applied to MDCs because it requires much effort in collecting traffic data online.

After having the results for some modifications, a draft of new “Guidelines for Traffic Signal in MDCs” was compiled. It includes six chapters: (0) Introduction, (1) Basic Principles, (2) Signal Program Design, (3) Interrelationships between Traffic Signal Control and Road Engineering Design, (4) Control Strategies, (5) Technical Design. Besides, it also has three annexes: Annex 1: Details on the Traffic Load, Annex 2: Traffic Flow Quality, and Annex 3: Traffic Engineering Calculation.

After testing this study at some signalised intersections in Hanoi and Ho Chi Minh City, these guidelines will be considered to be a foundation for establishing a Vietnamese Standard for Traffic Signals. The necessary “Formal Right Agreement“ between the German Forschungsgesellschaft für Straßen- und Verkehrswesen FGSV (Road and Transport Research Association) and the Vietnamese Ministry of Transport is in the approval process. Finally, also other MDCs, might consider applying this standard.

Zusammenfassung

Lichtsignalsteuerung wurde zuerst 1868 in Groß-Britannien eingeführt. Sie verbreitete sich schnell in weiteren Ländern, die heute meist sogenannte „entwickelte Länder" darstellen. Dementsprechend beschäftigte sich die Lichtsignalsteuerung in seiner langen Entwicklungsgeschichte mit Verkehr, in dem vierrädrige Fahrzeuge eine wesentliche Rolle spielen.

Allerdings existieren als Ergebnis der ungleichen Entwicklung verschiedener Länder heute viele vom Motorrad abhängige Städte (engl.: Motorcycle Dependent Cities – MDCs), z. B. Hanoi, Ho Chi Minh City (Vietnam), New Delhi (Indien), Taipei (Taiwan), Bangkok (Thailand), etc. In diesen Städten spielen Motorräder eine wichtige Rolle im motorisierten Verkehr.

Weil der von Motorrädern dominierte motorisierte Verkehr später entstanden ist als der Pkw-Verkehr, ist die vollständige Übertragung der Lichtsignalsteuerung aus entwickelten Ländern, die normalerweise auf vierrädrigen Fahrzeugen basiert, oft uneffektiv für die spezifischen Verkehrsbedingungen bei zweirädrigen Fahrzeugen.

Um die Probleme der Lichtsignalsteuerung in MDCs zu lösen, analysiert diese Studie die Anwendbarkeit der deutschen „Richtlinien für Lichtsignalanlagen (RiLSA, Entwurf 2009)" als Grundlagen eines Entwurfs von Richtlinien für die Lichtsignalsteuerung in MDCs. Um die bestehenden Anforderungen zu erfüllen, müssen einige Inhalte der RiLSA angepasst werden, von denen die geringen Anpassungen unmittelbar in den Richtlinien-Entwurf übernommen werden. Diese Studie konzentriert sich deshalb auf die umfangreichen Anpassungen bei der Knotenpunktgestaltung, des Signalprogrammentwurfs und der Steuerungsverfahren. Die vier Kriterien (i) Verkehrssicherheit, (ii) Qualität des Verkehrsablaufs, (iii) Umwelteinflüsse und (iv) Wirtschaftlichkeit werden jedoch bei der Anpassung von Inhalten der RiLSA immer berücksichtigt, wobei gerade die ersten beiden Kriterien eine wichtige Rolle in MDCs spielen. Insgesamt muss die Verkehrsregelung an Lichtsignalanlagen folgenden Prioritäten folgen: (i) Fußgänger, (ii) Radfahrer, (iii) öffentlicher Personenverkehr, (iv) motorisierter Verkehr.

Für den Knotenpunktentwurf ist es notwendig, nicht nur motorisierten Verkehr, sondern auch öffentlichen Personenverkehr, Fußgängerverkehr und Radverkehr zu berücksichtigen. Dafür können die Entwurfsvorgaben für Fußgänger und Radfahrer aus den RiLSA übernommen werden. Anpassungen des Knotenpunktentwurfs werden dagegen für den motorisierten Verkehr und den öffentlichen Personenverkehr vorgenommen, wobei das wichtigste Ziel die Priorisierung des öffentlichen Personenverkehrs ist, gefolgt von dem Ziel, Motorradverkehr zu priorisieren.

Für die Signalprogrammelemente beschäftigt sich diese Studie im Gegensatz zu anderen vorliegenden Studien, die nur Faktoren für die Umrechnung von Motorrädern in Pkw-Einheiten berücksichtigen, mit einer Reihe von Signalprogrammelementen wie Umlaufzeit, Freigabezeit, Gelbzeit, Zwischenzeiten etc., für die ein neues Konzept der Sättigungsverkehrsstärke genutzt wurde. Dieses Konzept geht von einem homogenen, gesättigten Verkehrsfluss für Motorräder und Pkw aus, und natürlich hängt die Sättigung von den jeweiligen Verkehrsstärken ab. Das bedeutet, dass die Sättigungsverkehrsstärke nicht als homogener Verkehrsstrom von Pkw oder Motorrädern betrachtet wird, sondern sie variiert in Abhängigkeit vom Verhältnis zwischen Motorrädern und Pkw im Verkehrsstrom. Aus diesem Konzept der Sättigungsverkehrsstärke wurden die Formeln für die Berechnung der Umlaufzeit und der Freigabezeit abgeleitet. Die letzten beiden Signalprogrammelemente, die Gelbzeit und die Zwischenzeit, werden entsprechend dem RiLSA-Verfahren berechnet, das auf der Lage der Haltlinien, den Geschwindigkeiten und der Bremsverzögerung der Fahrzeuge basiert.

Bezüglich der Steuerungsverfahren unterscheiden die RiLSA zwischen drei makroskopischen und drei mikroskopischen Steuerungsebenen, bei denen die mikroskopischen Ebenen von den makroskopischen Ebenen aktiviert wird. In MDCs stellt jedoch die Erfassung der Fahrzeuganzahl einen zu hohen Aufwand dar oder sie scheint unmöglich mit konventionellen Technologien. Deshalb kann in MDCs nur die die makroskopische Steuerungsebene „zeitplanabhängige Auswahl der Signalprogramme (A1)“ angewendet werden. Von dieser makroskopischen Steuerungsebene werden zwei mikroskopische Steuerungsebenen (Festzeitsignalprogramm (B1) und Signalprogrammanpassung einschließlich Freigabezeitanpassung (B2) , Phasentausch (B3), Phasenanforderung (B4) und Versatzzeitanpassung (B5)) in MDCs aktiviert, weil das Festzeitsignalprogramm immer leicht implementiert werden kann und die Signalprogrammanpassung keine Erfassung der Fahrzeuganzahl erfordert. Stattdessen benötigen sie nur eine Erfassung der Belegung, die auch für Motorräder möglich ist. Die letzte mikroskopische Steuerungsebene, die Signalprogrammbildung (B6), sollte nicht in MDCs eingesetzt werden, da sie zu hohe Anforderungen an die Echtzeit-Datenerfassung stellt.

Mit diesen Anpassungen wurde ein Entwurf von „Richtlinien für Lichtsignalanlagen in MDCs“ erstellt. Er beinhaltet sechs Kapitel: (0) Einführung, (1) Grundsätze, (2) Entwurf des Signalprogramms, (3) Wechselwirkungen zwischen Lichtsignalsteuerung und dem Entwurf von Straßenverkehrsanlagen, (4) Steuerungsverfahren, (5) Technische Ausführung. Außerdem enthält der Entwurf drei Anhänge: Anhang 1: Hinweise zur Verkehrsbelastung, Anhang 2: Qualität des Verkehrsablaufs, Anhang 3: Verkehrstechnische Berechnung.

Nach Feldversuchen zu den Ergebnissen dieser Studie an ausgewählten Knotenpunkten in Hanoi und Ho Chi Minh City wird erwogen, diesen Richtlinien-Entwurf als Grundlage für die Einführung als vietnamesische Richtlinien für Lichtsignalanlagen zu verwenden. Der erforderliche Vertrag zu den Verwendungsrechten zwischen der deutschen Forschungsgesellschaft für Straßen- und Verkehrswesen (FGSV) und dem vietnamesischen Verkehrsministerium ist in der Genehmigungsphase. Zukünftig werden vielleicht auch andere MDCs die Anwendung dieser Richtlinien erwägen.

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List of Abbreviations

BfV	Bundesministerium für Verkehr, Bau und Stadtentwicklung
CO	Carbon Monoxide
CO_2	Carbon Dioxide
CITA	International Motor Vehicle Inspection Committee
FGSV	Road and Transportation Research Association
HPC	Hanoi's People Committee
HCMC	Ho Chi Minh City
HCM	Highway Capacity Manual
HBS	German Highway Capacity Manual
HSRa	German Guidelines for Signalisation of Cycle Traffic
ITE	Institute of Transportation Engineers
ICCT	International Council on Clean Transportation
JICA	Japanese International Cooperation Agency
JSTE	Japan Society of Traffic Engineers
MDC	Motorcycle Dependent City
MDCs	Motorcycle Dependent Cities
MoT	Ministry of Transport of Vietnam
MUTCD	Manual on Uniform Traffic Control Devices
MCU	Motorcycle Unit
MC	Motorcycle
NO	Nitrogen Monoxide
NO_2	Nitrogen Dioxide
PM	Particle Matter
PCU	Passenger Car Unit
PCE	Passenger Car Equivalent
RiLSA	German Guidelines for Traffic Signals
RASt 06	German Guidelines for Urban Road Design, edition 2006
RAS-K-1	German Guidelines for Road Design, Section 1: At-grade Intersection
StVZO	German Road Traffic – Permission – Regulation
VOC	Volatile Organic Compound

1. Introduction

1.1. Background of the study

Nowadays, motorcycles play an important role in traffic in many cities in Asia and Africa. Until now, public transport systems in these cities are usually not good in services as well as in infrastructure, and they are, therefore, not good enough to attract passengers. Private cars are still extremely expensive not only in buying but also in using them (parking fee, fuel prices, etc.) for almost all citizens who usually have much lower income than those in developed countries. The proper choice for mobility is the motorcycle traffic mode. This choice is reasonable because, on the one hand, motorcycle is inexpensive, flexible, and has relatively high speed compared with car traffic. On the other hand, the objective conditions of the climate, such as not severe in the winter (no snow), also allow using motorcycles. In general, these cities show the following picture of traffic: low car usage, no good public transport systems, high ownership and usage of motorcycles, and relatively low bicycle usage. And regarding infrastructure, road traffic systems have been constructed based on car-oriented traffic.

However, besides the advantages above, there are also some problems caused by using too many motorcycles in the city, such as traffic safety, traffic congestion, and critical impacts on the environment.

As a matter of fact, the number of motorcycles has been steadily increasing for recent decades in many cities. For example, in Hanoi, motorcycle proportion was counted over 80% in the traffic flows, and most of the trips were made by motorcycles. However, in the future, the number of trips made by motorcycles will decrease to 30% - 50% by the year of 2020 (see Figure 1). It means that in over 10 years, motorcycles still play a very important role in traffic in Hanoi.

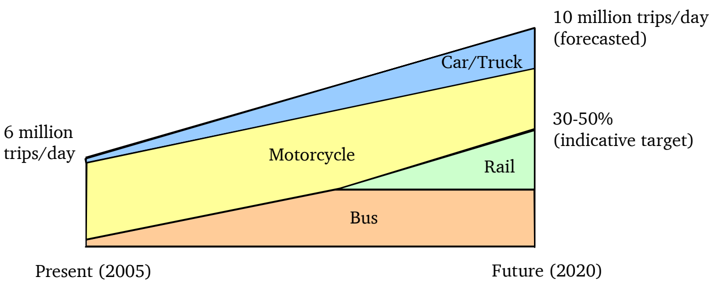


Figure 1: Planning target for 2020 in Hanoi
(JICA and HPC, 2007)

As a result, transport planners and traffic engineers have to face the problems caused by motorcycles. Therefore, it is very necessary to carry out studies regarding traffic management as well as traffic engineering under such specific traffic conditions.

This study on traffic signals in MDCs focuses on one of the most important measures of traffic management in order to contribute to the solution of problems at traffic signals in MDCs.

However, the approach of this study is based on the concept of traffic signals in Germany, a developed country which has very high quality traffic signal systems. In order to achieve the goals and objectives for the case of MDCs, it is very necessary to analyse in detail and point out what can be completely applied, what must be modified for the specific traffic conditions in MDCs. All of these issues will be presented in the next chapters.

1.2. Motivations and significance of the study

The first question is that it is necessary to conduct this study whereas traffic signals have been developed for over 140 years already (since 1868), and accompanied with thousands of studies. The answer on this question is considered to be the motivation and the significance of this study, and it will be addressed in the following texts.

Under any specific traffic condition, traffic signals should be considered in a suitable way, the evidences for this have been answered in reality, for example: particular considerations of public transport, pedestrian traffic, and cycle traffic at traffic signals are implemented. Another example in Münster city of Germany, that is, when cycle traffic plays an important role in the city, the new guidelines for traffic signals have been conducted, and established in 2007 in order to give priority and comfort to cyclists at traffic signals. Nowadays, when traffic demand has been increasing, traffic signals become a very important measure in traffic management. Therefore, some special forms of signalisation were born to satisfy specific requirements, e.g. partial signalisation, bottle-neck signalisation, lane signalisation, ramp metering control, etc.

Since motorcycle dependent cities have been formed in the 1990s, they caused many traffic problems, and transport planners as well as traffic engineers started to conduct studies to solve these problems. So far, in the field of traffic signals, some studies related to MDCs have been conducted already such as: some issues on capacity of intersection (Phan Cao Tho, 2003), characteristics of traffic flows at signalised intersection (Phan Cao Tho, 2003), some issues on saturation flow rate at signalised intersections (Phan Cao Tho, 2003), analysis of motorcycle behaviour at midblocks and signalised intersections (Chu Cong Minh, 2007), different models of saturation flow in traffic dominated by motorcycles (Hien Nguyen and Frank Montgomery, 2007). However, these studies dealt with only some specific issues at traffic signals under mixed traffic conditions.

Unlike the above studies, this study will go through a series of issues about traffic signals in MDCs. Of course, the available studies related to traffic signals in MDCs will be analysed in detail to consider their utilisation. Then, all of these issues will be considered within their interrelationships in order to solve the existing problems at traffic signals in MDCs.

Finally, there is, until now, no guideline for traffic signals in MDCs. Therefore, establishing the draft of “Guidelines for Traffic Signals in MDCs” is a major motivation of this study.

1.3. Goals and objectives of the study

As mentioned above, the main goal of this study is to establish a draft of the “Guidelines for Traffic Signals in MDCs” based on the “German Guidelines for Traffic Signals (RiLSA, edition 2009)”. In order to achieve this goal, the following issues have to be clearly addressed:

- Problems at traffic signals in MDCs have to be comprehensively analysed, in which three major problems are defined: (i) traffic safety, (ii) intersection layout, (iii) signal program design.

- Some models of the intersection layout in MDCs have to be proposed in which all types of vehicles such as motorcycles, passenger cars, buses, cycles and pedestrians have to be taken into account.
- Signal program elements such as cycle time, green time, intergreen time, and amber time have to be properly calculated according to the models of the intersection layout.
- The control strategies have to be defined according to traffic situations, and to the models of the intersection layout in MDCs.
- Some investigations have to be implemented to check the theoretical results of this study.
- This study has to be tested at some signalised intersections in Hanoi and HCMC.
- The Guidelines for Traffic Signals in MDCs have to be compiled based on this study.

1.4. Scope of the study

As mentioned above, traffic signal control is an important operational measure of the road traffic management. Therefore, in large scale, this study is within the framework of traffic management, but in detail, it is limited in the field of traffic signals under the specific traffic conditions in MDCs.

More deeply, this study is conducted mainly based on the German Guidelines for Traffic Signals (RiLSA, edition 2009) as well as other materials related to traffic signals in Germany such as: German Highway Capacity Manual (FGSV, 2001), Guidelines for Signalisation of Cycle Traffic (FGSV, 2005), Supplement for RiLSA edition 1992 (FGSV, 2003), Guidelines for Urban Road Design (FGSV, 2006), etc. However, the concepts from other countries, especially from the United States and Great Britain, on individual issues of traffic signals are sometimes mentioned in order to make these issues clear, comprehensive and worldwide.

Regarding the specific traffic conditions in MDCs, the general traffic data as well as available materials are collected from some countries such as: Taiwan, India, Thailand, etc. However, the detailed traffic data being used in this study are collected in Vietnam, where there are many typical motorcycle dependent cities such as Hanoi and Ho Chi Minh City.

To achieve the final goal to establish a draft of “Guidelines for Traffic Signals in MDCs”, some values of the signal program elements in RiLSA are kept unchanged, some other elements must be modified. However, the reasons for changing or unchanging these values are always given. Before this draft becomes official, it is necessary to test this study at some intersections.

1.5. Methodology and structure of the study

As mentioned above, because traffic signal control is only one measure of traffic management, the methodology and structure of this study are implemented according to the transport planning process of Germany, which was defined by the German Forschungsgesellschaft für Straßen- und Verkehrswesen FGSV (Road and Transport Research Association) (see Figure 2).

The pre-orientation which indicates deficiencies as well as political decisions, and legal requirements are sometimes mentioned in chapter 1 (introduction) and chapter 2 (traffic problems in MDCs), in which each city has its own deficiencies. However, almost all MDCs, in general, lack comprehensive knowledge as well as experiences about traffic signals. Therefore, they do not have clear concepts as well as official policies and guidelines for traffic signals.

The analysis of problems is presented in chapter 2. This chapter emphasizes problems at traffic signals including problems of traffic safety, problems of the intersection layout and problems of the signal program design in which these problems are analysed according to the criteria of traffic safety and traffic flow quality.

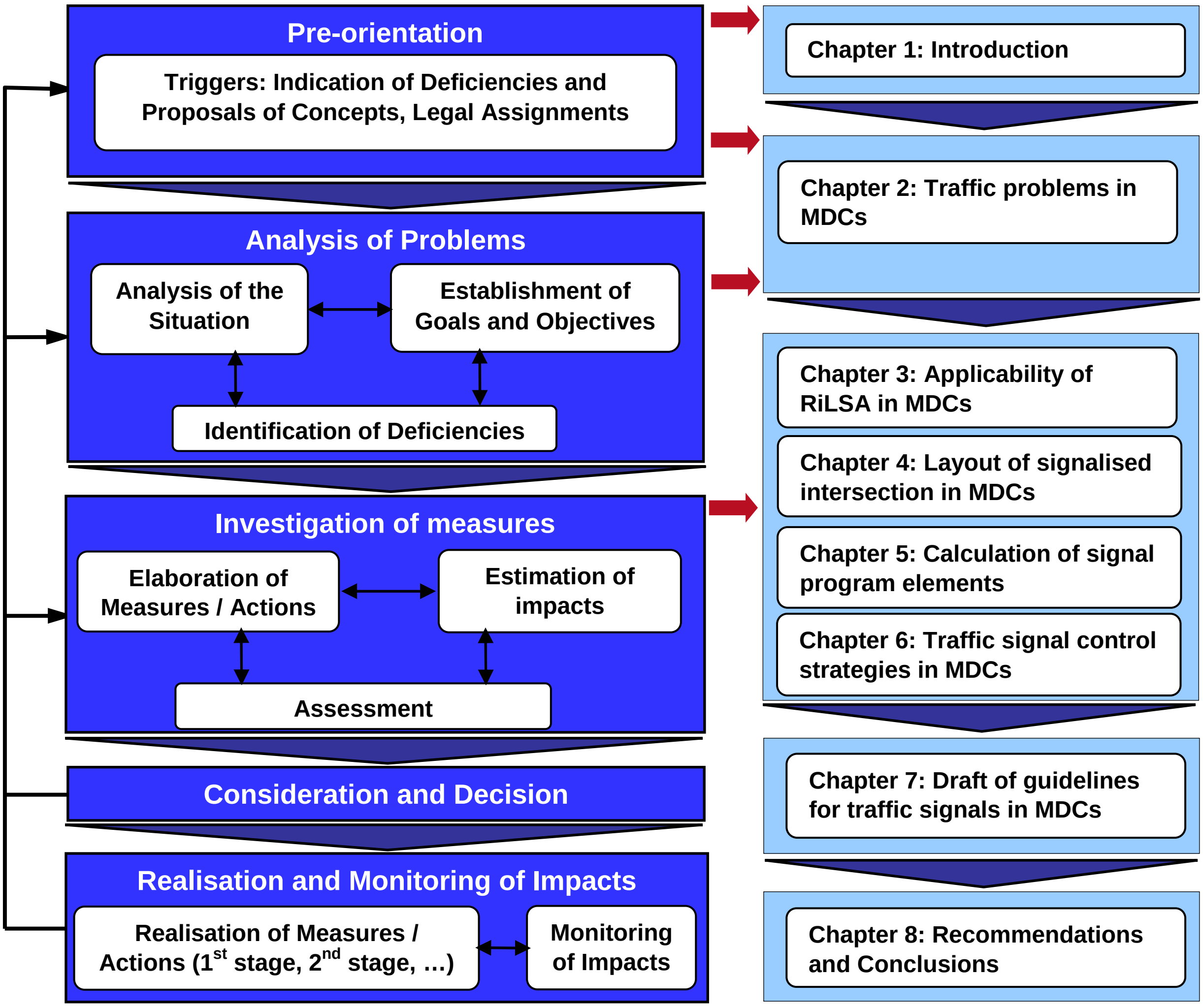


Figure 2: Transport planning process

(FGSV, 2001)

The investigation of measures is presented in chapter 3, chapter 4, chapter 5, and chapter 6 in which chapter 3 analyses the applicability of RiLSA edition 2009 in MDCs. In order to apply RiLSA, chapter 4, chapter 5, and chapter 6 have to be researched in which chapter 4 (layout of signalised intersections in MDCs) proposes some models of the intersection layout for MDCs. Then, chapter 5 (calculation of signal program elements) and chapter 6 (traffic signals control strategies) are implemented based on the intersection layouts that were proposed in chapter 4. Overall, all of these procedures have to be evaluated by the criteria of traffic safety and traffic flow quality. Chapter 7 (draft of "Guidelines for Traffic Signals in MDCs") is seen as the major product of this study. People, who are responsible for traffic signals in a city, can base on this draft to make a decision for implementing the official guidelines for traffic signals of the city. Finally, some recommendations and conclusions are given in chapter 8.

1.6. Definition of Motorcycle Dependent City (MDC)

The definition of a Motorcycle Dependent City has been presented in the doctoral dissertation: ''Traffic Management in Motorcycle Dependent Cities'' by Khuat (2006).

According to Khuat (2006), the motorcycle dependence is defined by three groups of indicators as shown in Table 1, in which the level of dependence on motorcycle traffic is also defined.

Table 1: Indicators for defining MDC

Indicators			Level					
			Low		Medium		High	
Main criteria	**Sub-criteria**	**Measurements**	**Value**	**Grade point**	**Value**	**Grade point**	**Value**	**Grade point**
Vehicle ownership	Motorcycle ownership	MCs /1000 inhabitants	<150	1	150-350	2	>350	3
	Private car ownership	PCs /1000 inhabitants	<150	3	150-350	2	>350	1
Availability of Alternatives to motorcycle and car	Bus transport availability	Buses /1000 inhabitants	<1	3	1-2	2	>2	1
	Bicycle availability	Bicycles /1000 inhabitants	<150	1	150-350	2	>350	3
Use of motorcycle	Motorcycle shared in the traffic flow	% of MCs in the traffic flow (in vehicle unit)	<30%	1	30-50%	2	>50%	3
	Modal split of Motorcycle	% of Trips by MC	<20%	1	20-40%	2	>40%	3
	Modal split of Public Transport	% of Trips by Public Transport	<20%	3	20-40%	2	>40%	1
	Modal split of Private Car	% of Trips by Cars	<20%	3	20-40%	2	>40%	1
	Modal split of NMT	% of Trips by NMT	<20%	3	20-40%	2	>40%	1
Average grade point (GPA)			**>2.5 :**	**high dependence**				
			= 2.0 - 2.5 :	**medium dependence**				
			<2.0 :	**low dependence**				

(Khuat, 2006)

In general, a typical motorcycle dependent city has some characteristics such as: motorcycle ownership is higher than 350 per 1000 inhabitants; private car ownership is lower than 150 per 1000 inhabitants; public transport availability is lower than 1 bus per 1000 inhabitants; modal split of motorcycle is higher than 40%, while modal splits of private car and public transport are lower than 20%, and modal split of non-motorised transport is about 30% to 50% (Khuat, 2006).

However, proportion of motorcycles sharing in the traffic flow varies depending on the function of roads as well as the classification of roads. In most cases, the proportion of motorcycles in the traffic flow is higher than 50%. But on the ring-roads or the arterial roads outside the city centre, this proportion may be lower (from 30% to 50%).

1.7. International review of Guidelines for Traffic Signals

According to Schnabel (1975), traffic signal control was first operated on December 10th 1868 in front of the parliament building in London. The signals ''Caution'' and ''Stop'' were of the semaphore-arm type with red and green gas lamps for night use. By the signal ''Caution'', all person in charge of vehicles and horses are warned to pass over the crossing with care, and due regard to the safety of foot passengers. The signal ''Stop'' will only be displayed when it is necessary that vehicles and horses shall be actually stopped on each side of crossing to allow the passage of persons on foot.

In 1913, James Hoge first inaugurated traffic signals in Cleveland/USA. In 1917, co-ordination of intersections was operated by traffic signals in Salt Lake City. In 1928, traffic-actuated control was operated in USA (Schnabel, 1975).

According to Webster and Cobbe (1966), in 1918 the first manually operated three-colour light signals were installed in New York, and in 1925 manually operated coloured light signals were used by the police in Piccadilly, London.

Through a long history of development, until now, many countries, more or less, are having their own guidelines or standard for traffic signals. However, there are also many countries, which do not have their own standard, and these countries are having more or less difficulties in applying the standards from abroad because each country has its own traffic policy as well as own specific traffic conditions.

In this sub-chapter, some guidelines and standards for traffic signals of some countries are going to be generally introduced. Of course, it is impossible to introduce all the guidelines and standards all over the world due to the limitation of languages as well as of the material collection. And the following introductions are also not intending to compare between the standards from country to country.

In Austria, according to Boltze, Friedrich, Jentsch, Kitller, Lehnhoff, Reusswig (2006), there is no detailed specific guidelines for traffic signals in RVS (Richtlinien und Vorschriften für den Straßenbau) of Austria.

In Switzerland, according to Boltze, Friedrich, Jentsch, Kitller, Lehnhoff, Reusswig (2006), there have been many standards related to traffic signals since 1992, such as: SN 640 832 (traffic signal systems, 1992a); SN 640 837 (traffic signal systems – transition time and minimum times, 1992b); SN 640 838 (traffic signal systems – intergreen time, 1992c); SN 640 833 (traffic signal systems – using, 1994a); SN 640 836 (signal head design, 1994b); SN 640 834 (traffic signal systems – signal phasing, 1996); SN 640 835 (traffic signal systems – estimating capacity, 1997); SN 640 842 (traffic signal systems – acceptance, 1998); SN 640 802 (traffic influence – lane signal systems, FLS, 2000b); SN 640 836-1 (traffic signal systems – signals for visual-handicapped, 2000d); SN 640 855c (signalisation of construction site on roads and highways, 2000e); SN 640 886 (temporary signalisation on major and minor roads, 2002); SN 640 839 (traffic signal systems – considerations of public transport at traffic signal systems, 2003a); SN 640 840 (traffic signal systems – coordination on arterials with the method of green band point of intersection, 2003b).

In Great Britain, according to Boltze, Friedrich, Jentsch, Kitller, Lehnhoff, Reusswig (2006), traffic signals and signal sequence were not compiled in an exclusive chapter in “Traffic sign regulations and general directions (TSRGD)” (DFT, 2003b). In “Design Manual for Roads and Bridges”, traffic signals and signal sequence were not included (Highways Agency et al., 2005). One major part for Puffin-systems was in the “Specification for Traffic Controller”. Furthermore, there were many traffic advisory leaflets of the Department for Transport (DFT), which concern Puffin-systems (DFT, 2001; 2002a).

In France, according to Boltze, Friedrich, Jentsch, Kitller, Lehnhoff, Reusswig (2006), traffic signals such as: the signal head description as well as the permitted signal sequence for individual signal heads, were written in chapter II, article 110C in the “Instruction Interministérielle sur la Signalisation Routière” (Ministre de L’interieur et le Ministre de L’equipement, du logment, des Transports et de Léspace, METL, 1996).

In Japan, there was the “Manual on Traffic Signal Control” established by Japan Society of Traffic Engineers (JSTE) in 1994 in Japanese language, but it was not the official guideline and also not more developed (Boltze, Kitller, Nakamura, 2006). Then, in 1998, traffic signals were compiled as a small part in “the Planning and Design of At-Grade Intersections” established by JSTE in English language such as section 3.3: basic concept on the capacity of signalised intersections, section 3.4: computation of saturation flow rate, section 3.5: level of service and capacity of signalised intersection, section 3.6: examination of signalised intersection capacity and examples of its calculation. In 2004, this book was revised and renamed “Manual on At-grate Intersection Planning and Design’’. In 2006 JSTE established the Japanese Edition 2006 of Manual on Traffic Signal Control (Tang and Nakamura, 2007).

In the United States, traffic signals were compiled as a part of the Manual on Uniform Traffic Control Devices (MUTCD). Table 2 shows the evolution of the MUTCD:

Table 2: Evolution of the MUTCD

Year	Title	Revisions Issued
1927	Manual and Specification for the Manufacture, Display, and Erection of U.S. Standard Road Markers and Signs	4/29, 12/31
1930	Manual on Street Traffic Signs, Signals, and Markings	None
1935	Manual on Uniform Traffic Control Devices for Street and Highways	2/39
1943	Manual on Uniform Traffic Control Devices for Street and Highways – War Emergency Edition	None
1948	Manual on Uniform Traffic Control Devices for Street and Highways	9/54
1961	Manual on Uniform Traffic Control Devices for Street and Highways	None
1971	Manual on Uniform Traffic Control Devices for Street and Highways	11/71, 4/72, 3/73, 10/73, 6/74, 6/75, 9/76, 12/77

1978	Manual on Uniform Traffic Control Devices for Street and Highways	12/79, 12/83, 9/84, 3/86
1988	Manual on Uniform Traffic Control Devices for Street and Highways	1/90, 3/92, 9/93, 11/94, 12/96, 6/96, 1/00
2000	Manual on Uniform Traffic Control Devices for Street and Highways – Millennium Edition	6/01

(Roger P.Roess, Elena S.Prassas, William R.Mcshane, 2004)

Until now, the United States has the latest version of the MUTCD (edition 2003) in which traffic signals were presented in part 4 “Highway Traffic Signals”. Table 3 shows general contents of this part.

Table 3: Contents of part 4 in the MUTCD, edition 2003

Item	Content	Page
Chapter 4A	General	4A-1
Chapter 4B	Traffic control signals - General	4B-1
Chapter 4C	Traffic control signal needs studies	4C-1
Chapter 4D	Traffic control signal features	4D-1
Chapter 4E	Pedestrian control features	4E-1
Chapter 4F	Traffic control signals for emergency vehicle access	4F-1
Chapter 4G	Traffic control signals for one-lane, two-way facilities	4G-1
Chapter 4H	Traffic control signals for freeway entrance ramps	4H-1
Chapter 4I	Traffic control for movable bridges	4I-1
Chapter 4J	Lane-use control signals	4J-1
Chapter 4K	Flashing beacons	4K-1
Chapter 4L	In-roadway lights	4L-1

(http://mutcd.fhwa.dot.gov/pdfs/2003r1r2/ch4.pdf)

In China, until now, there is no national standard for traffic signals. In order to establish the national standard, Ministry of Construction of China has organized a research group led by Tongji University to address the first version of Manual on At-grade Urban Intersection Planning and Design (Tang and Nakamura, 2007). However, in 2006, the German Guidelines for Traffic Signals (RiLSA, edition 1992) was translated into Chinese language by Keping Li and published by Chinese Architecture & Construction Press. This Chinese version was used as a reference for traffic engineers in China.

In Vietnam, according to Nguyen, Q.T (2007), since 1954, traffic signals were appeared and operated manually with two phases. In 1982, Vietnam used two-phase semi-automatic traffic signal systems. Since 2000, Vietnam has been using automatic traffic signal systems. In 2003, the control with three phases, leading and lagging green time at some intersections started applying. Furthermore, ‘’Green Wave’’ on some one-way arterials was developed. However, all of these traffic signal systems were imported from abroad including traffic engineering design and operation. Therefore, until now, traffic signal systems from France, Germany, Japan, etc. still exist in Hanoi, but there is no any official standard or guideline for traffic signals in Vietnam, and of course Hanoi has to face the non-synchronisation of traffic signal systems. Furthermore, in spite of the exploration of motorcycle traffic since the 1990s, most of these traffic signal systems were still designed for car traffic. According to Nguyen, Q.D (2007), until now (2007), Vietnam did not have guidelines or standards as well as materials and textbooks concerning traffic signals. He extracted article 41 of the Vietnamese traffic law: “road traffic signal devices including: (i) traffic signal systems, (ii) traffic signs, (iii) marking, (iv) traffic post, traffic railing, retaining wall, kilometre post, etc.”. Although the last three items have been concretized by the standard 22TCN 237-01, now being replaced by 22TCN 237–07 by the Ministry of Transport of Vietnam, traffic signal systems have been not concretized yet. Nevertheless, traffic signal systems have been rarely and poorly mentioned in the traffic law. For example, article 10 of the Vietnamese traffic law says: “traffic signal light includes three colours, the meaning of each colour is expressed as follows: a) the green signal is allowed to go, b) the red signal is prohibited to go, c) the yellow signal warns of the signal changing. When the yellow signal turns on, the drivers must stop in front of the stop-line except the drivers, who have already been behind the stop-line, are allowed to continue to go, d) the yellow flashing signal means that drivers are allowed to go, but must pay attentions”. However, in the traffic engineering point of view, item c) above is incorrect because, in practice, when the yellow signal turns on, there are still drivers, who could not stop, have to continue to cross the stop-line. Until November of 2007, the Vietnamese-German Symposium on Traffic Signal Control was organized by the University of Transport and Communications of Vietnam (UTC) to discuss comprehensively about traffic signals, and to introduce the translation of RiLSA (edition 1992) in Vietnamese language. Then, this Vietnamese version became a reference for Vietnamese traffic engineers. Both Nguyen, Q.T (the professor in UTC) and Nguyen, Q.D (the professor in the University of Construction of Vietnam), in this symposium, asserted that it was very necessary to research and establish the Guidelines for Traffic Signals in Vietnam.

In Germany, the first traffic signal control was operated in Potsdamer Platz in Berlin in 1924. Then, the development of traffic signals was interrupted because of the Second World War (Schnabel, 1975). Later, the German Guidelines for Traffic Signals (RiLSA) has step by step been established, and it had a long history. According to Boltze (2007), the first version, which was very much based on experience of a very small expert group under the supervision of Retzko in Darmstadt, was delivered in 1964. It had a 39-page document in a small format. This version was reviewed in 1966. The first comprehensive “RiLSA” has been developed since 1968 and was published in 1977, already with 104 pages in A4 format. After minor changes in 1981 and an additional paper in 1985, which dealt with the consideration of public transport, bicycles and pedestrians, the 1992-version became official, which is in principle still the current version. After a long period of time applying RiLSA 1992 with so much highly valuable experiences, in 2003, some old contents in RiLSA 1992 were developed and some new contents were added in order to be adapted for the development of traffic. All these changes were presented in the “Teilfortschreibung 2003” (FGSV, 2003). However, the questions of capacity and quality of traffic flow were shifted to the German Highway Capacity Manual HBS (FGSV, 2001).

Also in 2003, the RiLSA 1992 was translated into English language with minor modifications. Then, the English version was translated into Vietnamese language in 2007, and the original RiLSA 1992 was translated into Chinese language in 2006 as mentioned above.

In 2006, the “Analyse und Bewertung neuer Forschungserkenntnisse zur Lichtsignalsteuerung” was conducted by Boltze, Friedrich, Jentsch, Kitller, Lehnhoff, Reusswig to do a research for an amendment to RiLSA 1992. As a result, in 2009, RiLSA 1992 was revised including the “Teilfortschreibung 2003”. Consequently, the RiLSA edition 2009 was established, and it reflects the latest state of the art in the field of traffic signals in Germany.

Currently, there are many official guidelines, handbooks, and materials related to traffic signals in Germany (see Table 4). However, RiLSA 2009 is the most important and comprehensive one among them.

Table 4: The current German guidelines relating to traffic signals

Text code	***Title***	***Related contents***	***Published year***
RiLSA	Guidelines for Traffic Signals	All the chapters	2009
HBS	German Highway Capacity Manual	Chapter 6: Signalised Intersection	2001
HSRa	Guidelines for signalization of cyclist	All the chapters	2005
-	Signalization for cycle traffic	Applying to Münster city only	2007
RASt 06	Guidelines for Urban Road Design (R1)	6.3.4.1. Guidance and Signalization of Pedestrian Traffic 6.3.4.2. Guidance and Signalization of Cycle Traffic 6.3.4.3. Guidance of Public Transport 6.3.5.11. Signalization of large round-about intersections	2006
RAS-K-1	Guideline for Road Design (RAS). Part: Intersection (RAS-K) Section 1: At-grade Intersection RAS-K-1	Scattered in some sections	1988

In order to gain the Guidelines for Traffic Signals in MDCs, especially for cities in Vietnam, the applicability of RiLSA 2009 will be analysed and estimated in chapter 3.

2. Traffic Problems in MDCs

2.1. General problems and situations

In general, traffic problems in MDCs have already been discussed by Khuat (2006) including traffic accidents, traffic congestion, and environmental impacts.

In this study, traffic problems in MDCs will be summarized and added with updated data. Then, traffic problems at traffic signals will be analysed in order to find out solutions that will be presented in detail in the next chapters.

2.1.1. Traffic accidents

For recent years, MDCs have been facing traffic accidents, especially the accidents involved with motorcycles. Hsu (2003) had some statistics on the accidents involving with motorcycles in some countries such as: Taiwan 51.2%, Malaysia 49%, and Vietnam 73%. According to the statistic of the Road Traffic Police Administration of Vietnam that was reported in the International Scientific Conference for preventing accidents on the 26th and 27th of October in 2006, the number of road traffic accidents has been increasing during the first ten months in 2006, of which 73% was caused by motorcycles. Only during the first eight months of 2006, there were approximately 10.000 traffic accidents. In 2005 the whole country had 14.141 traffic accidents, of which 73.4% was caused by motorcycles. Furthermore, according to the National Traffic Safety Committee, road traffic accidents made approximately 13.000 people died in 2006 in Vietnam. According to the General Statistics Office of Vietnam, there were 14.600 road traffic accidents in 2007 that made approximately 13.200 people died and 10.500 people injured.

Since December 15th of 2007, an obligatory regulation of wearing helmet was established by the Vietnamese government. The impact of this regulation brought a certain effect. As a result, according to the Vietnamese Ministry of Transport, during the first eleven months of 2008, the number of people died by traffic accidents reduced by 1.486 people (12,86%), and the number of people injured reduced by 2.435 people (25,45%) compared with those at the same time of 2007.

Although traffic flows in urban areas are at slow speeds, severe accidents still occur very often for motorcycle riders, especially accidents between motorcycles, cars, and buses (see Figure 3 and Figure 4).

Figure 3: An accident on an urban road **Figure 4: Sequent accidents**

(Spiegel newspaper, 2007) (http://vnexpress.net, 2008)

Besides, pedestrians also have high risk of accident when they cross the roads with full of motorcycles surrounding them (see Figure 5 and Figure 6).

All these accidents resulted from the mixed traffic flows dominated by motorcycles and from the weakness in traffic management.

Figure 5: Chaotic traffic at intersection

(Nguyen, H.M, 2007)

Figure 6: High risk of accident for pedestrians

(Hoang Ha, 2006)

2.1.2. Traffic congestion

Traffic congestion is a big problem not only in MDCs but also in many cities all over the world. However, the reasons for traffic congestion in MDCs are different. MDCs have the specific traffic conditions dominated by motorcycles, but currently, most of the measures for traffic management come from developed countries where cars are highly dominated. Therefore, in order to minimize traffic congestion, individual traffic management measures in developed countries must be considered carefully in order to apply to MDCs in a suitable way. In addition, the critical behaviours of motorcycle riders also contribute to traffic congestion because motorcycle riders have to suffer more emission and more severe weather condition than car drivers due to they are not shielded by an enclosed compartment.

From the data in Table 5 and Table 6, it is seen that the motorcycle ownership rates in Hanoi and Ho Chi Minh City are very high, and the number of motorcycle trips per day is, therefore, extremely high comparing with that of cars and buses.

Table 5: Vehicle ownership rates among Hanoi and HCMC households

Vehicle Type		Hanoi 2005	HCMC 2002
Car		1.8	1.7
Motorcycle	More than 2	44.7	58.9
	one	39.8	33.8
Bicycle		11.5	4.4
None		2.3	1.3
Total		100	100

(JICA and HPC, 2007)

Table 6: Urban transportation demand in Hanoi

Mode		No. of Trips (000/day)			Modal Share (%)	
		1995	2005	2005/1995	1995	2005
Vehicle	Bicycle	2,257	1,592	0.7	73.2	25.1
	Motorcycle	632	4,047	6.4	20.5	63.8
	Car/ Taxi	7	227	32.4	0.2	3.6
	Bus	21	427	20.3	0.7	6.7
	Others	165	47	0.3	5.4	0.7
	Subtotal	3,082	6,340	2.1	100	100
Walking		3,141	2,173	0.7	50.5	25.5
Total		6,223	8,513	1.4	100	100

(JICA and HPC, 2007)

As a result, traffic congestion frequently occurs in peak hours in a large area, especially in the morning and in the afternoon. In urban areas in Hanoi, traffic flows dominated by motorcycles on the main corridors travel at very slow speeds, from 5 to 10 km/h in peak hours, especially many traffic flows almost do not move for a long period of time (from 15 to 30 minutes) under the severe sunny weather and emission. Most of these phenomena originate from congestion at intersections. However, traffic control measures as well as the infrastructure at these intersections are still ineffective, and there is lack of comprehensive knowledge about that. Consequently, there are too many conflicts caused by motorcycles at intersections that lead to congestion. Figure 7 and Figure 8 below show the pictures of traffic congestion that are usually seen in Hanoi and Ho Chi Minh City.

Figure 7: Traffic congestion at intersection

(http://tienphongonline.com.vn, 2007)

Figure 8: Traffic congestion along road

(http://tienphongonline.com.vn, 2006)

2.1.3. Environmental impacts

- **Air pollution**

According to the International Motor Vehicle Inspection Committee (2006), there are about 200 million motorcycles in Asia. Vehicle emissions in emerging Asian countries account for 31% PM, 13% NO_X, 30% VOC, 24% CO of the world, of which, motorcycle emissions contributed 29% PM, 7% NO_X, 69% VOC and 61% CO. Therefore, Asian motorcycles contribute a considerable amount of PM, CO and VOC emissions.

The number of motorcycles in cities in Vietnam has been increasing without monitoring vehicle's quality. Consequently, air pollution reached an alarming level. According to the International Council on Clean Transportation (ICCT, 2007), air pollution in some places in Hanoi has reached 500 $\mu g/m^3$ in which emission from motorcycles played a major cause, this value was nearly equal to air pollution in fog in London in 1952 that made approximately 4000 people died. In respect of economy, according to the computation of the Institute for Labour Heath of Vietnam (2007), every year Hanoi lost 20 million USD and Ho Chi Minh City lost 50 million USD because of emission from motorcycle traffic. In addition, according to the Environmental Protection Office (2007), in Ho Chi Minh City, at some intersections such as: Hang Xanh roundabout, Phu Lam roundabout, Dinh Tien Hoang – Dien Bien Phu intersection, the amount of PM_{10} has reached from 3 to 7 times more than that of the Vietnamese standard (average daily $PM_{10} \leq 150$ $\mu g/m^3$ according to the Vietnamese standard TCVN 5937:2005, whereas this value in German standard is 50 $\mu g/m^3$).

To protect the environment from road traffic, in 2008, the Ministry of Transport of Vietnam promulgated a decision for monitoring quality and protecting environment in producing, assembling, importing motorcycles and three-wheel motor vehicles for handicapped, in which emission from vehicles must be ensured at least equivalently to the Euro II standard.

- **Noise Pollution**

Motorcycles are one of the significant sources of traffic noise, and they present a unique situation. Firstly, unlike cars, trucks, and buses, tire noise contributes rather insignificantly to the overall amount of noise produced by motorcycles (Sharp & Donovan, 1979). Thus, the type of engine, acceleration rate, and other issues that are relevant to the engine system rather than the tires become more important when considering motorcycles as a noise source. Secondly, unlike passengers in cars, trucks, and buses, the motorcycle riders are not shielded by an enclosed compartment from the noise produced by their vehicle. Thirdly, motorcycles can be particularly noisy. Cars generally produce noise levels in the range of 67-75 dB, whereas motorcycle noise generally ranges from 72-83 dB, but can reach levels as high as 120 dB immediately behind the cycle (Burgliarello et al., 1976).

According to the Vietnamese standard TCVN 6436:1998, the maximum permitted noise level of a motorcycle with an engine capacity up to 125 cm^3 is 95 dB, and this value is 99 dB for the engine capacity more than 125 cm^3.

2.1.4. General traffic situations

Almost all motorcycles in MDCs have an engine capacity of from 70 cm^3 to 150 cm^3, in which the engine capacities from 80 cm^3 to 125 cm^3 are very popular. These kinds of vehicles can be driven much more flexibly than other motorcycles in developed countries that are usually bigger in size and have higher engine capacity. According to Hsu (2003), motorcycles in almost all Asian countries have the following characteristics:

- Motorcycles are small in size (2m x 1m), having manoeuvring flexibility
- Motorcycles are agile to weave through queues in congestion areas
- Motorcycles usually drive in lateral lanes because of safety reasons
- Motorcycles have cruise speed lower than that of cars

- Motorcycles have higher acceleration rate, and the motorcycle riders have shorter reaction time than that of cars at the intersections when the green time starts.
- Motorcycles tend to get in front of the queue at the intersection and overcome cars in the inner intersection areas during the green time.

In urban areas in Vietnam, motorcycles, cars, public transport, trucks are mixed on the roads, but bicycles usually have a lateral exclusive lane on the carriageway and join signalisation with motorised traffic. Pedestrians go on the sidewalk.

From the above characteristics of traffic, traffic management measures should be properly considered for MDCs, in which traffic signals is one of the very important measures.

2.2. Problems at traffic signals

The design of a traffic signal system covers the selection of the control strategy, the traffic engineering description of control, the calculation of the signal program elements as well as the road traffic engineering design of the intersection, road section or part of a network including the corresponding traffic control measures (FGSV, 1992). All of these are intending to increase traffic safety and/or improve traffic flow quality.

Compared to MDCs, until now, almost all cities do not have their own guidelines for traffic signals. They completely apply the design methods from developed countries considering only the equivalent factor for converting motorcycles into passenger car units. They used traffic volume in passenger car units as input data for the design of the fixed-time signal control. As a result, all the traffic signal systems in MDCs are having problems of traffic safety. These problems resulted from the improper intersection layout design and the improper signal program design. The control strategy is always being used as a fixed-time signal program, and the traffic flow quality has not been evaluated yet.

2.2.1. Problems of safety

- **Problems of motorcycles and cars (motorised traffic)**

  According to the general traffic situations in MDCs presented in section 2.1.4, mixed motorised traffic causes the following problems at traffic signals.

  - ***Problems when approaching the traffic signal systems***

    When vehicles are approaching the traffic signal systems during the amber time, motorcycles and cars sometimes hit each other due to the improper amber time design because the demands on the amber time of these two vehicular types are different (see Figure 9).

    When vehicles are approaching the traffic signal systems during the green time, collisions between motorcycles and cars on the same approach occur due to turning vehicles (see Figure 10). In this figure, the car could not turn right during the green signal due to motorcycles surrounding. Nevertheless, if motorcycles and cars in the same direction are approaching the intersection during the green time, there are no collisions between them on the approach.

When vehicles are approaching the traffic signal systems during the red signal, both cars and motorcycles usually decelerate, but then motorcycle riders try to get in front of cars when both speeds of cars and motorcycles are low.

Figure 9: Accident during the amber time
(http://vietnamnet.vn, 2006)

Figure 10: Collisions between MC and Cars
(Do, 2005)

- ***Problems during waiting at the red signal***

According to the observations, while cars were waiting at red signals, the motorcycle riders tried to get in front of cars as long as they found spaces ahead. This has an advantage that it saves the road spaces as well as the queue length on the approach, but it also has a disadvantage that motorcycles and cars on the same lane still have collisions when the green signal starts under the mixed traffic condition (see Figure 11).

Figure 11: Waiting during the red signals
(Pham, 2007)

- ***Problems during the green signal***

  When the green time begins, the previous waiting vehicles start moving. Under the mixed traffic conditions and a higher acceleration rate, motorcycles manoeuvre flexibly and try to overcome cars on the approach as well as in the inner intersection area. Therefore, collisions between them occur, these make the capacity of the intersection impaired. In addition, there are a lot of conflicts between vehicles (including motorcycles and cars) occurring in the inner intersection areas in case of permitted traffic flows, and the capacity is, therefore, impaired much (see Figure 12).

Figure 12: Conflicts during the green time in the inner intersection area

(Do, 2005)

- **Problems of public transport**

  In almost all MDCs, there are no trams on urban roads. Therefore, public transport focuses only on buses and plays an important role in MDCs. Nevertheless, in these cities, there is no priority to buses at traffic signals. Besides having the same problems of motorised traffic that were presented above, buses have other problems as follows:

  + Buses have to stop at the bus-stops for boarding and alighting passengers, this traffic situation creates many critical collisions with motorcycles, especially when the bus-stop is located near the intersections (see Figure 13). In this traffic situation, while the bus is changing lane and stopping at the bus-stop, motorcycle riders are in a quite difficult traffic situation. They are enforced toward the sidewalk and have to make a decision either to overcome the bus or to decelerate to get behind it. However, both decisions are unsafe. In addition, the bus driver is very stressful while many motorcycles are surrounding the bus. Therefore, high risk of accidents between buses and motorcycles occurs.

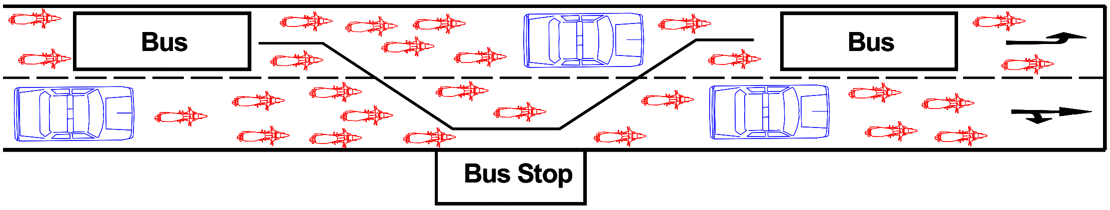


Figure 13: Collisions between buses and other vehicles

+ Because of large in size, the bus drivers have difficulties in paying attentions to many motorcycles surrounding, especially in the inner intersection areas (see Figure 14 and Figure 15).

Figure 14: Buses in inner intersection areas

(http://www.tienphongonline.com.vn, 2007)

Figure 15: Buses in mixed traffic on road

(http://www.tienphongonline.com.vn, 2006)

- **Problems of cyclists**

Because of lower speed than motorcycles, one right-hand lateral lane on the carriageway is usually reserved for cyclists. Otherwise, they will share lanes with motorised traffic. Cyclists, therefore, always join signalisation with motorised traffic in MDCs. Problems for left turning cyclists occur very often because they have many conflicts with motorcycles on the same approach as well as with motorcycles of the permitted traffic flows during the green time (see Figure 16 and Figure 17).

Figure 16: Cyclists share lanes with motorised traffic

(Do, 2005)

Figure 17: Exclusive cycle lane

(Do, 2005)

- **Problems of pedestrians**

  In MDCs, there are some problems for pedestrians at traffic signals. Firstly, there is no priority for pedestrians in the permitted phases even the time lead to confirm their priority to vehicles; therefore, they have a high risk of accident when crossing the intersections because the motorised drivers do not pay attentions on pedestrians (see Figure 18). Secondly, there is no demand phase for pedestrians; the waiting time, therefore, is usually very long. Thirdly, in some cases, the green time for pedestrians is too short disregarding the intersection geometry; therefore, they have to confront with the vehicles of the next phase coming up.

Figure 18: High accident risk for pedestrians at traffic signals

(http://vietnamnet.vn, 2007)

2.2.2. Problems of intersection layout

In almost all MDCs, signalised intersection layouts are being designed and operated for car traffic, they almost disregard the presence of motorcycle traffic. Even in some countries, they do not have any guideline or standard for at-grade intersection design. Consequently, intersections are designed very simply and poorly. In conurbation, many intersections were installed with traffic signal systems, but the intersection layout and signal-timing program were not an entity, they were separated. As a result, traffic signal systems are being operated ineffectively, for example the intergreen time is too long, locations of pedestrian crossing are incorrect, continuous lanes, left-turning lanes, and right-turning lanes are not designed reasonably. Figure 19 shows an unreasonable signalised intersection in Hanoi.

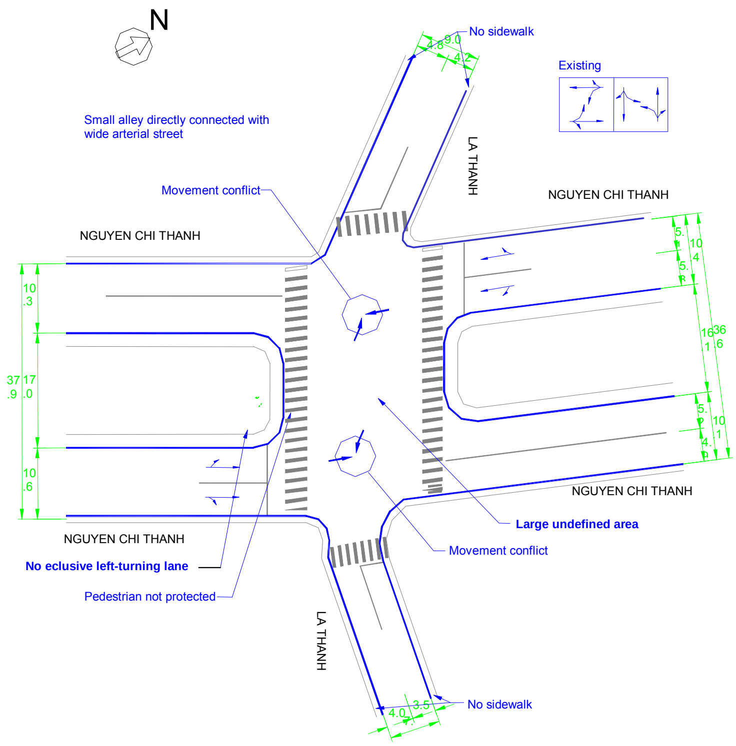


Figure 19: An unreasonable intersection in Hanoi

(Do, 2005)

Because the traffic signal system and the intersection layout have some impacts on each other, the intersection layout must correspond to the signal timing program and conversely. In the more particular case of MDCs, it also needs some modifications of the intersection layouts and the signal timing programs that can take motorcycles into account.

As an example, Taiwan is one of the countries where there are many studies on motorcycle traffic. Su, Wu, and Hung (2006) presented a two-stage left turning regulation for motorcycles at major multi-lane signalised intersections as shown in Figure 20.

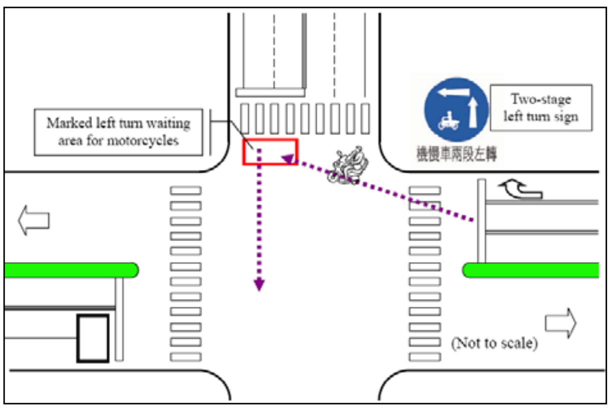


Figure 20: Left turning regulation for MCs in Taiwan
(Su, Wu, Hung, 2006)

Figure 21: Mixed traffic in Taiwan
(Hsu, 2003)

Figure 21 is a minor signalised intersection in Taiwan. It shows mixed traffic dominated by motorcycles on the approach similarly in Hanoi.

One of the limitations of the two-stage left turning regulation is that it cannot apply to the case of a high proportion of motorcycles in the traffic flows because, on the one hand, the inner intersection areas are insufficient for motorcycles waiting; on the other hand, it is very hard for the motorcycle riders to turn left on the small waiting area (see the waiting place for motorcycles in Figure 20). Therefore, this type of intersection layout is only suitable when the motorcycle proportion is very low as at some major intersections in Taiwan.

2.2.3. Problems of signal program

As already mentioned above, signal programs in MDCs are now being designed and operated as fixed-time signal programs due to lack of knowledge as well as technology in applying traffic-actuated signal programs. Nevertheless, fixed-time signal programs themselves in MDCs have also many problems due to lack of guidelines for traffic signals.

For example, in Hanoi, many intersections have only one cycle time with two phases controlling for all day, it is unreasonable when traffic loads are changing every period (peak period, inter-peak period, and off-peak period). More dangerously, according to Tran, D.T (in an interview of VnExpress newspaper, 2006), the director of the Hanoi Control Centre, signal timing programs at 10 intersections using controllers from inland did not take care about pedestrians. Green time and clearance time for pedestrians were fixed disregarding the geometrical elements of intersections. Therefore, motorised vehicles had usually come up the conflict areas before the pedestrians cleared the crossing. Furthermore, there isn't any protected phase for pedestrians at signalised intersections even the number of pedestrians is high. The permitted phases are always used in which the signal programs do not provide pedestrians the time lead to confirm their right of way. As a result, pedestrians are surrounded by motorcycles and have to face a high risk of accident when they cross the approach.

In India, some intersections are operating with a too long cycle time (even reaching 300 hundred seconds), and the intergreen time is also fixed disregarding the intersection geometry.

In MDCs, it seems that designers are only interested in the equivalent factor for converting motorcycles into passenger car unit (PCU). Then, they apply completely the guidelines for traffic signals of PCU traffic. However, these are insufficient for improving traffic safety and traffic flow quality because the actual traffic situations occurring at intersections are far different from that of passenger car traffic. For example, the equivalent factor does not represent traffic safety of motorcycles as well as conflicts between motorcycles and other types of vehicles. Some signal program elements, such as the amber time, the intergreen time, the cycle time, the green time, etc. for passenger car units are not suitable for motorcycle traffic. For example, the deceleration rate of car and motorcycle is different; the amber time, therefore, must be determined carefully and safely for both motorcycles and cars. The intergreen time must take the speed and dimensions of motorcycle into account. The cycle time must be determined with the proper saturation flow rates, etc. And the signal programs have to provide priorities to public transport as well as to pedestrians.

2.3. Strategies to solve problems at traffic signals in MDCs

2.3.1. Definition of strategies in traffic management

Strategies in traffic management were discussed in ‘’Hinweise zur Strategieentwicklung in dynamischen Verkehrsmanagement’’ (FGSV, 2003) in which the terms: scenario, strategies, problems, situations, measures, systems, must be distinguished.

According to FGSV (2003), the dynamic traffic management must always respond to various traffic situations. The aim for forming the strategies is to develop the defined concepts for the occurring situations in which the most appropriate measures from various measure categories are chosen and applied.

The traffic situation is defined as a combination of the problem, the incident, and the state of traffic. It means that when a problem is identified, the incident and the state of traffic accompanied will form the traffic situation.

The traffic scenario is defined as the combination of a situation and a strategy.

All the terms above can be illustrated by the following figure:

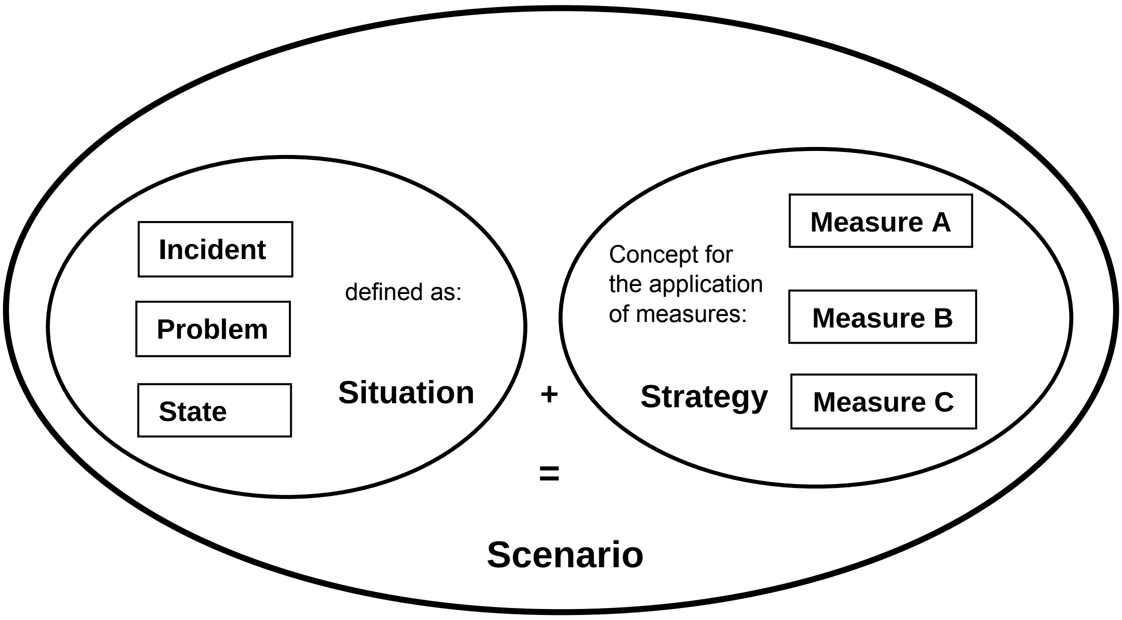


Figure 22: Situation, Strategy, and Scenario

(FGSV, 2003)

According to Boltze (2006), the measures are developed to eliminate or to avoid deficiencies in the transport systems. They can be classified into some categories as follows:

- Measures for public transport
- Measures for motorised traffic
- Inter-modal measures
- Multi-modal measures
- Other measures, for example: measures for pedestrian and bicycle traffic

  (Boltze, 2006)

To express which measures are suitable to which problems, normally the problem categories and the measure categories are presented in form of a matrix table. Similarly, the measure categories and the systems are also presented by the same way.

2.3.2. Strategies to solve problems at traffic signals in MDCs

The strategies to solve general problems in MDCs have already introduced by Khuat (2006). In this study, only strategies to solve problems at traffic signals in MDCs are mentioned; therefore, all measure categories use only one type of system, which is the traffic signal system.

From the definition of strategy above and the problems at traffic signals analysed in chapter 2, the problem categories and the measure categories in MDCs are presented in the following table:

Table 7: Strategies for solving problems at traffic signals in MDCs

Measure categories	Problems categories	Traffic safety	Road traffic engineering design	Signal program
Public Transport	Public transport routing improvement	x	x	-
	Public transport scheduling improvement	o	-	x
	Giving priority to public transport	o	x	x
	Exclusive signals for public transport	x	x	x
	Separating public transport	x	x	x
	Public transport information services	o	o	-
Car	Separating car traffic	x	x	o
	Right turning and go straight cars share lanes	o	x	o
	Displace the stop-line of cars backward	x	x	o
	Applying detectors for car traffic	x	o	x
Motorcycle	Giving motorcycles an opportunity to get in front of cars during the red time	x	x	x
	Separating motorcycle traffic on roads	x	x	-
	Exclusive signals for motorcycles if necessary	x	x	x
	Temporarily protected turning movements for motorcycles	x	x	x
Pedestrian	Protected movements for pedestrians	x	o	x
	Demand phases by sensor detections	x		x
	Applying amber flashing with a pedestrian symbol to warn other vehicles	x	-	x
Bicycle	Joint signalisation with either pedestrian or motorised traffic	o	x	x
	Two left turning stages for bicycles	x	x	x

x: fully applicable; o: partly applicable; -: not relevant

2.4. Conclusions

From major problems above, MDCs are facing a great challenge for urban traffic, and they need a set of measures to solve these problems. Regarding the traffic signal control measure, it is necessary to establish the guidelines for traffic signals that are suitable for the specific traffic conditions dominated by motorcycles. These guidelines must base on the following principles:

- Traffic signal systems must be easy in using for all road users,
- Traffic signal systems must be reasonable in economy and technology,
- The traffic safety criterion is the most important one for MDCs, the second important one is traffic flow quality, and the third important one is capacity of the intersection.

Within the framework of this study, these guidelines will be developed based on the "German Guidelines for Traffic Signals". The applicability of these German Guidelines for Traffic Signals to MDCs will be analysed in the next chapter.

3. Applicability of RiLSA

3.1. General remarks

These guidelines are going to be analysed in detail to find out:

i. which parts of the text are **not relevant**,

ii. which parts of the text are **relevant, but *no modification*** needed,

iii. which parts of the text are **relevant and *minor modifications*** needed and

iv. which parts of the text are **relevant and *major modifications*** needed for the MDCs.

The next chapters of this study will focus on researching on major modifications. Minor modifications will be directly written in the draft of "Guidelines for Traffic Signals in MDCs" that will be presented in chapter 7 of this study. Consequently, what is not relevant to MDCs will not be presented in chapter 7.

In order to achieve the aims above, individual chapters of RiLSA edition 2009 have to be analysed to answer the question how far they could be transferred to MDCs.

3.2. Analysing the applicability of RiLSA edition 2009

3.2.1. Chapter 1: Basic Principles

The principles of traffic signals described in RiLSA are basically valid for MDC conditions, as well. Only in some parts of the RiLSA text, minor modifications are needed (see Table 8)

Table 8: Applicability of RiLSA to MDCs – Chapter 1: Basic Principles

Chapter and sub-chapter	Contents of RiLSA, 2009	Application for MDCs			
		(i) not relevant	(ii) relevant and no modifications needed	(iii) relevant and minor modifications needed	(iii) relevant and major modifications needed
1	**Basic principles**				
1.1	***General remarks***			x	
1.2	***Criteria for the use of traffic signal and the affects to be achieved***				
1.2.1	Traffic safety			x	
1.2.2	Traffic flow quality			x	
1.2.3	Fuel consumption and emissions		x		
1.2.4	Balancing of conflicting objectives		x		
1.3	***Traffic signals and signal sequences***			x	

Section 1.1: General remarks, is basically kept the same contents as in RiLSA because it discussed about the basic concepts of traffic signals that are very necessary for MDCs.

Section 1.2: Criteria for the use of traffic signal and the affects to be achieved discusses about the criteria for the use of traffic signals. It can be summarised in Table 9 below:

Table 9: Criteria for the use of traffic signals

Criteria	Concerned Problems
Traffic Safety	Accident density and severe accidents Visibility on approaches Protection of pedestrians and cyclists
Traffic Flow Quality	Traffic volume (major- and minor directions) Traffic process for pedestrians and cyclists Traffic process for public transport Requirements of emergency vehicles Guides for cars in the road network Prevent sub-network from congestion
Environmental Impacts	Noise Emissions Exhaust Emissions
Economic	Waiting time, Travel time Fuel consumption Worn-out vehicles

(Boltze, 2007)

Section 1.2.1: Traffic safety, is basically kept the same contents as in RiLSA if we consider motorcycles and cars to be motorised traffic. However, in the traffic safety point of view, it is necessary to reduce collisions between motorcycles and cars in motorised traffic. In addition, one paragraph mentioning chapter 8 “Quality Management” will be removed because traffic in MDCs has not reached the high level as in developed countries, especially Germany.

Section 1.2.2: Traffic flow quality is basically kept the same contents as in RiLSA, but traffic process for motorcycles at traffic signals should be added. Furthermore, criteria to estimate traffic flow quality in RiLSA 2009 were shifted to HBS 2001. Therefore, these contents should be presented in the annex of the “Guidelines for Traffic Signals in MDCs” in which the following criteria would be calculated:

Table 10: Criteria to estimate traffic flow quality

Vehicle	Criteria
Car and motorcycle (motorised traffic)	Average waiting time Number of stops or through-drives Number of motorised traffic in congestion Saturation degree Number of cycle times congested
Public transport	Average waiting time Number of stops
Pedestrian and Cyclist	Average waiting time Maximum waiting time

(Boltze, 2007)

Section 1.2.3: Fuel consumption and emission will be kept the same texts as in RiLSA, and there will be no more detail discussions about this topic in this study. They should be researched in detail in a separate thesis.

Section 1.2.4: Balancing of conflicting objectives is kept the same texts as in RiLSA.

Section 1.3: Traffic signals and signal sequences needs some minor modifications. For example, there is no Red/Amber signal in MDCs; exclusive signals for motorcycles need to be discussed, and signals for public transport in MDCs also need to be discussed whether they can be applied from Germany or not. Countdown signals should be also mentioned in MDCs. All these minor changes will be directly written in the draft of Guidelines for Traffic Signals in MDCs.

3.2.2. Chapter 2: Signal Program Design

The whole chapter 2: Signal Program Design in RiLSA 2009 is relevant to MDCs. However, in order to apply to MDCs, some parts do not need any modification, some parts need minor modifications, and some other parts need major modifications, as described in Table 11.

Table 11: Applicability of RiLSA to MDCs – Chapter 2: Signal Program Design

Chapter and sub-chapter	Contents of RiLSA, 2009	Application for MDCs			
		(i) not relevant	(ii) relevant and no modifications needed	(iii) relevant and minor modifications needed	(iii) relevant and major modifications needed
2	**Signal program design**				
2.1	***Terms and definitions***		x		
2.2	***Documents and pre-studies***			x	
2.3	***Signal program structure***				
2.3.1	Signal phasing				
2.3.1.1	General remarks			x	
2.3.1.2	Left-turning movements			x	
2.3.1.3	Right-turning movements			x	
2.3.1.4	Trams and buses				x
2.3.1.5	Pedestrian traffic		x		
2.3.1.6	Cycle traffic			x	
2.3.2	Number of phases		x		
2.3.3	Phase sequence		x		
2.3.4	Phase transitions		x		
2.4	***Transition times***				x
2.5	***Intergreen times***				x
2.5.1	Determination of clearing and entering distances				x
2.5.2	Crossing and clearance time				x
2.5.3	Entering time				x
2.5.4	Checking intergreen time		x		
2.6	***Cycle time***				x
2.7	***Green time and red time***				x
2.7.1	Green time calculation				x
2.7.2	Return to the same phase		x		
2.7.3	Maximum and minimum red time			x	
2.7.4	Minimum green times				x
2.7.5	Time lead at the conflict area		x		
2.7.6	Delayed green time beginning		x		
2.8	***Signal timing plan***			x	

Section 2.1: Terms and definitions should be kept the same texts as in RiLSA.

Section 2.2: Documents and pre-studies should have following minor modifications:

- ***The method for collecting traffic data*** in RiLSA 2009 was shifted to chapter 2 and chapter 6 of HBS 2001. Therefore, these contents should be transferred to the annex of the “Guidelines for Traffic Signals in MDCs”.
- ***The results of accident studies*** in RiLSA 2009 were shifted to the “Merkblatt für die Auswertung von Straßenverkehrsunfällen“. However, in MDCs, it is enough to take the results of accident studies from RiLSA edition 1992.
- If necessary, ***figure 2.1: Example of a signal layout plan*** in RiLSA 2009 can be replaced by a typical intersection layout in MDCs, or can be removed.

Section 2.3.1.1: General remarks mentioned the priority rules according to §9 Abs.3 and 4 in StVO. These priority rules at traffic signals should be directly written in the draft of “Guidelines for Traffic Signals in MDCs”. And the text “sign 209 StVO” should be replaced by “turning enforcement signs”.

Section 2.3.1.2: Left-turning movements should be added with the content of “leading green for left-turning motorcycles”. Because if the waiting areas for motorcycles are designed in front of the waiting areas for cars, using “leading green time for motorcycles” will reduce the high risk of accident for crossing pedestrians due to the lower number of critical conflicts between them and motorcycles.

Section 2.3.1.3: Right-turning movements should have the following minor modifications:

- It is recommended to use “leading green for right-turning motorcycles” to reduce the number of conflicts between motorcycles and pedestrians.
- In case ***right-turners are routed by triangular islands or by exclusive lanes***, it is not necessary to separate motorcycles from car traffic because right-turning traffic flows are not as critical as left-turning or go-though traffic flows.

Section 2.3.1.4: Trams and buses phasing was discussed in RiLSA 2009 with the exclusive signals and lanes for public transport. However, in almost all MDCs, public transport focuses on buses only, and there are not so many roads that reserved exclusive lanes for buses (only the ring roads and the arterials). Therefore, economic and technology aspects as well as compatibility of road infrastructure must be considered when applying the exclusive signals for public transport from Germany. In this study, it is recommended to apply the exclusive signals for buses in MDCs if the separate lanes for buses are available.

Section 2.3.1.5: Pedestrian traffic is kept the same contents as in RiLSA except the content of separate railway crossing because this traffic situation does not exist in MDCs.

Section 2.3.1.6: Cycle traffic needs some minor modifications because some traffic situations of cycle traffic in RiLSA 2009 do not exist in MDCs, for examples: there is no cycle path on the sidewalk; separate signalisation for cyclists becomes much more difficult in MDCs because of considerations for motorcycles. If cyclists join signalisation with pedestrians, one more possibility is designing a transitional segment from carriageway upward to the sidewalk and conversely (this will be discussed in the section of intersection layout design) so that cyclists riding on the carriageway can join the waiting area with pedestrians. And the phrase "joint signalisation with motorised traffic" should be changed by another one: "joint signalisation with motorcycles".

Section 2.3.2: Number of phases; 2.3.3: Phase sequence; 2.3.4: Phase transitions do not need any modifications because they are general basic principles.

Section 2.4: Transition time needs a major modification. In MDCs, the transition time focuses on the amber time only because the signal Red/Amber does not exist. In the modification, not only cars but also motorcycles have to be taken into account.

Section 2.5: Intergreen time also needs a major modification. Basically, the method to determine the intergreen time is kept the same as in Germany. However, motorcycle traffic must be taken into account. Thus, some values for determining the intergreen time from Germany should be changed in order to be suitable for MDCs. Furthermore, case 3 for determining the intergreen time in Germany (trams are clearing without stop before the intersection) does not exist in MDCs. Case 4 (public transport vehicles with stop before the intersection) should consider buses only (without trams).

Section 2.6: Cycle time needs a major modification because all the formulas for determining the cycle time all over the world, especially in RiLSA, are applied to car traffic. Therefore, if traffic is dominated by motorcycles, the cycle time should be adjusted in which saturation flow rate plays an important role. In this study, two methods for determining the cycle time in RiLSA 2009 will be analysed and adjusted for MDCs.

Section 2.7.1: Green time calculation also needs a major modification because it is calculated according to the cycle time and the saturation flow.

Section 2.7.2: Return to the same phase, and ***section 2.7.3: Minimum red time*** does not need any modifications.

Section 2.7.3: Maximum and Minimum red time needs a minor modification. For example, maximum red time depends on the acceptance of motorcycle riders. And the values of waiting time in RiLSA 2009 was shifted to HBS, this content should be transferred to the annex of the draft of "Guidelines for Traffic Signals in MDCs".

Section 2.7.4: Minimum green time needs a major modification to find out the suitable values for MDCs.

Section 2.7.5: Time lead to the conflict area, and ***section 2.7.6: Delayed green time beginning*** do not need any modifications.

Section 2.8: Signal timing planning does not need any modification.

3.2.3. Chapter 3: Inter-relations between Traffic Signal Control and Road Design

In order to give priority to motorcycles at traffic signals, the intersection layout has to be designed so that motorcycle riders have an opportunity to stop and wait in front of cars during the red time. Therefore, most parts of RiLSA need to be modified as described in Table 12 below:

Table 12: Applicability of RiLSA to MDCs – Chapter 3: Inter-relations between Traffic Signal Control and Road Design

Chapter and sub-chapter	Contents of RiLSA, 2009	Application for MDCs			
		(i) not relevant	(ii) relevant and no modifications needed	(iii) relevant and minor modifications needed	(iii) relevant and major modifications needed
3	**Interrelationships between traffic signal control and intersection layout**				x
3.1	***General remarks***		x		
3.2	***Lanes***				x
3.2.1	Continuous lanes			x	
3.2.2	Left-turning lanes				x
3.2.3	Right-turning lanes and right-turning Carriage-ways				x
3.2.4	Partial public transport lanes on the approaches				x
3.2.5	U-turn lanes			x	
3.3	***Guidance for cycle traffic***				x
3.4	***Central reservations and separating strips***		x		
3.5	***Tram crossings***	x			
3.6	***Cycle and pedestrian crossings***		x		
3.7	***Bus stops***			x	
3.8	***Facilities***				
3.8.1	Stop-lines				x
3.8.2	Route marking lines			x	
3.8.3	Signing			x	

Section 3.1: General remarks should be kept the same contents as in RiLSA and do not need any modification.

Section 3.2: Lanes needs the following major and minor modifications:

- ***Section: Head-start lanes for motorcycles*** should be added. This section will present how to design the head-start lanes for motorcycles in order to provide motorcycle riders an opportunity to get in front of cars during the waiting time (major modification).
- ***Section 3.2.1: Continuous lanes*** is basically kept as in RiLSA. However, some contents discussing about the utilizations of railway track area for motorised traffic, and some contents related to public transport on continuous lanes should be removed. Therefore, this section needs minor modifications.
- ***Section 3.2.2: Left-turning lanes*** needs a major modification. The calculation of the left-turning lane's length in RiLSA was shifted to HBS. This content will be transferred to the annex of the draft of "Guidelines for Traffic Signals in MDCs" with considerations of the lane length for head-start motorcycles. In case the approach width is less than 5.5 m, the

left-turning lane should not be separated, but the solution for these widths will be presented in chapter 4. Left-turning lanes related to public transport should be removed because of the head-start lanes for motorcycles.

- ***Section 3.2.3: Right-turning lanes and right-turning carriageway*** needs a major modification. Similarly, the calculation of right-turning lane's length in RiLSA was shifted to HBS. This content will be transferred to the annex of the draft of "Guidelines for Traffic Signals in MDCs". Unlike left-turning lanes and go-through lanes, it is not necessary to establish the head-start lanes for motorcycles on the exclusive right-turning lanes or right-turning carriageways because the right-turning traffic streams do not have as many critical conflicts as left-turning lanes and go-through lanes do. The detailed layout design will be presented in chapter 4.
- ***Section 3.2.4: Partial public transport lanes*** must be removed because the head-start lanes for motorcycles have been already considered, and there is no more road space to design the partial public transport lanes on the approach. Instead, other solutions of the layout for public transport are addressed in chapter 4.
- ***Section 3.2.5: U-turn lanes*** is basically kept the same contents as in RiLSA. However, some contents related to the separate railways will be removed.

Section 3.3: Guidance for cycle traffic is basically kept the same contents as in RiLSA. However, it needs a minor modification in which cyclists can join signalisation with motorcycles in the head-start lanes.

Section 3.4: Central reservations and separating strips, and ***section 3.6: Cycle and pedestrian crossing*** should be kept the same content as in RiLSA 2009.

Section 3.5: Crossing stations at tramways will be removed because this traffic situation does not exist in MDCs.

Section 3.7: Bus stops needs a minor modification. For example, the contents of "Bus Sluice", Stop Island, and Dynamic Island will be removed because "motorcycle sluice" has already been considered, and "Bus Sluice", therefore, could not be implemented due to lack of road space. Stop Island and Dynamic Island do not exist in MDCs.

Section 3.8.1: Stop-lines needs a major modification in which the stop-lines for motorcycles have to be properly designed. And the dimensions of the stop-lines should be followed by the design standard of each country.

Section 3.8.2: Route marking lines needs a minor modification in which the head-start areas for motorcycles should be marked by dark-red colour. And the route marking for motorcycles should be added.

Section 3.8.3: Signing needs a minor modification in which the code number of traffic signs should be changed depending on the signing standard of each country.

3.2.4. Chapter 4: Control Strategies

The control strategies described in chapter 4 of RiLSA are basically applicable to MDCs. Only some parts need minor modifications as shown in Table 13.

When applying the German control strategies to MDCs, the technology and the cost should be considered.

RiLSA edition 2009 introduced three macroscopic control levels and three microscopic control levels. The macroscopic levels consider long-term changes of traffic loads, while the microscopic levels are activated from the macroscopic levels and consider short-term periods only (within seconds or a cycle time).

Three macroscopic control levels are time-dependent signal program selection (A1), traffic-dependent signal program selection (A2), and traffic-dependent signal program formation (A3). However, the last two control levels (A2 and A3) are very difficult to apply to MDCs due to the online-requirement on traffic parameters in the network. For example, it will be very complicated and costly to collect traffic volume, especially for motorcycles in the road network. Nevertheless, the first macroscopic control level (A1) can be applied to MDCs because the high-load periods can be off-line forecast and stable during the course of the day or week, especially in peak hours or off-peak hours.

Regarding the microscopic control levels, RiLSA 2009 introduced three control strategies, the first one is fixed-time signal program (B1), the second one is signal program adaption including green time adjustment (B2), phase swapping (B3), demand phase (B4), and time-offset adjustment (B5), the third one is free modifications possible (B6). The first one is always easy to apply to MDCs, especially in case the traffic load is stable for a long period of time because there is no any change of the signal program elements, and it does not require online traffic collection. The second one (B2, B3, B4, B5) can be, in principle, applied to MDCs because they only require detecting vehicles (not counting vehicles) for control parameters. Note that, in practice, the conventional detectors can detect motorcycles, but cannot count the number of motorcycles because more than one motorcycle can be detected by the same detector at the same time. The biggest problem for applying detectors to MDCs is how to increase the inductance of motorcycles, and how to guide motorcycles driving correctly on their assigned lanes because their flexible manoeuvres can make the detectors disturbed. For examples, left-turning motorcycles can easily invade go-through lanes on approaches; downstream motorcycles can invade departure lanes of opposing directions, etc. The third one (B6) cannot be applied to MDCs because it requires too much effort in collecting traffic data online.

Going back to detailed analyses, the individual sections are analysed and estimated as follows:

Section 4.1: Overview on the control strategies should be remained in order the readers to be able to imagine the overview on the control strategies. But at the end, one paragraph asserts that the macroscopic level (A1) and the microscopic levels (B1, B2, B3, B4, and B5) could be applied to MDCs, should be added.

Section 4.2.1: Combination of parameters should be remained as in RiLSA 2009.

Table 13: Applicability of RiLSA to MDCs – Chapter 4: Control Strategies

Chapter and sub-chapter	Contents of RiLSA, 2009	Application for MDCs			
		(i) not relevant	(ii) relevant and no modifications needed	(iii) relevant and minor modifications needed	(iii) relevant and major modifications needed
4	**Control Strategies**				
4.1	***Overview on the control strategies***			x	
4.2	***Control parameters***				
4.2.1	Combination of parameters		x		
4.2.2	Collecting and processing parameters				
4.2.2.1	Traffic-actuated selection of signal programs	x			
4.2.2.2	Green time request			x	
4.2.2.3	Time headways			x	
4.2.2.4	Degree of occupancy measurement	x			
4.2.2.5	Congestion and queue length measurement		x		
4.3	***Detail on the control strategies***				
4.3.1	Selection of signal programs				
4.3.1.1	Boundary conditions		x		
4.3.1.2	Time-dependent selection of signal programs		x		
4.3.1.3	Traffic-actuated selection of signal programs	x			
4.3.2	Formations of signal programs		x		
4.3.3	Fixed-time signal programs		x		
4.3.4	Signal program adaption				
4.3.4.1	Green time adjustment			x	
4.3.4.2	Phase swapping		x		
4.3.4.3	Demand phases		x		
4.3.4.4.	Time-offset adjustment		x		
4.3.5	Signal program formation		x		
4.4	***Co-ordination***				
4.4.1	Goals and objectives		x		
4.4.2	Basic principles			x	
4.4.3	Co-ordination at intersections		x		
4.4.4	Co-ordination on arterials		x		
4.4.4.1	Constructional pre-conditions		x		
4.4.4.2	Traffic engineering boundary conditions		x		
4.4.4.3	Consideration of public transport		x		
4.4.4.4	Consideration of cycle traffic		x		
4.4.5	Co-ordination in road network		x		
4.5	***Designing a control project***				
4.5.1	Rule-based implementation of control strategies			x	
4.5.2	Standard rule-based implementation of control strategies		x		
4.5.3	Model-based implementation of control strategies	x			
4.5.4	Switching procedures		x		
4.5.4.1	General remarks		x		
4.5.4.2	Switching point		x		
4.5.4.3	Switching by compression and extension measures		x		
4.5.4.4	Switching without any defined switching point		x		
4.5.5	Test of the control		x		

Section 4.2.2.1: Traffic-actuated selection of signal programs discusses about an acquisition of current traffic parameters in the network. This is very complex and impossible to apply to MDCs as discussed above. Therefore, this section should be removed.

Section 4.2.2.2: Green time request is basically kept the same as in RiLSA 2009. However, the location of detectors for vehicles should be located in front of the stop-line of the head-start lanes for motorcycles (usually 1.0 m ÷ 1.5 m from the beginning of detectors).

Section 4.2.2.3: Time headways is remained as in RiLSA 2009. However, the distance between the detector and the stop-line of cars is referred for calculation.

Section 4.2.2.4: Degree of occupancy measurement cannot be applied to MDCs because it relates to traffic counting. It means that the degree of occupancy does not reflect traffic volume in case of motorcycles. The traffic volume can differ from each other even with the same degree of occupancy because the detector cannot detect the number of motorcycles. Therefore, it is unreliable to set up threshold value for the green time abortion according to the degree of occupancy.

Section 4.2.2.5: Congestion and queue length measurement can be completely applied to MDCs without any modification.

Section 4.3.1.1: Boundary conditions and ***section 4.3.1.2: Time-dependent selection of signal programs*** should be remained as in RiLSA 2009. However, ***section 4.3.1.3: Traffic-actuated selection of signal programs*** cannot be applied to MDCs as analysed above.

Section 4.3.2: Formations of signal program, and ***section 4.3.5: Signal program formation*** should be removed as analysed above.

Section 4.3.3: Fixed-time signal programs and ***section 4.3.4: Signal program adaption*** are remained as in RiLSA 2009. Only one content in ***section 4.3.4*** is removed, which is the green time adjustment by the degree of occupancy.

Section 4.4.1: Goals and objectives for coordination is remained as in RiLSA 2009 and does not need any modification.

Section 4.4.2: Basic principles needs a minor modification. Firstly, basically it is not necessary to create the head-start lanes for motorcycles at coordinated intersections. But, the coordination could not ensure 100% traffic volume passing intersections; therefore the head-start lanes for motorcycles should be designed by minimum dimensions. Secondly, traffic volume reflecting the width of the green band should be represented by either MCU (motorcycle unit) or PCU (passenger car unit); therefore motorcycles need to be converted into PCU and conversely.

Section 4.4.3: Coordination at intersections does not need any modification.

Section 4.4.4: Coordination on arterials does not need any modification. However, the technology for considerations of public transport needs to be considered.

Section 4.4.5: Coordination in the road network should be remained and does not need any modification.

Section 4.5.1: Rule-based implementation of control strategies is basically kept the same as in RiLSA.

Section 4.5.2: Standard rule-based implementation of control strategies should be remained and does not need any modification.

Section 4.5.3: Model-based implementation of control strategies cannot be applied to MDCs because it requires not only traffic models but also control models. Simulating online traffic models is impossible in MDCs. Therefore, section 4.5.3 should be removed.

Section 4.5.4: Switching procedures discussed about the procedures to switch alternative signal programs, and this section, therefore, does not need any modification.

3.2.5. Chapter 5: Special Forms of Signalisation

Basically, some parts of ***chapter 5: Special forms of signalisation*** such as Ram metering control, Lane signalisation are difficult to apply to MDCs. For example, motorcycle riders will not obey the ***Lane signalisation*** because of their flexible manoeuvres, and the ***Ramp metering control*** uses a regulation of "one green for one vehicle" at the access ramp, this is impossible for motorcycles. On the other hand, ***Partial signalisation*** and ***Bottleneck signalisation*** are not usually used in the urban areas, where there is usually high traffic load, especially in MDCs. Therefore, chapter 5 is not included in this study. However, in the future, when the number of motorcycle traffic in MDCs reduces, these forms of signalisation should be considered.

3.2.6. Chapter 6: Technical Design

In general, each country has its own technical design of traffic signals such as styles of the signal head, dimensions and styles of signal posts, etc. However, this study proposes the technical design from Germany for MDCs. Therefore, chapter 6 is merely a translation into English.

Table 14: Applicability of RiLSA to MDCs – Chapter 6: Technical Design

Chapter and sub-chapter	Contents of RiLSA, 2009	Application for MDCs			
		(i) not relevant	(ii) relevant and no modifications needed	(iii) relevant and minor modifications needed	(iii) relevant and major modifications needed
6	**Technical design**				
6.1	***Control devices***		x		
6.2	***Signal lamp***				
6.2.1	Light regulations		x		
6.2.2	Visibility of the signals		x		
6.2.3	Phantom light		x		
6.2.4	Size of the optical units		x		
6.2.5	Operating voltage		x		
6.2.6	Vehicle signal heads		x		
6.2.7	Pedestrian signal heads		x		
6.2.8	Bicycle signal heads		x		
6.2.9	Signal heads for public transport		x		
6.2.10	Auxiliary signal heads		x		
6.2.11	Lane signal heads		x		
6.2.12	Speed signal heads		x		
6.2.13	Uniform design for symbols in optical units		x		
6.2.14	Hoods at signal heads		x		
6.2.15	Backing boards for signal heads		x		
6.2.16	Acoustic and tactile for visual handicapped		x		
6.3	***Monitoring equipments***		x		
6.4	***Number and position of signal heads***				
6.4.1	Intersection layout		x		
6.4.2	Lane signalization		x		
6.5	***Construction***		x		

3.2.7. Chapter 7: Technical Acceptance and Operation

This chapter is not included in this study because it does not address the traffic engineering point of view directly. In addition, each city has its own technical acceptance and operation. For a later version of such guidelines for MDCs, this topic might be considered.

3.2.8. Chapter 8: Quality Management

Quality management for traffic signals is a new topic in Germany, and it needs a synchronisation concerning many other fields. This topic was conducted by Reußwig (2005). However, this topic is regarded as being of second priority in case of MDCs. Therefore, this chapter is not included in this study. However, in the future, this chapter should be considered.

3.2.9. Chapter 9: Instructions and Technical Regulations

This chapter introduced some other official German documents related to RiLSA edition 2009. Of course, this chapter is not included in the “Guidelines for Traffic Signals in MDCs” because the necessary contents in some other documents such as HBS, StVO, RAS-K-1, etc. are shifted to the annexes of the Guidelines for Traffic Signals in MDCs.

3.3. Conclusions

As mentioned above, RiLSA edition 2009 was delivered to apply to the entire country of Germany, a developed country. Therefore, to apply to MDCs, it needs to significantly modify some chapters. Firstly, there are the modifications of the intersection layout (chapter 3 in RiLSA 2009). Secondly, the modifications of the signal programs in which some boundary conditions of signal timing, parameters of traffic flow, and calculation of signal programs have to be modified (chapter 2 in RiLSA 2009). Finally, the selection of control strategy for MDCs has to be carried out (chapter 4 in RiLSA 2009).

Chapter 6 is basically kept the same as in RiLSA, and has minor modifications.

Chapter 5, Chapter 7, chapter 8, and chapter 9 are not included.

Thus, in this study, it is necessary to answer following major questions:

Question 1: How should signalised intersection layout be designed in MDCs?

Question 2: Which elements of the signal program need to be modified in MDCs?

Question 3: Which control strategies should be chosen in MDCs?

These questions are going to be answered in chapter 4, chapter 5, and chapter 6 of this study, respectively.

4. Layout of Signalised Intersections in MDCs

4.1. Basic principles

Like the general principles for designing an intersection, the intersection layout in MDCs also has some major requirements as follows: (i) traffic safety, (ii) traffic flow quality, (iii) environmental protection, and (iv) economy (FGSV, 1988).

In addition, according to RASt 06 (FGSV, 2006), intersections in urban areas must: (i) be recognisable early enough from all approaches, (ii) be clear and understood able so that all road users can easily understand their right of way, possible conflicts with other road users, as well as tidiness and turning possibilities, (iii) be arranged so tidily that priority road users can see early enough all the waiting areas when they are approaching the dangerous point, (iv) be able to be driven smoothly and safely.

However, within the framework of traffic signals, the intersection layout must be accompanied by the proper signal programs and the proper control strategies as presented in Figure 23.

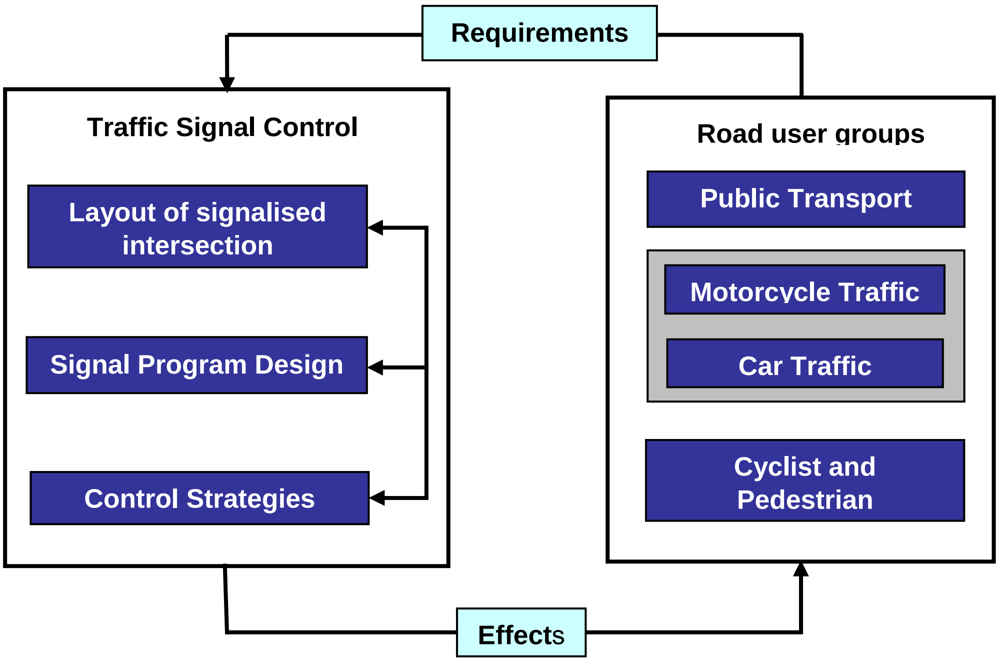


Figure 23: Road user groups and traffic signals

As analysed in chapter 2, traffic safety is one of the biggest problems at traffic signals in MDCs, and it resulted from the improper signal program and the improper intersection layout. In order to increase traffic safety at traffic signals, intersection layout should be designed so that traffic process is kept orderly, and motorcycle riders do not cause chaos due to their flexible manoeuvres.

At traffic signals in MDCs, the mixed traffic flows being operated lead to many disadvantages such as collisions between cars and motorcycles, unstable traffic flow, inaccuracy in calculating signal program elements, and difficulty in estimating traffic flow quality, etc. As presented in Figure 24, in spite of the same queuing length, traffic volume is not equal.

Therefore, one of the principles for the layout design is trying to decrease the mixed traffic status as much as possible. One of the measures is creating head-start lanes on the approach for motorcycles.

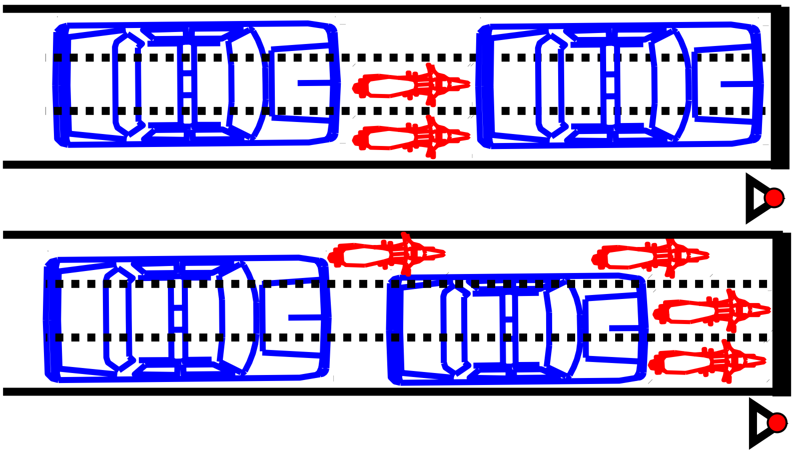


Figure 24: Unstable traffic flow with mixed traffic

Regarding signal program elements, in order to reduce the intergreen time periods, inner intersection areas should be kept as small as possible. Thus, vehicles can quickly pass the intersection, and of course capacity will be enhanced.

Cyclists and pedestrians should be taken into account when designing the intersection layout in which cyclists can join signalisation with pedestrians or motorcycles, or they can use their own signalisation. Direct and indirect left-turning cyclists should be also considered when designing the intersection layout.

To speed up public transport, locations of the bus-stops near intersections have to be taken into account. At intersections, it may also be necessary to establish exclusive lanes for buses if the approach width has more than two lanes.

Finally, layouts for pedestrians and cyclists can be basically applied as introduced in RiLSA 2009. However, layouts for motorised traffic and public transport need to be modified for MDC conditions, and they have some following basic principles:

- General design principles will be geometrical design (not dynamic speed design), because the speed limit in urban areas in MDCs is usually low. For example, in Vietnam, the speed limit is 50 km/h in urban areas.
- Inner intersection areas should be kept as small as possible.
- Decreasing the mixed traffic status as much as possible.
- If there is significant bus traffic, public transport should be also separated.

4.2. Separation of car and motorcycle traffic

4.2.1. Vehicle dimensions and lane width on approaches

- **Vehicle dimensions**

As presented in chapter 2, common motorcycles in MDCs have an engine capacity of from 70 cm^3 to 150 cm^3, popularly from 80 cm^3 to 125 cm^3. For trade purposes, many manufacturers provide many motorcycle styles that are suitable for Asian people. Besides, cars are provided by manufacturers mostly from developed countries. According to these manufacturers (Yamaha, Honda, etc.), one motorcycle can generally fit a block of 1 m x 2 m. Dimensions of a motorcycle and of a car are assumed as follows:

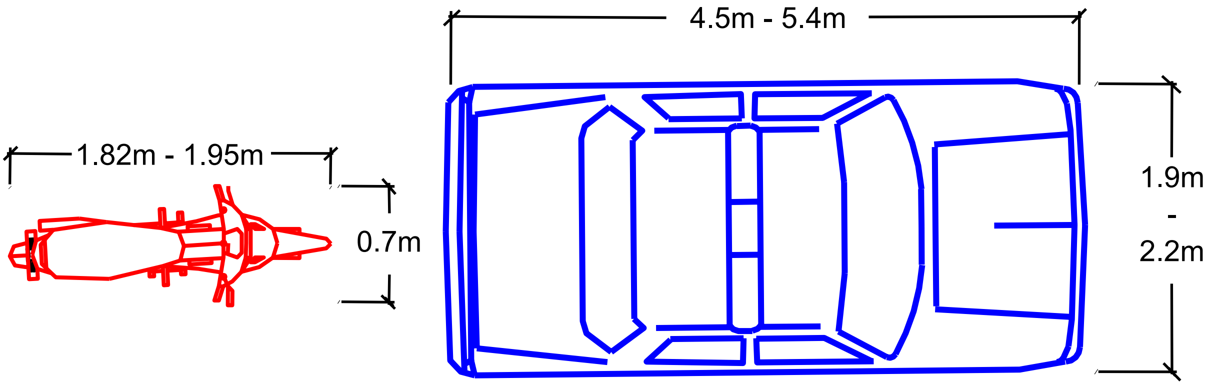


Figure 25: Vehicle's dimensions

(Yamaha and BMW, 2007)

- **Lane width on approaches**

For car traffic, according to RASt 06 (FGSV, 2006), the lane width depends on the design speed and on the function of the lane (go-through lanes, left-turning lanes or right-turning lanes). The go-through lane width at intersections shall be as wide as the lane width on the road if the go-through lane is not bordered by the edges or block areas. In constrained cases, the go-through lane width might be 0.25 m smaller than the lane width on the road. For the speed limit $V_{zul} \leq 50$ km/h on multi-lane approaches, the go-through lane width is normally 3.0 m, in exceptional case it will be reduced to 2.75 m when turning lanes are established. The left-turning and right-turning lane width is usually 0.25 m narrower than that of the go-through lane, but not smaller than 2.75 m. The bus lane width is not allowed to be narrower than 3.0 m.

For motorcycle traffic, according to Chu Cong Minh (2007), the motorcycle dynamic lane width (l_w) is determined as follows:

$$l_w = 0.07*V + 0.8 \text{ [m]} \qquad (1)$$

Where: V (m/s) is the motorcycle speed.

It is assumed that the speed limit of motorcycles is 40 km/h ($\approx$ 11 m/s), then $l_w \approx 1.57$ m.

According to Law, T.H and Radin Sohadi, R.U (2005) in the research to determine the comfortable safe width of an exclusive motorcycle lane on the Federal Highway Route 2 in Malaysia, an exclusive motorcycle lane needs a width of 3.81 meters (including marginal stripe of

0.38 meter at both edges of the road) for two riders to travel side by side comfortably at a speed of 70 km/h.

Going back to the lane width on approaches in MDCs, the design criterion is the maximum number of vehicles that can stop on the approach during the red time, hereby the queue length is shortened. With the block of 1m x 2m for a motorcycle, the motorcycle lane width at the stop-line (the stopping lane width) can be chosen as equal as multiple of 1.0 m. And from formula (1) with the speed limit at 40 km/h, the dynamic lane width for a motorcycle can be chosen as wide as 1.5 m. To satisfy both of the stopping lane width and the dynamic lane width for motorcycles, the lane width of 3.00 m should be chosen. It means that, in theory, three motorcycles can stop in a row on the lane during the red signal, and two motorcycles can drive parallel on the lane at a speed of 40 km/h during the green signal. In addition, according to RASt 06 (FGSV, 2006), the lane width of 3.00 m on the approach is also suitable for car traffic. Moreover, the lane width of 3.00 m is also suitable for the long-term plan to proceed to a car dependent city when the number of motorcycles is reduced.

Figure 26, in theory, shows the queue length on the lane width of 3.0 m of motorcycles and cars in which eighteen motorcycles can be contained on a queue length of 12 meters, which is equal to two passenger car units. In practice, it is necessary to have an adjustment factor for the number of motorcycles stopping on the lane to compare with this theoretical situation.

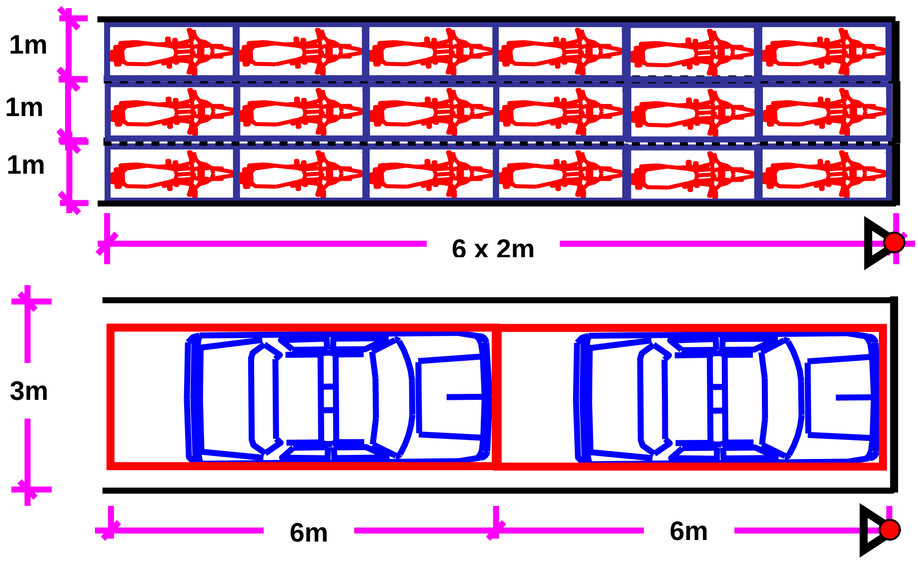


Figure 26: Lane width on approach

On the other hand, with the same queue length of 12.0 m, 18 motorcycles can carry the maximal number of 36 passengers passing the intersection, whereas two passenger cars can only carry the maximal number of 10 passengers. This comparison is not to encourage people using many motorcycles in the city, but regarding the capacity by the number of passengers passing the intersection, motorcycles play a significant role, and it is also one of the advantages of MDCs, where people density is very high.

4.2.2. Complete separation

- **Traffic regulation**

Along the road, the approach, and at the intersection, lanes for motorcycles are completely separated from car traffic as illustrated by Figure 27 and Figure 28.

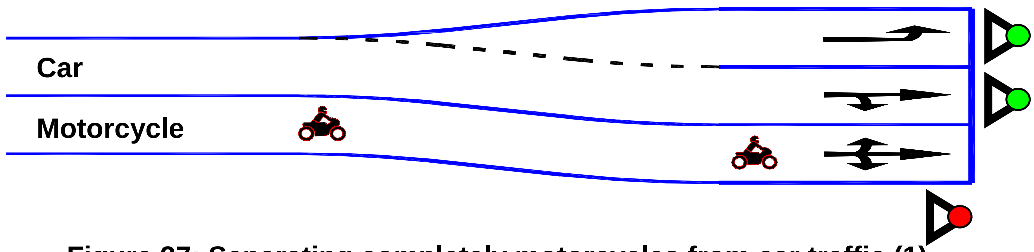


Figure 27: Separating completely motorcycles from car traffic (1)

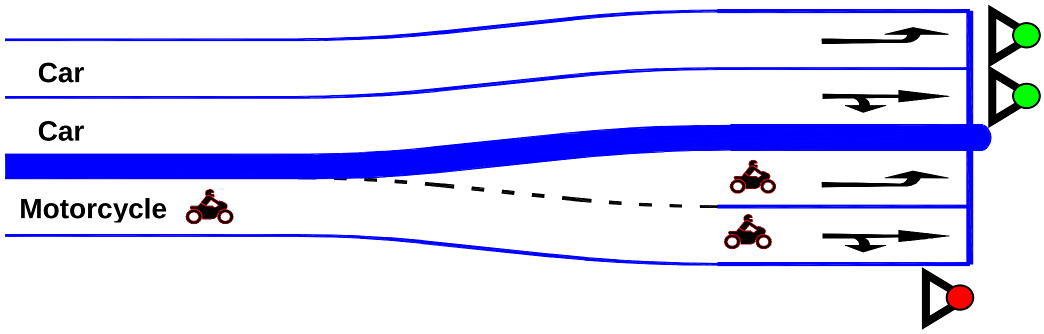


Figure 28: Separating completely motorcycles from car traffic (2)

- **Advantages**

Obviously, traffic safety is increased on the road and on the approach because motorcycles and cars are no longer mixed together. There is no any collision between them on the approach, and car's cruising speed is improved. Traffic flows are homogeneous flows of either motorcycles or cars; therefore the parameters of traffic flows for traffic signals are more accurate than that of mixed traffic flows.

- **Disadvantages**

The disadvantage is that the complex traffic process occurs in the inner intersection areas. For motorised traffic, each approach has six directions of traffic (three directions of motorcycle traffic, and the other three directions of car traffic). Thus, the traffic process in the inner intersection areas will be very complex. To solve these problems, many signal phases must be created to reduce the number of conflicting traffic streams. However, too many phases mean that the total intergreen time increases, the capacity is decreased, the cycle time becomes too long and leads to the long waiting time, too; therefore traffic flow quality is declined. Actually, Hanoi has tested this model on Tran Khat Tran Street in March 2008. As a result, the test had many traffic problems at many intersections on the street, and it was not successful.

- **Conclusion**

Finally, because of the unsolvable disadvantages, it can be concluded that ***this model of separation cannot be applied in MDCs***.

4.2.3. Partial separation

4.2.3.1. Concept of partial separation

According to the observation of behaviour of motorcycle riders on approaches, the motorcycle riders always try to get in front of cars during the red time in order to pass the intersection more quickly as soon as the signal turns to green. Therefore, the concept of "partial separation" is invented to give the motorcycle riders an opportunity to get in front of cars on the approach, it means that on the approach, during the red time, motorcycles have an opportunity to slowly move up and get in front of cars that are waiting backward from the stop-line. This concept enables the traffic streams to be operated as "partial separation of mixed traffic flows" in which motorcycle riders drive ahead and cars are driven successively when the signal turns to green. However, if motorcycles and cars are arriving at the intersection during the green time, they will share a lane and pass the intersection continuously as the mixed traffic flow. Figure 29 illustrates a "partial separation of mixed traffic flow" at the intersection.

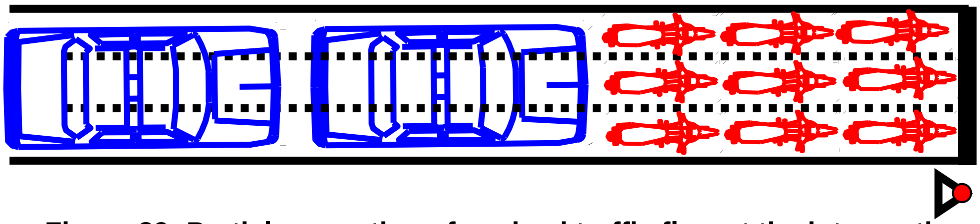


Figure 29: Partial separation of a mixed traffic flow at the intersection

However, the question is given that how to create this traffic model at intersections. The answer will be given in the following sections in which this traffic model can only be created if the approach has more than one lane. Otherwise, mixed traffic must be used because there is no more road space available to create the partial separation.

4.2.3.2. Case of high motorcycle proportion

As presented in chapter 1 about the definition of MDCs, one sub-criterion is the motorcycle proportion in the traffic flow. It is called high motorcycle proportion if this proportion is higher than 50%.

- **Traffic regulation**

  High motorcycle proportion on the road usually leads to a traffic situation with a low proportion of cars. On the road, a little number of cars means that the probability to appear long time gaps between cars is relatively high. It depends on actual car traffic volume. To save road space, motorcycles should be driven on car's lanes between these long time gaps. Therefore, the mixed motorised traffic flows along the road will be operated. But, on the approach at the intersection, mixed traffic is partly separated as illustrated in Figure 30.

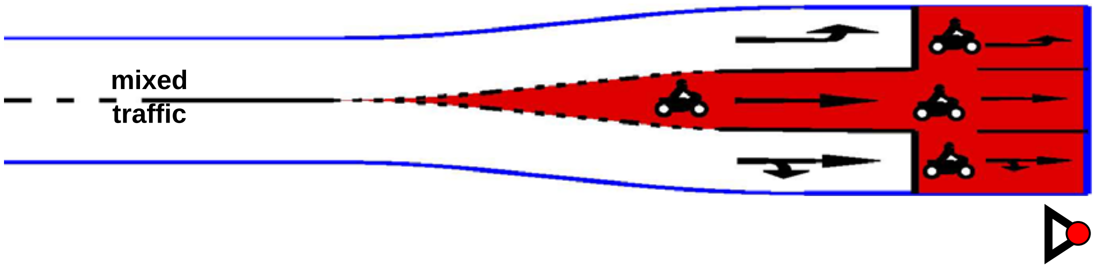


Figure 30: Approach layout in case of high proportion of motorcycles

The traffic regulation is defined as follows: "If vehicles (motorcycles and cars) arrive at the intersection during the green time, they can successively pass the stop-line. If vehicles arrive at the intersection during the red time, cars must stop in front of their own stop-lines that are backward from the intersection, then motorcycle riders who are driving behind cars, have to decelerate and turn into the central lane in order to get in the head-start lanes. When the signal turns green, these motorcycles will first pass the stop-line and cars are followed". The central lane is only used for motorcycles during the red time. If vehicles arrive at the intersection during the amber time, they can decide either to cross or to stop in front of their own stop-lines. To make the drivers comprehensively understood able about this traffic regulation, the central lane and the head-start area for motorcycles should be marked by dark-red colour.

- **Advantages**

  + Advantage of traffic safety

  Compared to mixed traffic, critical collisions between motorcycles and cars on the approach and in the inner intersection areas will be reduced. When the green time begins, the phenomenon in which motorcycles flexibly manoeuvre and overcome cars is eliminated. Traffic process will be in order.

  The opportunity for motorcycle riders to get in front of cars during the red time will abate their stress due to emission and weather conditions because unlike car drivers, the motorcycle riders are not shielded by an enclosed compartment.

  By this approach layout, the leading green time for turning motorcycles can be applied. Hereby, the number of conflicts with crossing pedestrians is reduced.

  + Advantage of traffic flow quality

  The waiting time will be shorter comparing to mixed traffic because with the same traffic volume on the approach, this traffic model gives a shorter cycle time. This is due to the saturation flow rate of this traffic model is higher than that of mixed traffic (note that mixed traffic flow is unstable, and see the section of the cycle time calculation).

  + Advantage of capacity

  Because of higher saturation flow rate comparing to mixed traffic flow, this traffic model will give a higher capacity than that of the mixed traffic flow.

+ Does not much influence on the long-term plan for car-oriented layout

Because this approach layout is only applied in case of a high motorcycle proportion shared in the traffic flow, the central lane width for motorcycles accessing the head-start lanes should be designed as equally as 3.0 m. Therefore, it is easier to change this layout approach (see Figure 30) into the layout approach for car-oriented traffic by marking (see Figure 31). This will be implemented when the proportion of motorcycles is no longer high.

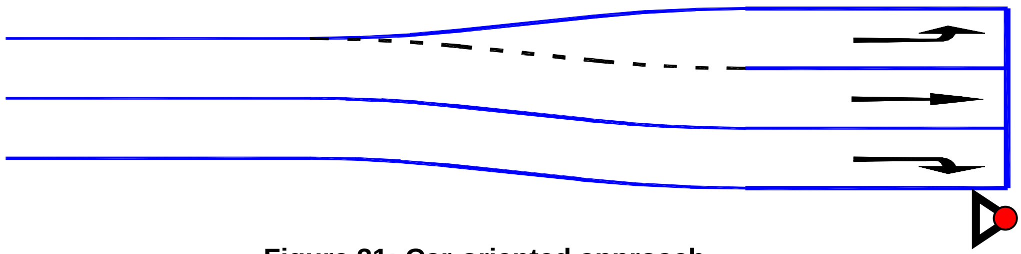


Figure 31: Car-oriented approach

- **Disadvantage**

The disadvantage of this layout approach (see Figure 30) is that right-turning and go-through cars have to share a lane because of the limitation of road space due to establishing the head-start area for motorcycles. However, this is acceptable because, in this case, car traffic volume is not very high.

- **Conclusion**

It is very suitable to apply this model of separation for the cases of high motorcycle proportion on approaches (higher than 50%).

- **Dimensions of the head-start area**

The width of the head-start lanes is designed based on the lane width that was presented in section 4.2.1. However, in some exceptional cases, the head-start lane width can be wider than the lane width on the road.

The length of the head-start lanes is designed based on the number of motorcycles arriving at the intersection during the red time. Firstly, the cycle time is calculated from traffic volume of motorcycles and cars (see section the cycle time calculation). Secondly, the green time is calculated for each phase. Finally, from the cycle time (t_U), the green time (t_{Fi}), and the amber time (t_{Gi}), the red time t_{Ri} is calculated by the following formula:

$$t_{Ri} = t_U - t_{Fi} - t_{Gi} \text{ [second]}$$

The average number of motorcycles arriving at the intersection during the red time is determined as follows:

$$q_{Ri}^{mc} = \frac{q^{mc}}{3600} . t_{Ri} \text{ [vehicles]}$$

In theory, the length of the head-start lane is determined as follows:

$$l_i^{mc} = \frac{q_{Ri}^{mc}}{w_i^{mc}}.2 \text{ [m]}$$

Where: w_i^{mc} = the number of motorcycles that can stop in a row on the lane (normally, one motorcycle is fit to 1 m wide. Hereby, w_i^{mc} can be understood as the width of the head-start lane, which is chosen as multiple of 1 m).

2 is the length of the block for one motorcycle stopping [m].

Depending on motorcycle traffic volume on each lane, the length of the head-start lanes may be various as illustrated by following figures in which Figure 32 is applied when right-turning and go-through motorcycle traffic volume is high, and Figure 33 is applied when left-turning motorcycle traffic volume is high, and the width of the left-turning head-start lane can be 4 m or 5m. In both of Figure 32 and Figure 33, it is not necessary to locate the stop-lines equally on the car lanes as illustrated in Figure 30.

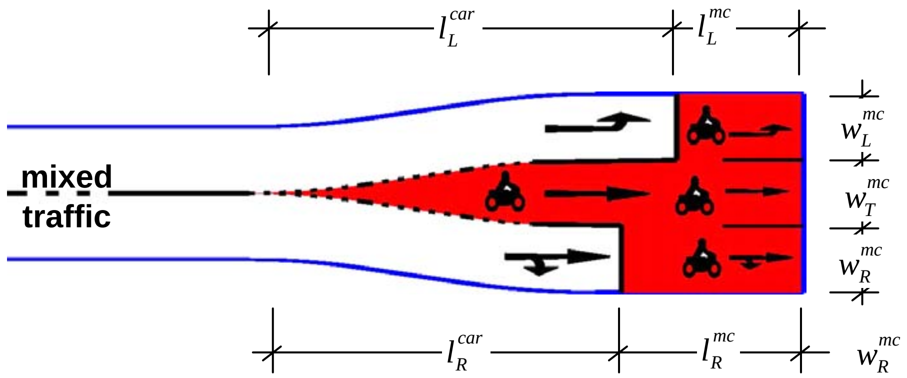


Figure 32: High traffic volume of right-turning and go-through motorcycles

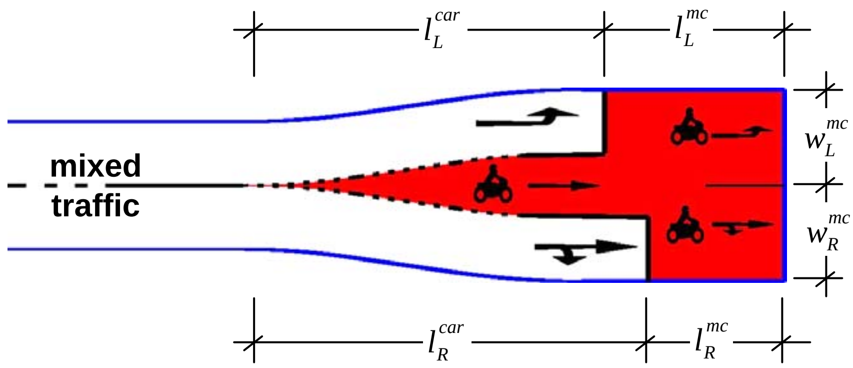


Figure 33: High traffic volume of left-turning motorcycles

- **Dimensions of the central lane for motorcycles**

The length of the central lane for motorcycles l_i^{car} is designed so that the stopping cars do not prevent motorcycles from accessing the head-start area. Therefore, this length should be equal to the queue length of cars as computed according to HBS 2001 that was transferred to annex 3 of the draft of "Guidelines for Traffic Signals in MDCs".

As discussed above, the width of the central lane is designed as equally as 3.0 m. The central lane is also marked by red-dark colour, and its borders are designed depending on the route marking of the adjacent car's lanes.

4.2.3.3. Case of medium motorcycle proportion

As defined in chapter 1, it is called a medium motorcycle proportion if the motorcycles, which are shared in the traffic flow, are from 30% to 50%.

a. Using an exclusive signal head for motorcycles

- **Traffic regulation**

Contrasting with the case of a high motorcycle proportion, the number of cars in the traffic flow is higher, especially on the ring roads or on the arterials, where the speed limit of car traffic is usually higher. Therefore, the aim of this case is to reduce influences of motorcycles on car traffic, thus car's cruising speed will be improved.

Along the roads, motorcycles are separated completely from car traffic, but on the approach at the intersection, motorcycles have an opportunity to get in front of the stopping cars during the red time as illustrated in Figure 34 and Figure 35.

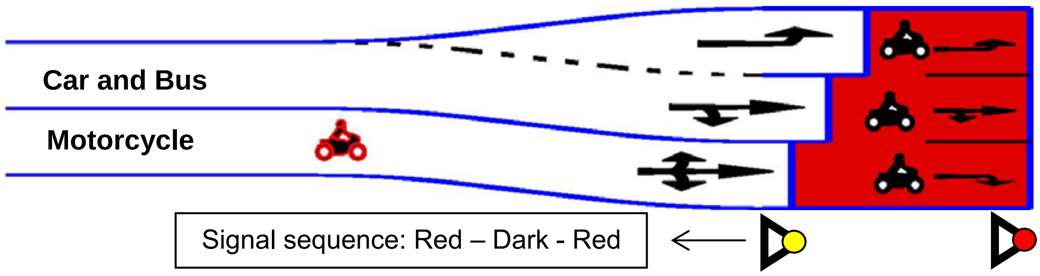


Figure 34: Approach layout in case of medium motorcycle proportion (1)

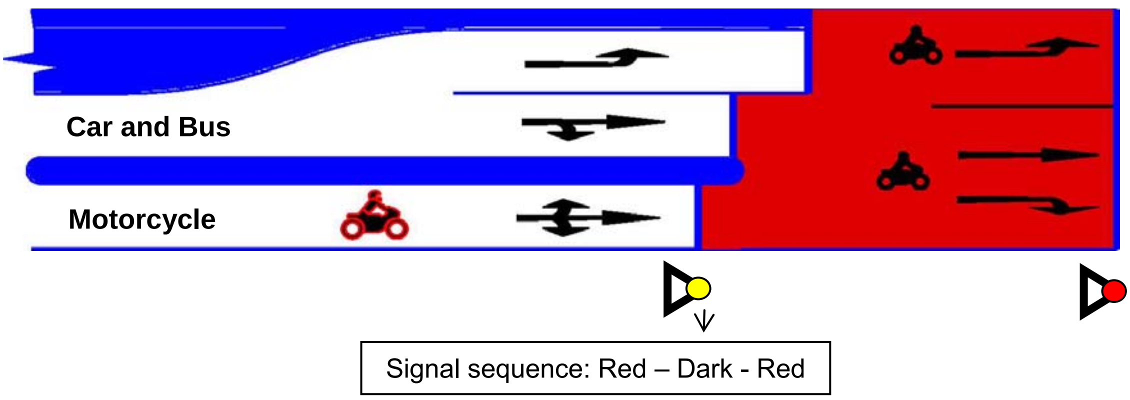
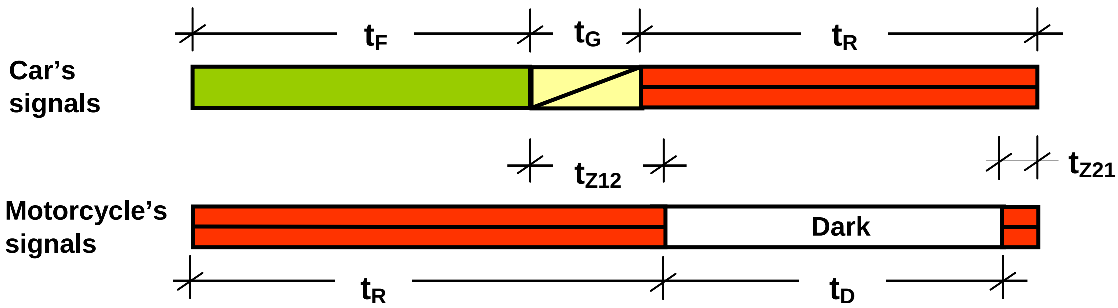


Figure 35: Approach layout in case of medium motorcycle proportion (2)

The exclusive signal head for motorcycles is used to distribute motorcycles over the head-start area in every red phase of the approach. To eliminate the misconception of motorcycle riders from the main signal head of the approach, the exclusive signal head for motorcycles should have only one lens with the motorcycle symbol, and it has the signal sequence: Red – Dark – Red. The exclusive signal head for motorcycles and the main signal head for the approach must be co-ordinated as follows:



- **Advantages**

- Motorcycles do not influence on car traffic on the road, car's cruising speed and traffic safety on the road, therefore, is improved.
- Traffic safety at the intersection is also improved by using the exclusive signal head for motorcycles.

- **Disadvantages**

- The waiting time for motorcycle riders becomes long because they can only access the head-start area when all phases of car traffic on the approach is red.
- Capacity of motorcycles is decreased due to the long waiting time.
- Arising the intergreen time between the exclusive motorcycle flows and the motorised flows on the approach.

- **Conclusion**

This approach layout should only be applied when the motorcycle traffic volume is relatively low, for example from 30% to 40% shared in the traffic flow. Therefore, most of motorcycle riders can be released in every cycle time.

b. Not using the exclusive signal head for motorcycles

- **Traffic regulation**

Traffic regulation on the road is the same as in **case a** in which motorcycles are separated from car traffic on the road. However, on the approach, the exclusive signal head for motorcycles (see Figure 34 and Figure 35) is not used to distribute motorcycles over the head-start area. The motorcycle riders in this case have to pay attentions themselves on car traffic to join the current green phase or to access the head-start lanes.

According to this traffic regulation, the left-turning motorcycle riders have a difficulty in accessing the left-turning head-start lane because of their difficult trajectories, and the queue length of the right-turning and go-through motorcycles available on the head-start area may prevent them. On the other hand, the stopping left-turning motorcycle riders on the exclusive motorcycle lane may prevent the right-turning and go-through motorcycles from accessing the head-start area.

In order to make the traffic regulation easier and more feasible in case of high left-turning motorcycle traffic volume, the approach layout in Figure 34 and Figure 35 should be modified as follows:

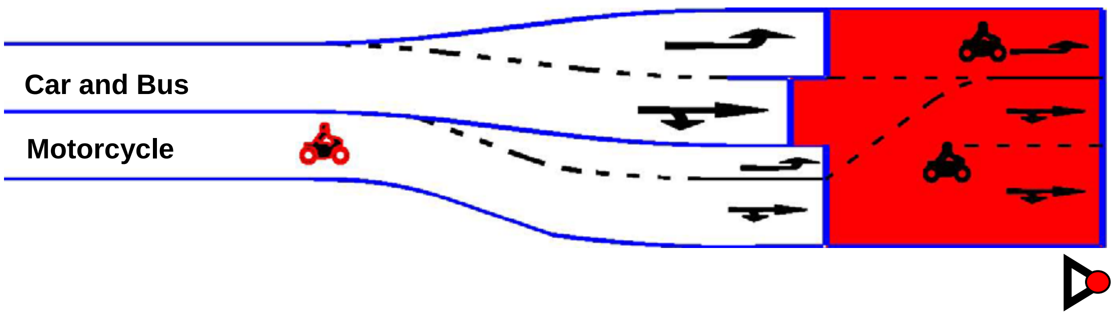


Figure 36: Left-turning lane for motorcycles

According to this approach layout, the traffic regulation is defined as follows:

- If the signal for right-turning and go-through vehicles is red, and the signal for left-turning vehicles is green, then the left-turning motorcycle riders on the exclusive lane can join the left-turning green phase by accessing the area in front of the go-through car lane (see Figure 36), and the right-turning and go-through motorcycles can access and wait in the head-start area.
- If the signal for right-turning and go-through vehicles is green and the signal for left-turning vehicles is red, then left-turning motorcycle riders have to wait on the exclusive left-turning motorcycle lane while right-turning and go-through motorcycles can continuously pass the intersection.
- If the signal for the whole approach is red, then all left-turning, right-turning and go-through motorcycles can access the head-start area.

- **Advantages**

- Improving traffic safety on the road and the approach.
- Motorcycles on the exclusive lane can continuously join the motorised green phase, therefore reducing the waiting time for motorcycles comparing to **case a**.
- Increasing capacity of motorcycle due to be able to continuously join the green phase.
- Traffic regulation is more advantageous than that of **case a**.

- **Disadvantages**

The only disadvantage is that the intersection layout needs more space for exclusive left-turning motorcycles (at least 1.5 m wider than **case a**).

- **Conclusion**

This intersection layout can be applied at the large intersection with a relative high proportion of motorcycles, from 40% to 50% in the traffic flow.

4.2.3.4. Case of low motorcycle proportion on the road

When the proportion of motorcycles in the traffic flow is lower than 30%, traffic will be operated as in developed countries. It means that motorcycles and cars are mixed together, and the intersection layout does not need any modifications.

4.2.3.5. Case of one-lane roads

In this case, there is no more road space to separate motorcycles from car traffic; mixed traffic on approach, therefore, is operated as in Figure 37.

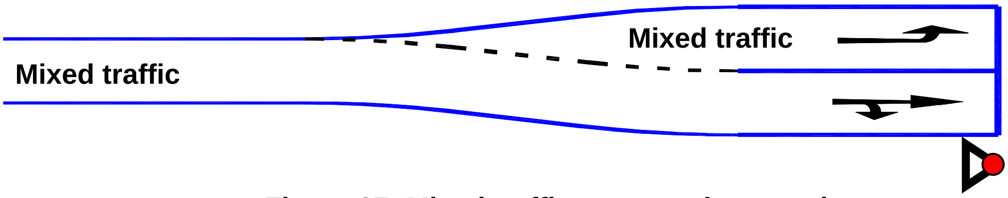


Figure 37: Mixed traffic on a one-lane road

In this case, usually, large buses are prohibited to increase traffic safety. Instead, it is possible to operate mini-buses. The speed limit of vehicles is also lower than that of the other cases.

4.3. Exclusive lanes for buses

As discussed in chapter 2 about the problems of public transport (focusing on buses) in MDCs in which buses are being operated under mixed traffic conditions. In order to solve these problems, buses should be separated from motorcycle traffic along the roads and at intersections. Thus, two levels for implementing this purpose might be considered.

In the lower level, motorcycle traffic is separated from car traffic, and buses share lanes with passenger cars as presented in Figure 34, Figure 35, and Figure 36 already. In constrained cases due to the narrow road width, buses can be operated along the roads under the mixed traffic condition, but at intersection, buses and cars are partly separated from motorcycle traffic as presented in Figure 30, Figure 32 and Figure 33 already. In exceptional case, if the motorcycle proportion is very high (higher than 80%), and there is no road space for separating public transport along the roads, buses and motorcycles can share the lateral lane in which buses drive only on that lane, do not change lane to lane, and bus-stops are located along this lane. But, at the intersection, a partial lane only for buses is reserved by checked marking in order to give priority to buses and to prevent collisions from motorcycles when the signal turns green (see Figure 38).

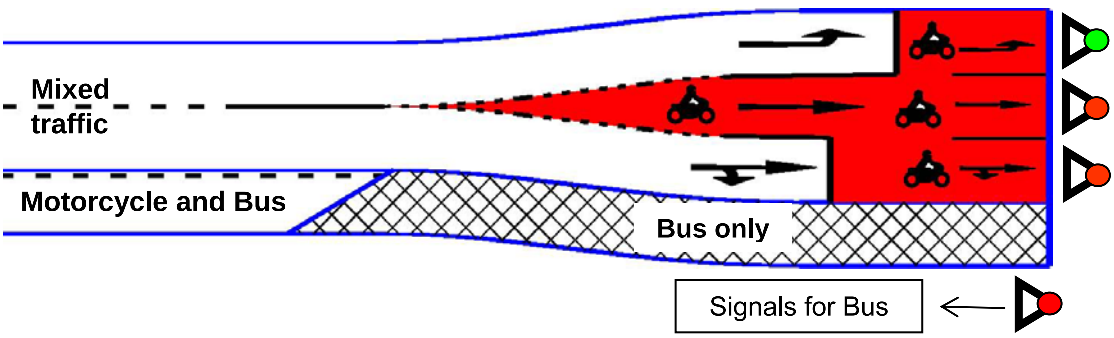


Figure 38: Partial lane for buses

In the higher level, an exclusive lane only for buses is established along the road and at the intersection (see Figure 39). This can be implemented on the ring roads or on the arterial roads that have more than two lanes and significant bus traffic. In cases of exclusive lanes for buses, it is necessary to give priority to them at intersections in which signal programs must be coordinated with the location of the bus-stops. If a bus-stop is located immediately in front of intersection, signal programs must ensure that buses will arrive at the intersection during the red time, during which passengers can board and alight, thus it will save time for buses, and the buses have only one stop at the intersection. If the bus-stop is located behind intersection, it will have a long period of time to prepare the green time for buses at the following intersection.

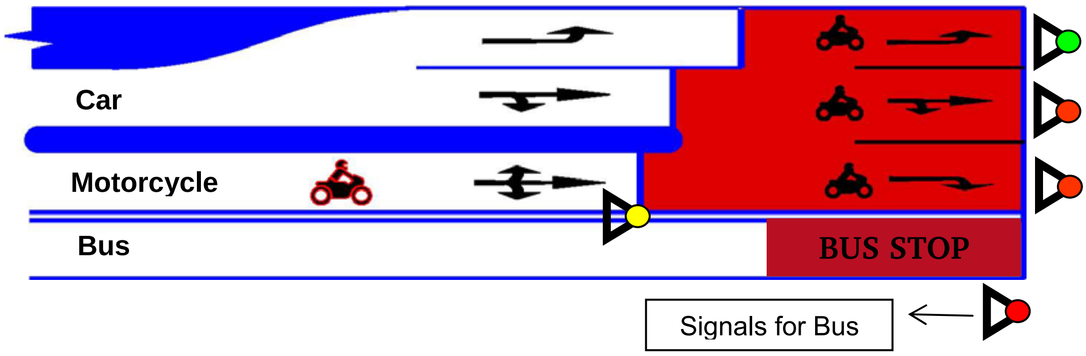


Figure 39: Exclusive lane for buses

If the exclusive lane for buses is established, the exclusive signals for them should be applied as introduced in RiLSA 2009.

4.4. Bicycle lanes and pedestrian crossings

As introduced in RiLSA 2009, cyclists can join signalisation with motorised traffic or with pedestrians, or have the exclusive signal head.

In Germany, the bicycle paths are usually designed on the sidewalks so that cyclists can easily approach the waiting areas and join signals with pedestrians. However, in MDCs, the cycle lanes are usually designed on the carriageways nearby the sidewalk. In these cases, one more possibility in the intersection layout for cyclists to join signalisation with pedestrians is recommended, which is illustrated in Figure 40. In this figure, transitional segments for cyclists to ride upward to the sidewalk and downward to the carriageway are needed.
The design of pedestrian crossing is kept the same as in RiLSA 2009.

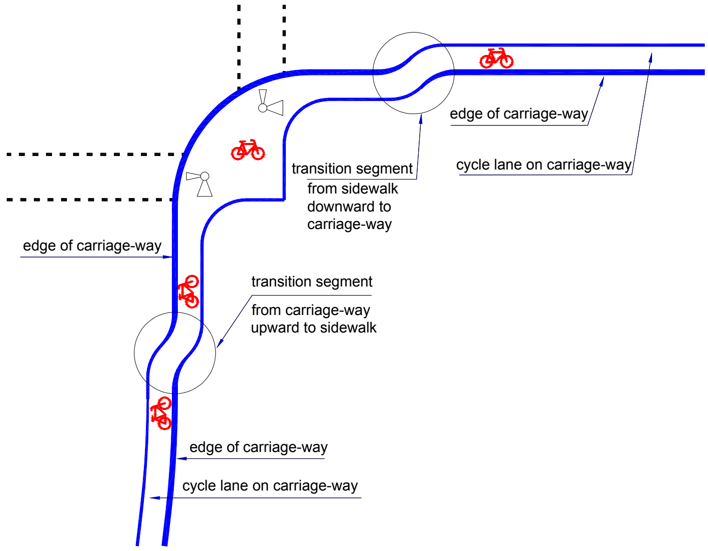


Figure 40: Transitional segments for cyclists

4.5. Right-turning lanes

4.5.1. Exclusive right-turning lanes

Generally, right-turning movements are not as critical as left-turning movements because they do not have conflicts with opposing go-through traffic. Therefore, it is not necessary to separate motorcycles from car traffic on the exclusive right-turning lanes (see Figure 41).

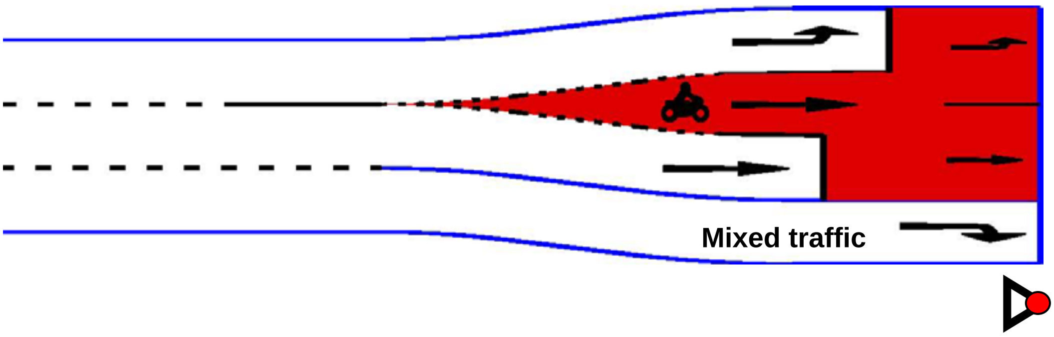


Figure 41: Exclusive right-turning lane

4.5.2. Right-turning carriageways with triangular islands

Generally, if triangular islands are established, right-turning movements will be driven faster and more comfortably. In these cases, it may be not necessary to install signalisation on the right-turning carriageways if cyclists and pedestrians concerned are not impaired. As discussed above, it is also not necessary to separate motorcycles from car traffic on the right-turning carriageways.

In case of a high proportion of motorcycles, mixed traffic along the road and on the right-turning lane will be operated. However, go-through cars have their own lane at the stop line while left-turning and go-through motorcycles are separated by the head-start area. When signal turns green, go-through motorcycles and cars will parallel drive on different lanes (see Figure 42).

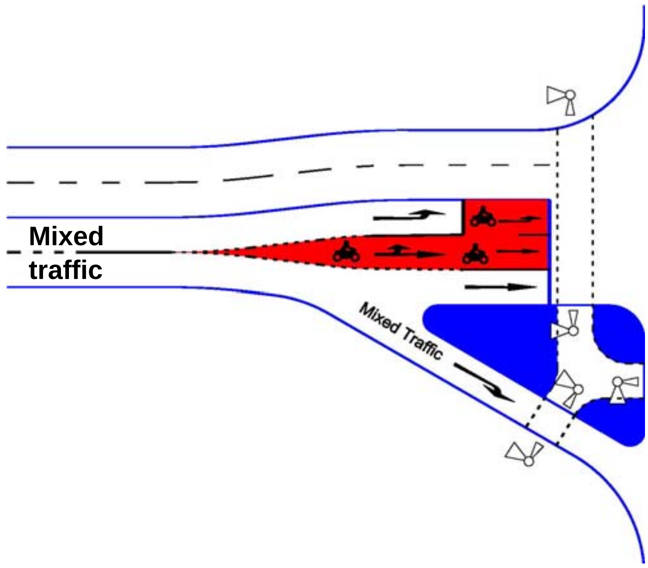


Figure 42: Right-turning carriageways with triangular islands (1)

In case of a low proportion of motorcycles, motorcycles are separated from car traffic along the road. But, mixed traffic on the right-turning carriageway is operated. Left-turning and go-through motorcycles will use the head-start area as described in section 4.2.3.3. However, the conflicts between right-turning cars and motorcycles at the beginning of the right-turning carriageway may occur, but this is acceptable when motorcycle traffic volume is low, and the car drivers, therefore, have to wait for time gaps between motorcycles to turn right (see Figure 43).

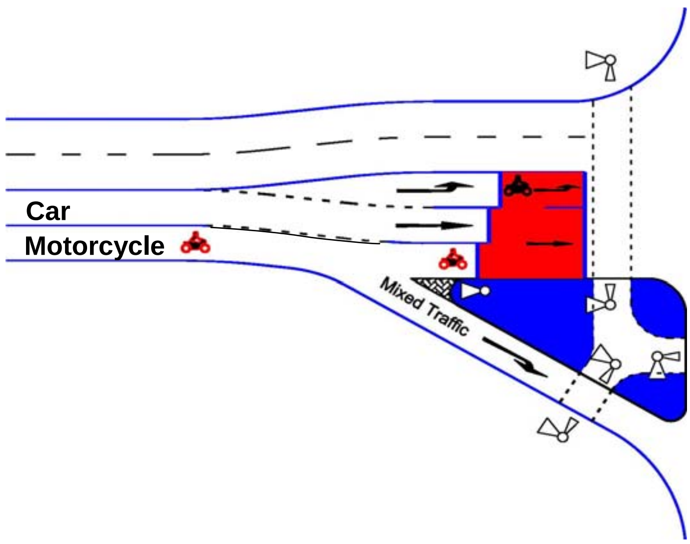


Figure 43: Right-turning carriageways with triangular islands (2)

5. Calculation of Signal Program Elements for MDCs

5.1. Saturation flow

5.1.1. General

Saturation flow is one of the most important parameters to determine the capacity of an intersection, the green times for traffic flows in each phase, the degree of saturation, and the cycle time.

Webster and Cobbe (1966) defined that: ''the saturation flow is the flow, which would be obtained if there was a continuous queue of vehicles and they were given a 100 percent green time. It is generally expressed in vehicles per hour of green time''.

Also according to Webster and Cobbe (1966), during the first few seconds of the green time and during the amber time, the average rate of flow passing the stop-line is lower (see Figure 44) because at the beginning of the green time vehicles need some time to start and accelerate to normal running speed, and at the ending of the green time (when the amber time starts) the vehicles tend to move slowly to stop at the stop-line before the red signal turns.

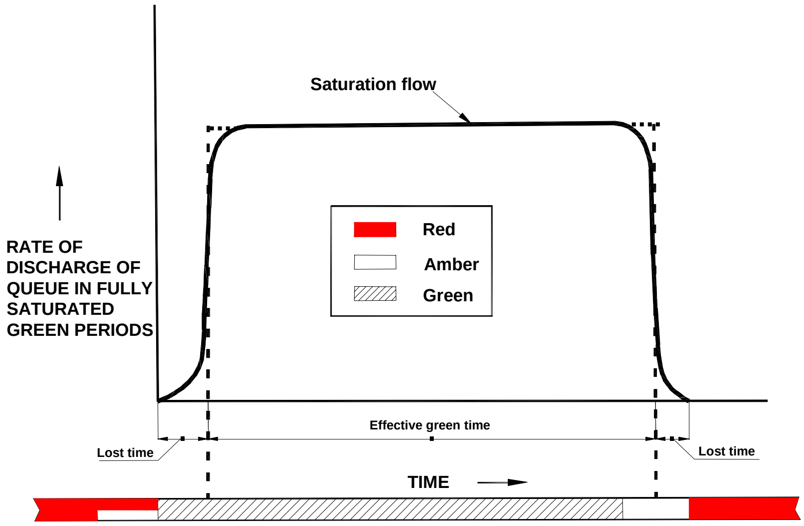


Figure 44: Variation with time of discharge rate of queue in a fully saturated green period

(Webster and Cobbe, 1966)

To have the constant value of the saturation rate, Webster and Cobbe (1966) used the concept of ''effective green'' period as shown in Figure 44. It means that the curve in Figure 44 is replaced by an equal area rectangle. Therefore, the effective green time is equal to the green time minus the start-up lost time, and plus a part of the amber time.

Also according to Webster and Cobbe (1966), the unit of p.c.u/h (passenger car unit per hour) is used to express the saturation flow rate. The saturation flow rate is affected by approach width, gradients of approach, traffic composition (each type of vehicle is equivalent to a number of private cars in respect of its road-capacity requirements), right-turning traffic, left-turning traffic, pedestrians, parked vehicles, and site characteristics. For example, the effect of approach width is estimated as follows:

Table 15: Effect of approach width on saturation flow

W	10 (3.05 m)	11 (3.35 m)	12 (3.66 m)	13 (3.96 m)	14 (4.27 m)	15 (4.57 m)	16 (4.87 m)	17 (5.18 m)	feet
S	1850	1875	1900	1950	2075	2250	2475	2700	p.c.u/h

(Webster and Cobbe, 1966)

The effect of the traffic composition is given by the equivalent factor as follows:

1 heavy or medium goods vehicle	= 1 ¾ p.c.u
1 bus	= 2 ¼ p.c.u
1 tram	= 2 ½ p.c.u
1 light goods vehicle	= 1.0 p.c.u
1 motorcycle, moped, or scooter	= 1/3 p.c.u
1 pedal cycle	= 1/5 p.c.u

(Webster and Cobbe, 1966)

According to the Highway Capacity Manual HCM 2000 (TRB, 2000), the definition of the saturation flow rate is similar to that of Webster and Cobbe (1966): ''The saturation flow rate is the flow in vehicles per hour that can be accommodated by the lane group assuming that the green phase were displayed 100 percent of the time''. HCM 2000 built the formula for estimating the saturation flow rate as follows:

$$s = s_o \, N \, f_w \, f_{HV} \, f_g \, f_p \, f_{bb} \, f_a \, f_{LU} \, f_{LT} \, f_{RT} \, f_{Lpb} \, f_{Rpb} \qquad (2)$$

Where:

s = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h)

s_o = base saturation flow rate (s_o = 1900 passenger cars/hour/lane)

N = number of lanes in lane group

f_w = adjustment factor for lane width

f_{HV} = adjustment factor for heavy vehicles in traffic stream

f_g = adjustment factor for approach grade

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group

f_{bb} = adjustment for blocking effect of local buses that stop within intersection area

f_a = adjustment factor for area type

f_{LU} = adjustment factor for lane utilisation

f_{LT} = adjustment factor for left turns in lane group

f_{LT} = adjustment factor for right turns in lane group

f_{Lpb} = pedestrian adjustment factor for left-turn movements

f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements.

Each individual factor is determined by its own formula in exhibit 16-7, chapter 16 in HCM 2000.

From formula (2) for estimating the saturation flow rate, it can be seen that the unit of saturation flow is vehicles per hour (not p.c.u/h), and there is no effect of traffic composition on the saturation flow rate except for heavy vehicles, which is expressed by its adjustment factor:

$$f_{HV} = \frac{100}{100 + \%HV(E_T - 1)} \qquad (3)$$

Where: % HV = % heavy vehicles for lane group volume,

E_T = 2 passenger car / heavy vehicle.

This can be understood that traffic volume of other types of vehicles in the United States is insignificant. Therefore, they do not affect much on the saturation flow rate.

In Germany, according to the HBS 2001 (FGSV, 2001), the saturation flow rate $q_{S,st}$ is defined as: ''The saturation flow rate is the maximum flow rate per one lane, which can pass the stop-line unimpeded during one hour green time''.

The unit of the saturation flow rate can be either p.c.u/h or veh/h if the proportion of heavy vehicles is less than 2%. Otherwise, it will be veh/h.

Depending on the saturation headway t_B (second/veh), the saturation flow rate $q_{S,st}$ is determined as follows:

$$q_{S,st} = \frac{3600}{t_B} \quad \text{(veh/h)} \qquad (4)$$

HBS 2001 gave the values of the saturation flow rate depending on the green time period as follows:

Table 16: Saturation flow values depending on the green time

Green time period t_F [s]	Saturation flow rate $q_{S,st}$ [veh/h]	Saturation headway [s/veh]
> 10	2000	1.8
10	2400	1.5
6	3000	1.2

(FGSV, 2001)

Then, these saturation flow rate values are adjusted by the following formula:

$$q_S = f_1 . f_2 . q_{S,st} \qquad (5)$$

Where: f_1 and f_2 = the two highest factors among the five factors as presented in Table 17

$q_{S,st}$ = saturation flow rate as given in Table 16.

Table 17: Adjustment factors for the saturation flow rate

Parameters			Adjustment factor
Heavy vehicle proportion	SV < 2 %		f_{SV} = 1
	SV = 2 ÷ 15 %		$f_{SV} = 1\text{-}0.0083\ e^{0.21\ SV}$
	SV > 15 %		f_{SV} = 1 / (1+0,015 . SV)
Lane width	2.6 m		f_b = 0.85
	2.75 m		f_b = 0.90
	≥ 3.00 m		f_b = 1.00
Turning radii	R	≤ 10 m	f_R = 0.85
		≤ 10 m	f_R = 0.90
		> 10 m	f_R = 1.00
Approach gradient	+5 %		f_S = 0.85
	+3 %		f_S = 0.90
	0 %		f_S = 1.00
	-3 %		f_S = 1.10
	-5 %		f_S = 1.15
Pedestrian traffic	high		f_F = 0.80
	medium		f_F = 0.90
	low		f_F = 1.00

(FGSV, 2001)

From Table 16, it can be seen that the saturation flow rate $q_{S,st}$ is impaired when the green time period becomes longer, and the saturation flow rate can reach 3000 veh/h when the green time is 6 seconds. This can be explained as: German people use the red and amber signal (t_{RG} = 1s) between the red and the green signal, thus they do not have to waste the start-up lost time. Therefore, they do not use the concept of ''effective green time'', instead they use the concept of ''green time is green time''. Hereby, when the green time t_F is short and traffic volume is too high, the saturation headway t_B reduces significantly due to a number of vehicles still crossing the stop-line during the amber time (note that $t_B = \frac{t_F}{\text{total vehicles crossed the stop - line}}$). When t_B becomes lower, the saturation flow rate $q_{S,st}$ will increase according to formula (4).

From the saturation flow concept in some countries, it can be comprehensively understood that: each country has its own prevailing traffic situations and therefore, the concept of saturation flow rate is more or less different.

5.1.2. Saturation flow in MDCs

5.1.2.1. General

Going back to MDCs, from the calculation of the cycle time and green time recommended in this study, the saturation flow rate value of homogeneous car traffic as well as that of homogeneous motorcycle traffic (q^{car} and q^{mc}) are required, in which the saturation flow rate of homogeneous car traffic can be taken from the German Highway Capacity Manual - HBS. Therefore, the saturation flow rate of homogeneous motorcycle traffic needs to be researched.

Since mixed traffic with many types of vehicle (private car, bus, truck, motorcycle, and bicycle) has been formed in MDCs, the research on the saturation flow rate becomes much complex and requires many efforts. Under this mixed traffic condition, the regression method is usually used to determine the saturation flow rate as well as equivalent factors converting other types of vehicle into the standard vehicle. In this study, two published researches are introduced and analysed in order to develop the concept of saturation flow for MDCs.

5.1.2.2. Saturation flow by using Passenger Car Unit (PCU)

The first one was researched by Chu Cong Minh (2003) as his master thesis in the Asian Institute of Technology (AIT) with the title ''Analysis of motorcycle effects to saturation flow rate at signalised intersection in developing countries''. This research was published by the journal of the Eastern Asia Society for Transportation Studies in October 2003.

In this study, he used the conventional concept of the saturation flow rate, that is, all other types of vehicle are converted into passenger car unit. He collected data at three signalised intersections in Bangkok in Thailand and four intersections in Hanoi in Vietnam to have a comparison between these two cities.

In Bangkok, the traffic data was collected in peak hours on the lane width from 3.2 m to 5 m depending on each approach. Traffic composition included only private cars, motorcycles, and buses, in which the proportion of motorcycles was in average of 20% in the traffic stream on lanes.

In Hanoi, the traffic data was also collected in peak hours on the lane width from 3.5 m to 5 m, trucks and heavy vehicles were prohibited at that time. The proportion of motorcycles was in average of 90%.

The varying saturated green times from 5 s to 45 s were recorded to estimate the equivalent factor of motorcycle into passenger car unit, the average saturation headway, and the saturation flow rate.

Chu Cong Minh assumed that: ''the relationship between dependent variables is linear'', and he, therefore, used the following regression formula through his study as follows:

$$t = a_1.n_1 + a_2.n_2 + a_3.n_3 \qquad (6)$$

Where t = saturated green time (s),

a_1, a_2, a_3: coefficients of motorcycle, private car, and bus, respectively,

n_1, n_2, n_3: number of motorcycles, private cars, and buses respectively, which pass the stop- line during the time t.

Chu Cong Minh used the initial vehicular equivalent factors in Table 18 for determining saturation condition while collecting data. It means that, every consecutive five-second of green times is recorded, and vehicles passing the stop-line during these five seconds are converted into passenger car units by the equivalent factors in Table 18. If more than three passenger car units pass the stop-line during 5 seconds, the traffic flow is considered to be saturated.

Table 18: Passenger car equivalent (PCE) for other vehicles (1)

Vehicle	PCE
Motorcycle, moped, scooter	0.25
Passenger car, van, taxi	1.00
Bus	2.00

(Chu Cong Minh, 2003 according to Mathetharan, 1997)

Then, the average saturation headway (after converting into passenger car unit) is determined as follows:

$$H = \frac{t}{n_1 p_1 + n_2 p_2 + n_3 p_3} \text{ [s/veh]} \qquad (7)$$

Where $p_1 = \frac{a_1}{a_2}; p_2 = 1; p_3 = \frac{a_3}{a_2}$ are equivalent factors of motorcycle, private car, and bus into passenger car unit.

From the average saturation headway H, the saturation flow rate is then determined by the formula: $\frac{3600}{H}$ [p.c.u/h].

After regression analyses from the collected data, Chu Cong Minh achieved the results as follows:

In Hanoi: $t = 0.207\ n_1 + 0.85\ n_2 + 1.918\ n_3$ with $R^2 = 0.99$

In Bangkok: $t = 0.281\ n_1 + 1.603\ n_2 + 3.487\ n_3$ with $R^2 = 0.99$

Table 19: Passenger car equivalent (PCE) for other vehicles (2)

City	Motorcycle	Car, van, taxi	Bus
Hanoi	0.24	1.00	2.26
Bangkok	0.18	1.00	2.18

(Chu Cong Minh, 2003)

Table 20: Comparison of estimated saturation flow rate

City	Average headway statistics		Saturation flow rate
	Mean (s)	Standard deviation	(p.c.u/h/5m width)
Hanoi	0.88	0.11	4092
Bangkok	1.60	0.12	2253

(Chu Cong Minh, 2003)

Looking at the results above, it is suspected that why the saturation flow rates in Hanoi and in Bangkok is quite different with the same lane width of 5m. The result of the saturation rate in Bangkok is too low while it is relatively high in Hanoi.

In order to make this suspicion clear, there are some considerations for the methodology of Chu Cong Minh as follows:

- The first consideration is in formula (6): $t = a_1.n_1 + a_2.n_2 + a_3.n_3$ [s]

  Because t is the saturated green time in unit [second]; n_1, n_2, n_3 are the number of motorcycles, private cars, buses in unit [vehicle]; a_1, a_2, a_3, therefore, could be understood as saturation headways of motorcycles, private cars, buses, respectively, and in unit [s/veh]. Therefore, formula (6) is fit to the following traffic model:

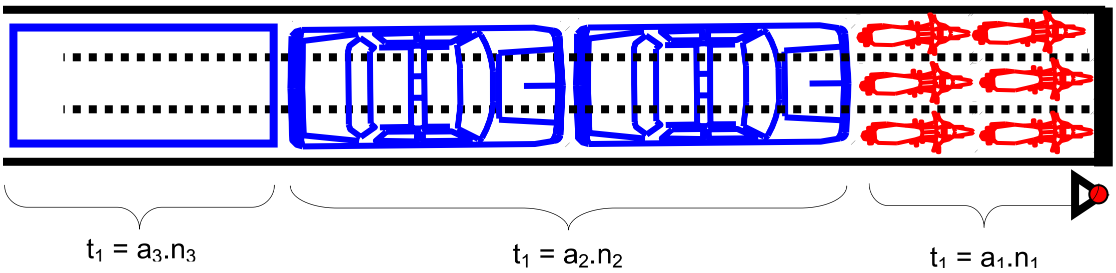


Figure 45: Analysing Chu Cong Minh's methodology

  Nevertheless, Chu Cong Minh collected data under the mixed traffic condition and applied these data to the traffic model above without any adjustment. As a result, there is a big difference between the assumption and the reality (note that the mixed traffic condition is quite different from the traffic model in Figure 45). The result of his research, therefore, is far different from the reality.

- The second consideration is the determination of the saturated condition.

  Chu Cong Minh used the equivalent factors in Table 18 as the initial values to determine the saturated condition for mixed traffic (more than three passenger car units passing the stop-line during 5 seconds). The questions are: why didn't he choose three PCUs passing the stop-line during 6 seconds (2 seconds /PCU)? Did the study give the same results if he would have chosen the initial equivalent factors different from as shown in Table 18? And of course, if he chooses the values in Table 18 for saturated conditions, his results after regression analysis will be close to these values. Finally, it can be concluded that the basis for determining the saturated condition for the data collection is unreliable.

Therefore, the results of saturation flow rate according to Chu Cong Minh's methodology cannot be applied for MDCs.

5.1.2.3. Saturation flow using Motorcycle Unit (MCU)

The second research named ''Saturation flow and vehicle equivalent factors in traffic dominated by motorcycles'' of Hien Nguyen and Frank Montgomery, which was published by TRB in 2007 shall be discussed. This research was then developed by the same authors with the title ''Different models of saturation flow in traffic dominated by motorcycles'' and published by the

Institute for Transport Studies of the University of Leeds in 2007. This research is the doctoral dissertation of Hien Nguyen, too.

This research also used the regression analysis, and collected data at twelve approaches in Hanoi at which road gradient is less than 1%, approach width varies from 3.9 m to 13 m, the traffic stream was composed of private cars, light van, minibus, bus, coach, motorcycle and bicycle, but motorcycle proportion is from 80 % to 95 %. In this traffic condition, the authors decided to use the concept of ‘’homogeneous motorcycle saturation flow rate’’, and other vehicles are converted into motorcycle unit (MCU).

According to the Road Note 34 method (Webster, 1963), when observing the saturation flow rate the number of vehicles is recorded in every consecutive 6 seconds of the green time. However, Hien Nguyen and Frank Montgomery decided to count in every consecutive 4 seconds in case traffic dominated by motorcycles because they found that a period of 4 s was found to be more useful, and the potential variation of saturation flow and MCU value during the green time could be explored more easily.

Regarding the regression function, at first, they assumed that S is the homogeneous motorcycle saturation flow rate during a certain period of time T (for example, 4 s). Now, if M motorcycles are taken out and replaced by N_c passenger cars, then the saturation flow will be (N_{mc} = S-M) remaining motorcycles and N_c passenger cars. If calling MCU is equivalent factor of one passenger car into motorcycles, then:

$$S = N_{mc} + MCU*N_c \qquad (8)$$

$$\Rightarrow \quad N_{mc} = S - MCU*N_c \qquad (9)$$

Assuming that MCU may vary linearly depending on the number of cars N_c, therefore:

$$MCU = m + n*N_c \qquad (10)$$

Substituting (10) in (9), gives:

$$N_{mc} = S - m*N_c - n*N_c^2 \qquad (11)$$

Hien Nguyen and Frank Montgomery proposed a model to estimate the homogeneous motorcycle saturation depending on the approach width, turning radii, proportion of turning motorcycles, and the position of the time period within the saturated green time as follows:

$$S = a + b*(w-3.5) + c*\frac{P_{rt}}{R_{rt}} + d*\frac{P_{lt}}{R_{lt}} + \delta_3*P_3 + \delta_4*P_4 + \delta_5*P_5 + \delta_6*P_6 + \delta_{>6}*P_{>6} + \varepsilon \qquad (12)$$

Where: w, P_{rt}, P_{lt}, R_{rt}, R_{lt} = approach width, proportion of right and left turning motorcycles, right and left turning radii,

a, b, c, d = coefficients of the independent variables,

$P_3 \div P_6$ = dummy variables representing periods 3, 4, 5, 6. P_i = 1 for the data of the period i, P_i = 0 for other periods,

$P_{>6}$ represents all periods after period 6,

$\delta_3 \div \delta_6$, $\delta_{>6}$ = coefficients of the dummy variables,

ε = the error term.

Substituting (12) in (11), then using regression analysis with five models as follows:

Model 1: The traffic stream contains only motorcycles and passenger cars, in which passenger cars drive straight-on only.

Model 2: Similar to model 1, but passenger cars can turn right or left.

Model 3: Traffic stream contains all types of vehicles, and all vehicles can go straight-on, turn right or left.

Model 4: Based on the model 2 with longer period of green time counting.

Model 5: Based on the model 3 with longer period of green time counting.

Note that model 4 and model 5 are developed respectively from the model 2 and model 3 with the longer periods of the saturated green time. They counted traffic volume based on the green of the cycle time, then converting the data based on 4 seconds by dividing their own period length and multiplying by 4. Then, they achieved the results.

The results of the regression analysis show that the coefficients of N^2 in formula (11) were insignificant, and all equations were shown to be accurate with R^2 value varying from 76% to 86%. The summarized results are shown in Table 21 and Table 22 as follows:

Table 21: Saturation flow and effects of factors at different traffic combinations

	Model 1	Period-based		Cycle-base	
		Model 2	Model 3	Model 4	Model 5
Saturation flow (MCU / 4s)	12.08	12.44	12.75	12.32	12.28
Period 3		-0.44	-0.48		
Period 4		-0.82	-0.86		
Period 5		-1.02	-1.02		
Period 6		-0.81	-0.73		
Periods >6		-0.68	-0.54		
w-3.5	2.13	2.03	2.04	2.12	2.15
P_{rt}/R_{rt}	47.12	37.23	41.66	66.68	62.60
P_{lt}/R_{lt}	36.15	40.46	40.59	38.26	34.05

(Hien Nguyen and Frank Montgomery, 2007)

Table 22: Comparison of MCU values at different traffic combinations

MCU value	Period-based			Cycle-based	
	Model 1	Model 2	Model 3	Model 4	Model 5
Straight-on car	3.67	3.04	3.19	4.58	3.81
Right turning car		4.61	4.56	4.69	5.70
Left turning car		4.63	4.88	5.55	5.81
Straight-on van			5.01		4.82
Right turning van			6.82		8.56
Left turning van			6.44		6.67
Straight-on bus			8.43		7.96
Right turning bus			8.73		9.07
Left turning bus			9.40		10.21

(Hien Nguyen and Frank Montgomery, 2007)

From Table 21, it can be seen that if the lane width is 3.5 m wide, the standard motorcycle saturation flow can be set approximately 12 MCUs per 4 seconds (approximately 11.000 MCU/h) and the saturation flow increases by more than 2 motorcycles in every 4 seconds (approximately 2.000 MCU/h) for each 1.0 m approach width increasing.

Throughout the methodology and the results of Hien Nguyen and Frank Montgomery, it can be seen that the concept of homogeneous motorcycle saturation flow is quite acceptable in case of a high proportion of motorcycles in traffic streams, and other types of vehicle should be converted into motorcycle unit (MCU) by the respective equivalent factors shown in Table 22.

Furthermore, all assumptions of this methodology were suitable and close to the reality, and they did not show any suspicion. However, there is a limitation of this research, that is, it covered only the range of lane width from 3.5 m to 13 m. It did not give the results with the lane width of 3.0 m and 2.75 m, which are used very often in urban intersection design. However, from the saturation flow result of 11.000 MCU/h for the lane width of 3.5 m, and the saturation flow increases 2000 MCU/h for each 1.0 m, it can be assumed that the saturation flow is 10.000 MCU/h for the lane width of 3.0 m, 9.500 MCU/h for the lane width of 2.75 m, and 13000 MCU/h for the lane width of 4.5 m.

5.1.2.4. Concept of "fictitious saturation flow"

The main idea of this concept is to calculate the saturation flows for the lanes of the intersection layouts that were proposed in chapter 4.

Case 1: Layout of the approach as in Figure 46 and Figure 47

The first layout of approach is illustrated in Figure 46. Then, the traffic flow is like Figure 47. Here, the green time of the phase is divided into two parts, the first one is used for motorcycles ahead, and the second one is used for cars following.

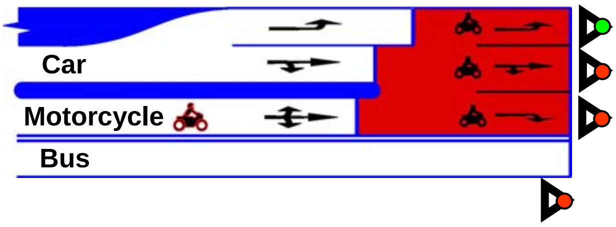


Figure 46: Approach at intersection (case 1)

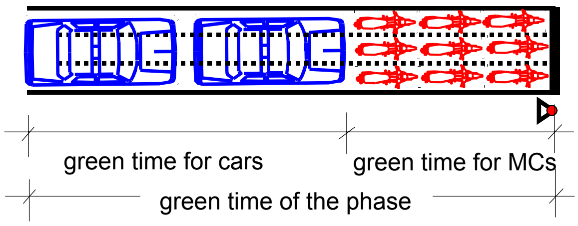


Figure 47: Traffic flow at traffic signals

Figure 48 shows how the traffic flow is operated:

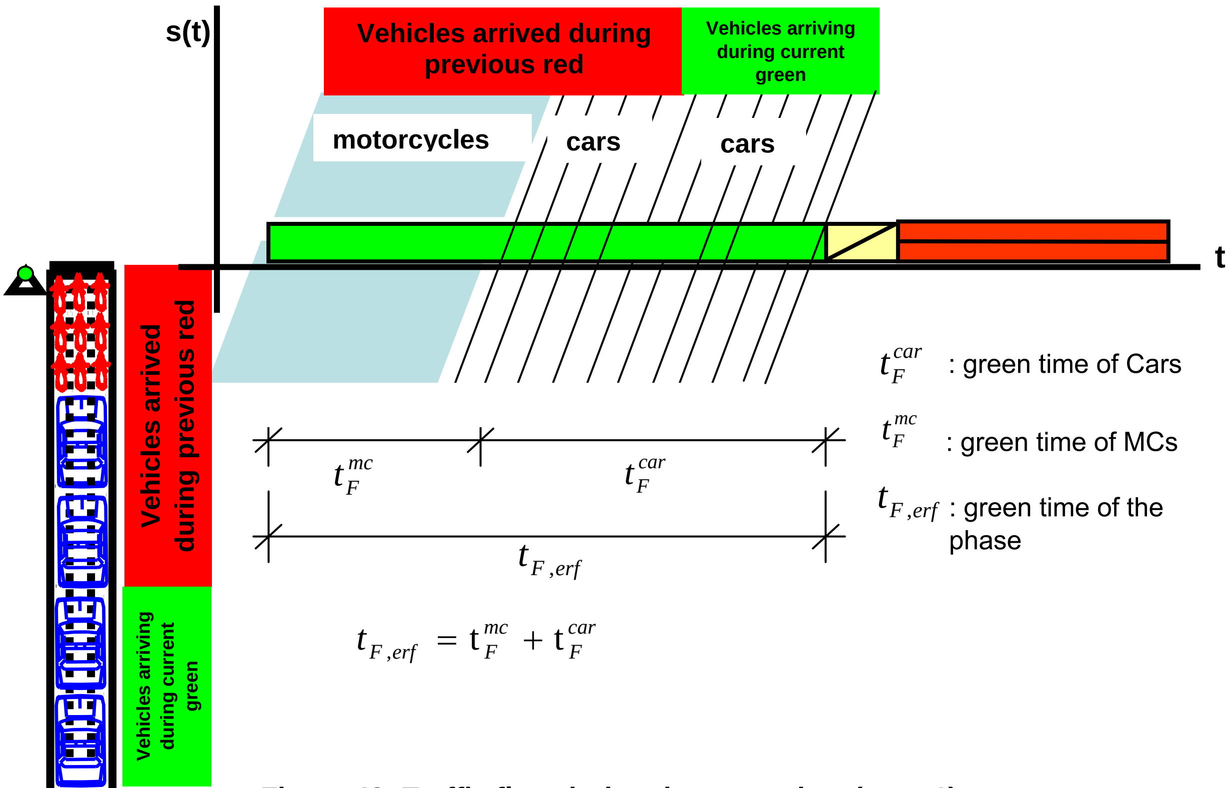


Figure 48: Traffic flow during the green time (case 1)

It is easy to recognize that the saturation flow rate of this traffic flow model will depend on the homogeneous motorcycle saturation flow and the homogeneous car saturation flow as well as on traffic volume of motorcycle and car traffic. Different proportions of motorcycles in this traffic model will give different values of the saturation flow rate. The methodology of the concept "fictitious saturation flow" is: ***during the green time, the number of fictitious vehicles is equal to the real number of cars and motorcycles passing the stop-line.***

It is assumed that:

- q^{mc} and q^{car} is traffic volume of motorcycle traffic and car traffic, respectively [veh/s].
- q_S^{mc} and t_F^{mc} is the homogeneous saturation flow rate [veh/s], and the green time [s] of motorcycles, respectively.
- q_S^{car} and t_F^{car} is the homogeneous saturation flow rate [veh/s], and the green time [s] of cars, respectively.
- q_S and $t_F = t_F^{mc} + t_F^{car}$ is the saturation rate [veh/s], and the green time [s] of the "fictitious flow".

During the green time of the phase, the number of vehicles passing the stop-line is:

$$t_F^{mc}.q_S^{mc} + t_F^{car}.q_S^{car} = t_F.q_S$$

$$\Rightarrow \qquad q_S = \frac{t_F^{mc}.q_S^{mc} + t_F^{car}.q_S^{car}}{t_F} = \frac{t_F^{mc}}{t_F}.q_S^{mc} + \frac{t_F^{car}}{t_F}.q_S^{car} \quad \text{[veh/s]} \qquad (13)$$

The relationship between t_F^{mc} and t_F^{car} is determined by the following system of equations:

$$\begin{cases} t_F = t_F^{mc} + t_F^{car} \\ t_F^{mc} : t_F^{car} = \dfrac{q^{mc}}{q_S^{mc}} : \dfrac{q^{car}}{q_S^{car}} \end{cases} \Rightarrow \begin{cases} \dfrac{t_F^{mc}}{t_F} = \dfrac{1}{1+\dfrac{q^{car}.q_S^{mc}}{q^{mc}.q_S^{car}}} = \dfrac{q^{mc}.q_S^{car}}{q^{mc}.q_S^{car} + q^{car}.q_S^{mc}} \\ \dfrac{t_F^{car}}{t_F} = \dfrac{1}{1+\dfrac{q^{mc}.q_S^{car}}{q^{car}.q_S^{mc}}} = \dfrac{q^{car}.q_S^{mc}}{q^{car}.q_S^{mc} + q^{mc}.q_S^{car}} \end{cases} \qquad (14)$$

Substituting formula (14) into formula (13), gives:

$$q_s = \frac{q^{mc}.q_S^{car}.q_S^{mc}}{q^{mc}.q_S^{car} + q^{car}.q_S^{mc}} + \frac{q^{car}.q_S^{mc}.q_S^{car}}{q^{car}.q_S^{mc} + q^{mc}.q_S^{car}} = \frac{q^{mc}}{\dfrac{q^{mc}}{q_S^{mc}} + \dfrac{q^{car}}{q_S^{car}}} + \frac{q^{car}}{\dfrac{q^{mc}}{q_S^{mc}} + \dfrac{q^{car}}{q_S^{car}}}$$

$$\Rightarrow \qquad q_S = \frac{q^{car} + q^{mc}}{\dfrac{q^{mc}}{q_S^{mc}} + \dfrac{q^{car}}{q_S^{car}}} \quad \text{[veh/s]} \qquad (15)$$

Therefore, formula (15) is used to calculate the fictitious saturation flow according to the traffic model as shown in Figure 46.

Case 2: Layout of the approach as in Figure 49 and Figure 50

In the second case, the green time of the phase is divided into three parts: the first one is for motorcycles, the second one is for cars, and the third one is for mixed traffic (see Figure 49, Figure 50, and Figure 51).

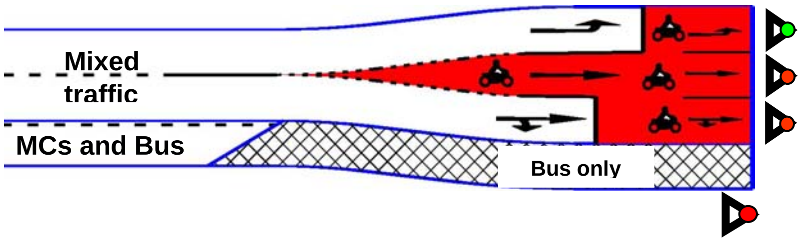


Figure 49: Approach at intersection (case 2)

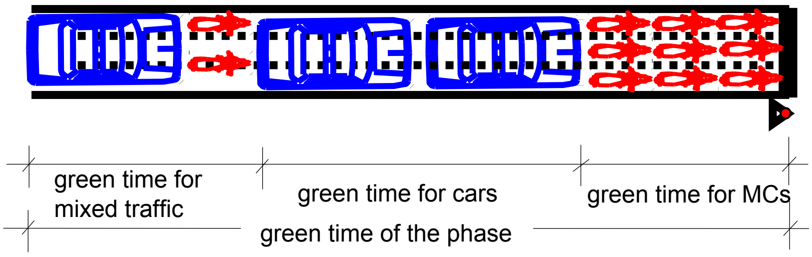


Figure 50: Traffic flow at traffic signals (case 2)

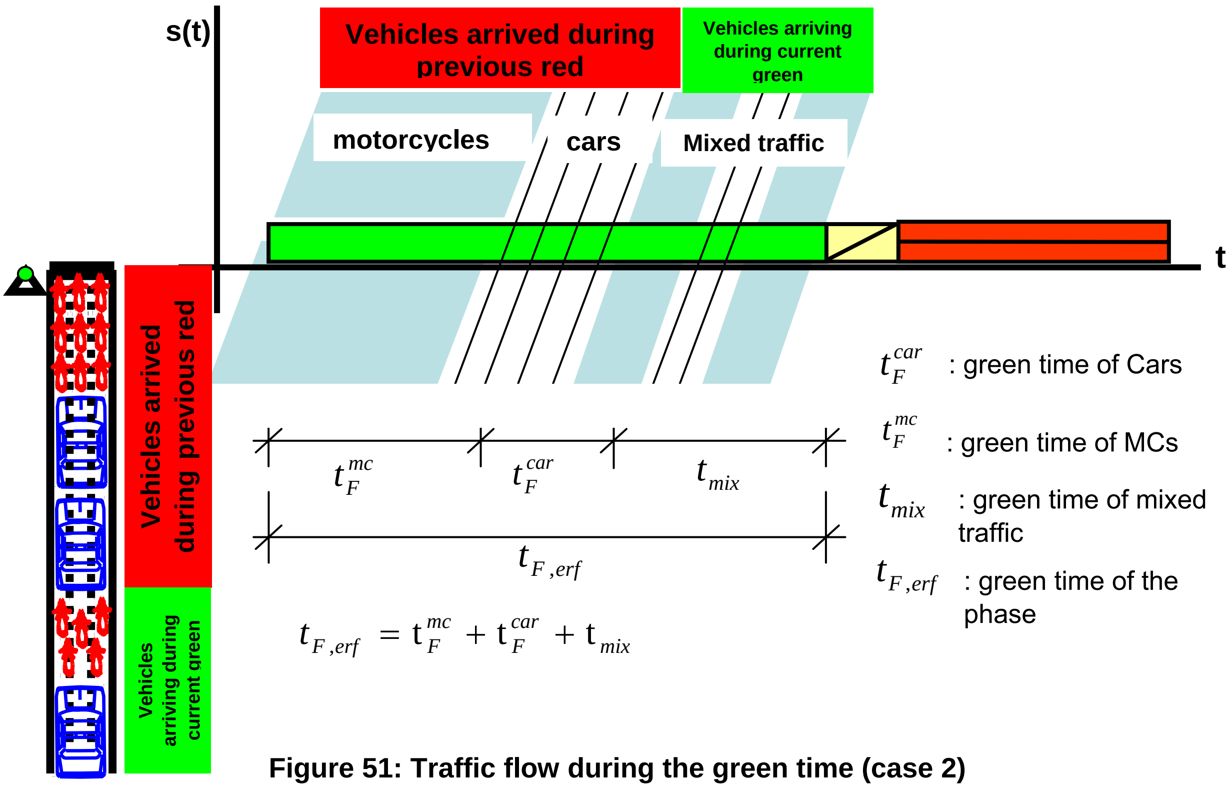


Figure 51: Traffic flow during the green time (case 2)

In this case, the fictitious saturation flow rate is impaired comparing to **case 1** due to mixed traffic operated during the third part of the green time. The longer green time for mixed traffic is, the lower traffic saturation flow will be. Therefore, the saturation flow in formula (15) is adjusted by an adjustment factor f < 1 as follows:

$$q_S = f.\frac{q^{car} + q^{mc}}{\frac{q^{mc}}{q_S^{mc}} + \frac{q^{car}}{q_S^{car}}} \quad \text{[veh/s]} \qquad (16)$$

Where: f is the adjustment factor for the saturation flow due to a part of the green time is used for mixed traffic (f < 1)

The adjustment factor f can be determined by doing a survey with the intersection layout as shown in Figure 49.

$$f = \frac{\text{the real number of vehicles by investigation}}{\text{saturation flow according to formula (15)}} \qquad (17)$$

However, in this study, it can be assumed that f = 0.9 ÷ 0.95 for calculating the saturation flow because only one part of the green time for mixed traffic is used. Therefore, the saturation flow is not so much impaired.

Case 3: Layout of the approach as in Figure 52 and Figure 53

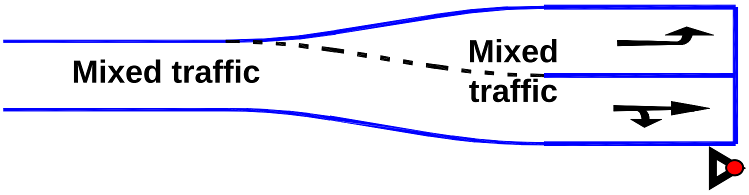


Figure 52: Approach at intersection (case 3)

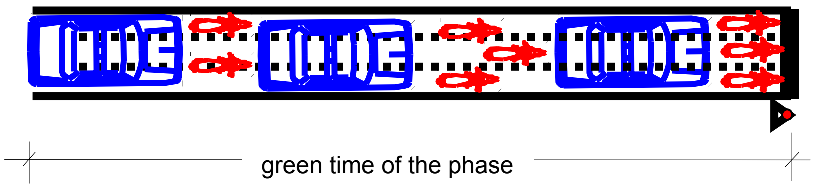


Figure 53: Traffic flow at traffic signals (case 3)

In this case, the fictitious saturation flow is even much more impaired because the whole green time is used for mixed traffic. The methodology for determining the saturation flow in this case is similar to that of **case 2**. However, the adjustment factor is much lower. In this study, it is assumed that f = 0.8 ÷ 0.9 for operating mixed traffic on lanes.

5.1.3. Conclusions

From the analyses above, the research results of Hien Nguyen and Frank Montgomery can be applied to the homogeneous motorcycle saturation flow. For the homogeneous car saturation flow, the procedure of the German Highway Capacity Manual HBS (FGSV, 2001) can be applied. Then, the saturation flow will be determined according to the layout of the traffic flow as follows:

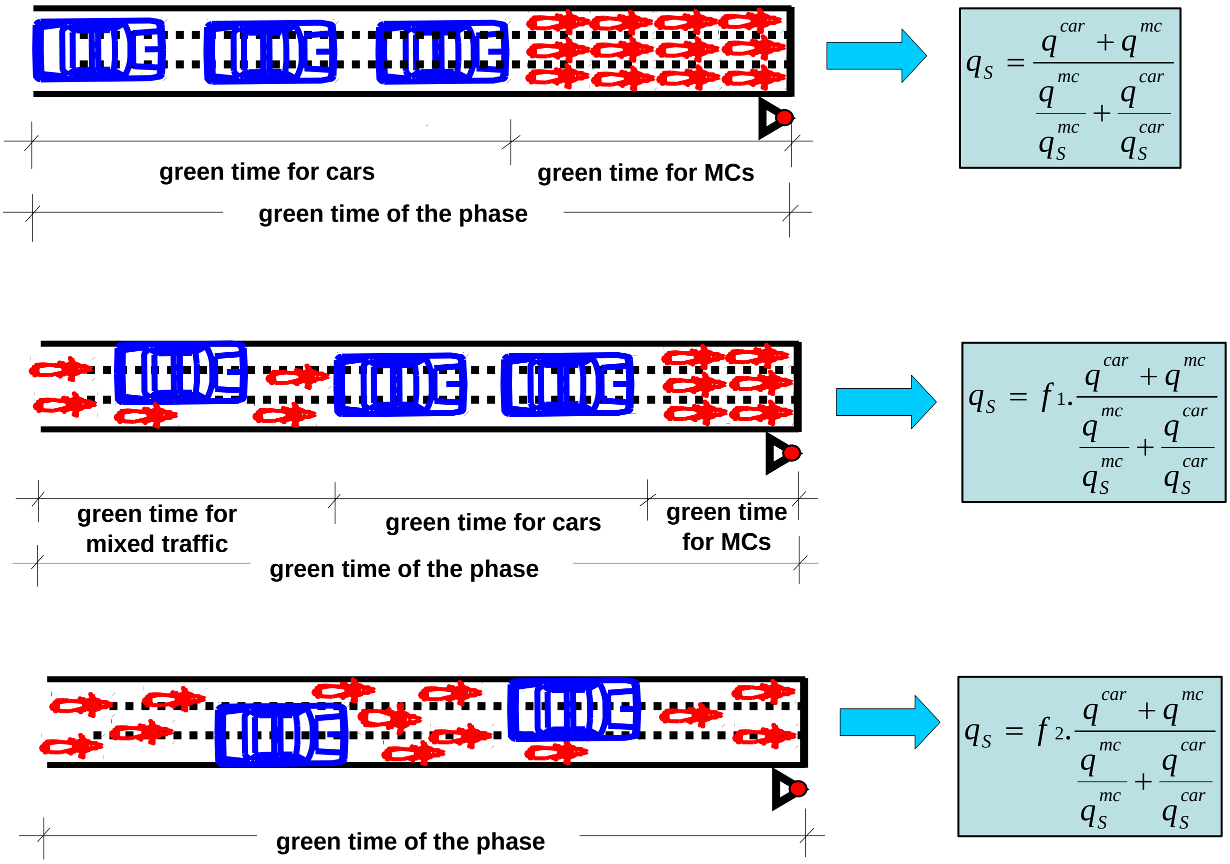


Figure 54: Determination of the saturation flow

Where: f: adjustment factor for the saturation flow due to a part of the green time used for mixed traffic.

f = 1 in case 1 (proved)

f = f_1 = 0.9 ÷ 0.95 in case 2 (recommended)

f = f_2 = 0.8 ÷ 0.90 in case 3 (recommended)

q^{mc}, q^{car}: traffic volume of motorcycles and cars, respectively.

q_S^{mc} ; q_S^{car} : homogeneous saturation flow of motorcycles and cars, respectively.

5.2. Cycle time

5.2.1. General

Up to now, there are several methods to determine the cycle time for an isolated intersection as well as for a road network. To determine the cycle time for an isolated intersection, the following methods can be considered: Greenshields (1947), Pavel (1974), Mäcke (1983), Richtlinien für Lichtsignalanlagen (FGSV, 2009), Road Research Laboratory (Webster, 1957), Highway Capacity Manual (TRB, 2000), Handbuch für die Bemessung von Straßenverkehrsanlagen HBS (FGSV, 2001). Regarding the cycle time for a road network, the method of Boltze (1988) can be mentioned. All these methods were researched for the traffic flows composed of cars and heavy vehicles.

Richtlinien für Lichtsignalanlagen (FGSV, 2009) introduced two methods to determine the cycle time: the minimum necessary cycle time and the optimal-delay cycle time. In this study, these two methods will be analysed under the traffic condition dominated by motorcycles in order to answer the question how far these methods can be applied to MDCs. In the following analyses, the symbols are kept unchanged in each method (for example: the saturation flow according to Webster named S, but according to FGSV named q_S; the cycle time according to Webster named c, but according to FGSV named t_U; etc.).

5.2.2. Optimal-delay cycle time

5.2.2.1. Method of Webster

This method was developed by Webster (1957), and it can be summarized by three steps as follows:

Step 1: Estimating the average delay per vehicle as follows:

$$d = \frac{c(1-\lambda)^{2}}{2(1-\lambda x)} + \frac{x^{2}}{2q(1-x)} - 0.65\left(\frac{c}{q^{2}}\right)^{\frac{1}{3}} x^{(2+5\lambda)} \ldots.. \qquad (18)$$

Where: d = average delay per vehicle on the particular arm of the intersection [seconds]

c = cycle time [seconds]

λ = ratio between effective green time g and cycle time c ($\lambda = \frac{g}{c}$)

q = traffic flow volume [vehicles per second]

s = saturation flow [vehicle per second]

x = degree of saturation ($x = \frac{q}{\lambda . s}$)

Step 2: Determining the total delay for the whole intersection:

$$D = \sum (\text{average delay per vehicle}) \text{ x flow}$$

Step 3: Solving the derivative equation of D according to the cycle time c

$$\frac{dD}{dc} = 0 \Rightarrow C_o = F.\frac{2.L}{1-Y} (\textbf{\textit{Theory}}) \Rightarrow C_o = \frac{1.5L+5}{1-Y} (\textbf{\textit{Practical determination}}) \qquad (19)$$

Where: C_o = optimal cycle time; L = total lost time per cycle; $Y = \sum y_i = \sum \frac{q_i}{s_i}$; F = an unknown quantity; q_i = traffic volume of the critical lane in phase i; s_i = saturation flow of that critical lane.

According to the three steps above, the last two steps are complicated but they are only steps to solve merely mathematical equations, in which Webster used a very important result found from the diagram of formula (18) as presented in Figure 55. This result is that the optimal cycle time was approximately equal to twice the minimum cycle time, and he did experiments to have formula (19).

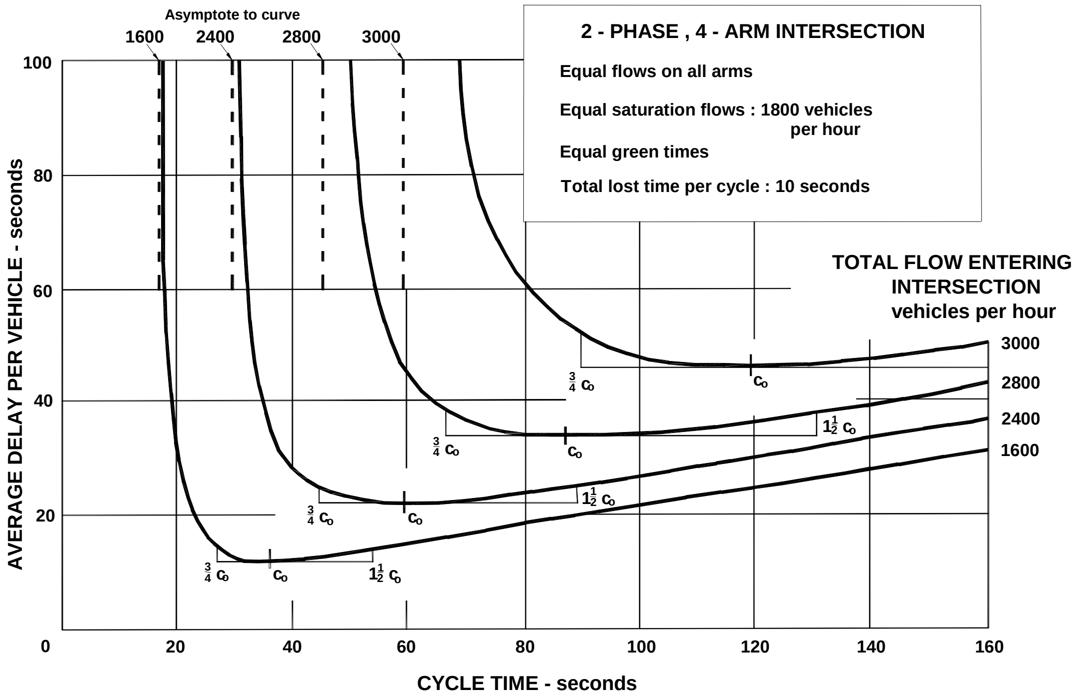


Figure 55: Effect on delay of variation of the cycle length

(Webster, 1957)

Thus, the substantial difference between motorcycle traffic and car traffic lies in the first step, in which traffic volume q, saturation flow s play important roles. For example, in homogeneous motorcycle traffic, the number of vehicles arriving at the intersection is usually high, but its saturation flow is also high comparing with car traffic. Therefore, the average delay according to formula (18) is changed, but how much it has been changed is a difficult question that needs to be answered. To deal with this problem, the principles having formed formula (18) must be analysed, then they will be considered to apply to different traffic models that were presented in chapter 4.

In order to achieve equation (18), Webster (1957) used the simulation method with the Pilot Model Machine ACE (Automatic Computing Engine) to reproduce traffic events on roads in the Laboratory with an assumption that vehicles arrive randomly at the intersection. The first two terms of equation (18) resulting from this simulation have a theoretical meaning, but the last term is purely empirical (the empirical correction term) and is usually from 5 to 15 % of the delay d, and it was usually ignored. The first term is the expression for the delay when traffic can be considered to be arriving at a uniform rate. The second term makes some allowances for the random nature of the arrivals. At low traffic flow, the random feature is less significant, therefore the result of d is close to the value of the first term (see Figure 56).

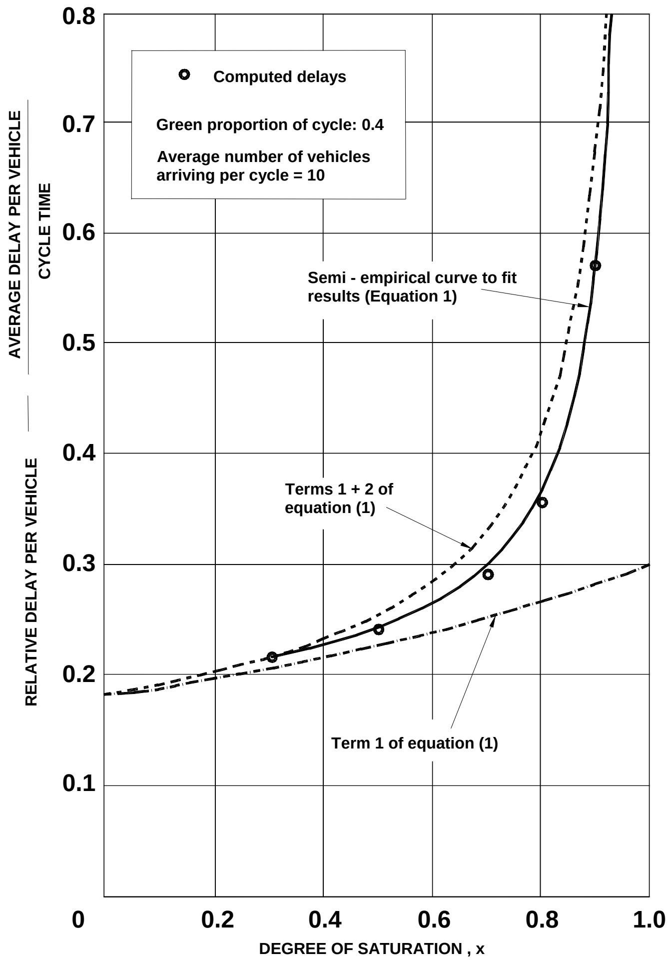


Figure 56: Typical fixed-time delay curve

(Webster, 1957)

According to Webster (1957), the Pilot ACE receives all instructions and data on Hollerith cards, and traffic was, therefore, prepared in this form by punching holes in the cards to represent the arrival of vehicles at the intersection. In this investigation, time is the fundamental variable and a quantized time scale was obtained by considering each position on the card as one unit of time. Each card of the pack extends the time scale by a given number of units. A hole punched in a particular position on a particular card means that a vehicle has arrived at the intersection in that unit of time. The shorter the unit of time, the nearer the time scale approaches a continuous one.

Traffic was generated using a sequence of random numbers (from published tables) to decide whether a vehicle arrives in each successive unit.

An example from Webster (1957): It is desired to generate traffic of 720 vehicles/hour. The unit of time on the cards is chosen to be ½ second. The average rate of arrival is, therefore, 0.1 vehicle per unit of time. A number is taken from a table of random numbers for each unit of time and interpreted as a decimal fraction; if it is less than 0.1, an arrival is assumed in the corresponding unit of time; if it is greater than 0.1, there is no arrival in that unit.

Since this method does not allow two or more vehicles arriving in the same unit of time, the unit should be chosen as small as practicable so that the difference towards reality is not important.

The computer is programmed to:

(a) Interpret the traffic cards.

(b) Act as the traffic signal by timing off alternate red and green periods.

(c) Keep a count of the queue, adding one for each arrival and subtracting one at constant intervals during the green time to represent the discharge of vehicles, until the queue becomes zero.

(d) Compute the total delay experienced by all vehicles (see below).

The total delay is computed by adding the number of vehicles in the queue into a storage counter every unit of time, e.g. if, in successive units of time, the number of vehicles in the queue is 2, 3, 3, 2, 3 and the unit of time is ½ second, then at the end of the first half second the total delay experienced is 2 x ½ vehicle-seconds, at the end of the second unit of time the total delay is (2 x ½ + 3 x ½) vehicle-seconds, and at the end of the fifth unit of time the total delay is (2+3+3+2+3) ½ vehicle-seconds. If n is the queue during any unit of time and u is the value of the unit then the total delay is $u.\sum n$ and the average delay per vehicle is obtained by dividing this by the number of arrivals. Because of the high operating speed of the ACE the delay to about 10.000 vehicles can be computed in 5 minutes, i.e. for a flow of 1000 vehicles/hour, 10 hours' traffic can be analysed in 5 minutes.

Going back to MDCs, according to this method, it does not matter if the arrival is whichever type of vehicle because each arrival is represented by one hole on the card. In case of motorcycle traffic, the number of vehicles arriving is usually high, therefore the unit of time must be considered to be as small as practicable in order to avoid more than one motorcycle arriving at the same unit of time. The substance of this method is in item (c): ''Keep a count of the queue, ***adding one for each arrival and subtracting one at constant intervals during the green time to represent the discharge of vehicles***, until the queue becomes zero''. It means that during the green time the traffic flow is discharging at the saturation flow rate, and the computer program subtracts automatically one vehicle at constant intervals (saturation headway) during the simulative green time, this is represented by the saturation flow s in formula (18). It can be concluded that the total delay in the simulation depends on the number of arrivals and the saturation flow rate. From these basic principles, the traffic models at traffic signals in MDCs will be analysed to find out the applicability of this method.

5.2.2.2. Application to MDCs

Before applying the Webster's method for determining the optimal-delay cycle time to MDCs, it is assumed that the practical coefficients in the numerator of formula (19) are kept unchanged. Hereby, only the denominator of this formula including traffic volume and the saturation flow rate is considered.

In this case, under mixed traffic condition, traffic volume arriving is $q = q^{car} + q^{mc}$ (in which q^{car}, q^{mc} is traffic volume of cars and motorcycles, respectively). The simulation method of the ACE machine will become more complicated because the saturation rates of motorcycle and car are different, the computer program could not recognize when are motorcycles and when are cars to subtract correctly according to their saturation flow rate during the green time. Therefore, to apply this method, it is necessary to have only one saturation flow rate. To achieve this, there are three solutions as follows:

- ***Converting motorcycles into passenger car unit (PCU) under saturation condition***

  Because the equivalent factor converting from motorcycle into PCU $k_1 < 1$, the number of arrivals (actually, sequence of random numbers putting into the ACE machine) will reduce from $q = q^{car} + q^{mc}$ to $q = q^{car} + k_1.q^{mc}$ and the saturation flow rate using in this simulation is s^{car} (car saturation flow). It seems that the total delay will be lower comparing with reality because the number of arrivals was reduced and the random feature of the traffic flow, therefore, has been less significant. Consequence, there will be an unexpected result. However, if this is accepted, the optimal cycle time is determined by the following equation:

$$C_o = \frac{1.5L + 5}{1 - \sum \frac{q_i^{car} + k_1.q_i^{mc}}{s_i^{car}}} \qquad (20)$$

- ***Converting cars into motorcycle unit under saturation condition***

  On the contrary, if cars are converted into the motorcycle unit (MCU) by an equivalent factor $k_2 > 1$, the number of arrivals putting into ACE machine will increase from $q = q^{car} + q^{mc}$ to $q = k_2.q^{car} + q^{mc}$, the saturation flow rate using in this simulation is s^{mc} (motorcycle saturation flow) and the total delay seems to be increased comparing with reality. The optimal cycle time is determined as follows:

$$C_o = \frac{1.5L + 5}{1 - \sum \frac{k_2.q_i^{car} + q_i^{mc}}{s_i^{mc}}} \qquad (21)$$

- ***Keeping the same number of arrivals $q = q^{car} + q^{mc}$, but using a fictitious saturation flow rate***

  As mentioned in the section of saturation flow, the concept of the fictitious saturation flow is: "***during the green time, the number of fictitious vehicles is equal to the real number of cars and motorcycles passing the stop line***". And the formula for calculating the fictitious saturation flow is (see section of Saturation flow):

$$s = f.\frac{q^{car} + q^{mc}}{\frac{q^{mc}}{s^{mc}} + \frac{q^{car}}{s^{car}}} \tag{22}$$

Therefore, substituting formula (22) into formula (19), the optimal delay cycle time is:

$$C_o = \frac{1.5L + 5}{1 - \sum \frac{1}{f}.\left(\frac{q^{mc}}{s^{mc}} + \frac{q^{car}}{s^{car}}\right)} \tag{23}$$

Where: q^{mc}, q^{car} = traffic volume of motorcycles and cars,

s^{mc}, s^{car} = saturation flow of motorcycles and cars,

f = adjustment factor depending on the traffic models at traffic signals as shown in Figure 54.

From three solutions above, it can be concluded that the last one is suitable for the Webster's method because it keeps the actual number of traffic volume as the input data.

5.2.3. Minimum necessary cycle time

5.2.3.1. Methodology

This method based on the assumption that: ''during the cycle time, on the critical lane of the phase, the number of vehicles arriving is equal to the number of vehicle releasing''. The critical lane of the phase is the lane, which has the highest traffic volume.

The average number of vehicles arriving during the cycle time is determined as follows:

$$\frac{q_{FS,maßg}}{3600}.t_{U,erf} \tag{24}$$

Where: $q_{FS,maßg}$: traffic volume of the critical lane

$t_{U,erf}$: minimum cycle time

The number of vehicles releasing during the green time of the cycle time is determined as follows:

$$\frac{q_{zul}}{3600}.t_{F,maßg} \tag{25}$$

Where: $q_{zul} = g.q_S$

q_S: saturation flow

g: degree of saturation (g = 0.8 ÷ 0,9)

$t_{F,maßg}$: green time

Normally, the highest number of vehicles that can release during the green time is $\frac{q_S}{3600}.t_{F,maßg}$.

However, to take a random variation of the traffic flow into account, the saturation flow q_S has to be reduced to q_{zul} by the degree of saturation g.

From (24) and (25), there is:

$$\frac{q_{FS,maßg}}{3600}.t_{U,erf} = \frac{q_{zul}}{3600}.t_{F,maßg} \tag{26}$$

$$\Rightarrow \quad t_{F,maßg} = \frac{q_{FS,maßg}}{q_{zul}}.t_{U,erf} \qquad \Rightarrow \quad \sum_i t_{F,maßg} = t_{U,erf}.\sum_i \frac{q_{FS,maßg}}{q_{zul}} \tag{27}$$

Where: i is the number of phases.

On the other hand, the cycle time is equal to the sum of the green times and the intergreen times. Therefore:

$$t_{U,erf} = \sum_i t_{F,maßg} + \sum_i t_{Z,erf} \qquad \Rightarrow \quad \sum_i t_{F,maßg} = t_{U,erf} - \sum_i t_{Z,erf} \tag{28}$$

Substituting (28) in (27) gives:

$$t_{U,erf} - \sum_i t_{Z,erf} = t_{U,erf}.\sum_i \frac{q_{FS,maßg}}{q_{zul}} \qquad \Rightarrow \quad t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1 - \sum_i \frac{q_{FS,maßg}}{q_{zul}}} \tag{29}$$

Thus, formula (29) is used to determine the minimum necessary cycle time.

Furthermore, formula (29) is basically the same as formula (30) for determining the cycle time in Highway Capacity Manual HCM 2000 (note that, in formula (29), $q_{zul} = g.q_S$):

$$C = \frac{L.X_c}{\left[X_c - \sum_i \left(\frac{v}{s}\right)_{ci}\right]} \quad \Leftrightarrow \quad C = \frac{L}{\left[1 - \sum_i \left(\frac{v}{s.X_c}\right)_{ci}\right]} \tag{30}$$

Where L = the total lost time

v = traffic flow

s = saturation flow

X_c = critical v/c ratio for the intersection

5.2.3.2. Application to MDCs

Case 1: Traffic model as shown in Figure 46 and Figure 47

In this model, as shown in Figure 46 and Figure 47, the green time is divided into two parts, one is for motorcycles ahead, the other is for successive cars.

The number of vehicles arriving at the intersection includes motorcycles and cars. According to the assumption: ''the number of vehicles arriving is equal to the number of vehicles releasing'', it means that the number of cars arriving is equal to the number of cars releasing, and the number of motorcycles arriving is equal to the number of motorcycles releasing. Thus, there are following equations:

$$\begin{cases} \dfrac{q^{mc}_{FS,maßg}}{3600}.t_{U,erf} = \dfrac{q^{mc}_{zul}}{3600}.t^{mc}_{F} \\ \\ \dfrac{q^{car}_{FS,maßg}}{3600}.t_{U,erf} = \dfrac{q^{car}_{zul}}{3600}.t^{mc}_{F} \end{cases} \Rightarrow \begin{cases} t^{mc}_{F} = \dfrac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}}.t_{U,erf} \\ \\ t^{car}_{F} = \dfrac{q^{car}_{FS,maßg}}{q^{car}_{zul}}.t_{U,erf} \end{cases}$$

$$\Rightarrow \qquad t_{F,maßg} = t^{mc}_{F} + t^{car}_{F} = \left(\frac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \frac{q^{car}_{FS,maßg}}{q^{car}_{zul}} \right).t_{U,erf}$$

Therefore, the total green time of all phases is:

$$\sum_i t_{F,maßg} = t_{U,erf} \cdot \sum_i \left(\frac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \frac{q^{car}_{FS,maßg}}{q^{car}_{zul}} \right) \qquad (31)$$

Substituting formula (28) in (31), gives:

$$t_{U,erf} - \sum_i t_{Z,erf} = t_{U,erf} \cdot \sum_i \left(\frac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \frac{q^{car}_{FS,maßg}}{q^{car}_{zul}} \right)$$

$$\Rightarrow \qquad t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1 - \sum_i \left(\dfrac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \dfrac{q^{car}_{FS,maßg}}{q^{car}_{zul}} \right)} \qquad (32)$$

Thus, formula (32) is used to determine the minimum necessary green time. Furthermore, it is seen that the denominator of this formula is substantially similar to the denominator of formula (23) by the application of Webster's method. Hereby, it is proved that the above methodologies applying to MDCs are true.

Note that if the concept of the fictitious saturation flow is applied directly to formula (29), then the same result as formula (32) is gained.

Case 2: Traffic model as shown in Figure 49 and Figure 50

Applying the concept of the fictitious saturation flow to formula (29), the following formula for calculating the minimum necessary cycle time can be given:

$$t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1 - \sum_i \dfrac{1}{f_1} . \left(\dfrac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \dfrac{q^{car}_{FS,maßg}}{q^{car}_{zul}} \right)} \qquad (33)$$

Where: f_1 = the adjustment factor for the saturation flow due to a part of the green time for mixed traffic, f_1 = 0.90 ÷ 0.95 (recommended).

Case 3: Traffic model as shown in Figure 52 and Figure 53

Similarly, there is the following formula for calculating the minimum necessary cycle time in this case:

$$t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1-\sum_i \frac{1}{f_2}\cdot\left(\frac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \frac{q^{car}_{FS,maßg}}{q^{car}_{zul}}\right)} \qquad (34)$$

Where: $f_2 = 0.8 \div 0.9$ (recommended).

In this case, the whole green time is used for mixed traffic.

5.2.4. Maximum cycle time in MDCs

When deciding the cycle time for operating a signalised intersection, it is necessary to consider all types of road users at the intersection. For example, the longer cycle time will give the higher capacity of motorised traffic because the longer cycle time usually gives the higher green time ratio (t_F / t_U). However, the average waiting time of the road users will become longer, especially for pedestrians.

In practice, the cycle time in Germany is usually not longer than 90 s, in exceptional cases the cycle time may be 120 s. These values are also mentioned in RiLSA (FGSV, 2009) and HBS (FGSV, 2001).

In Japan, the maximum cycle time in the Manual on Traffic Signal Control (JSTE, 2006) is 120 s. In exceptional cases, the cycle time is 180 s.

In Vietnam, where there are many MDCs, it may be necessary to give priority on considering the capacity. However, then it is also necessary to consider the average waiting time. On the other hand, Vietnam cannot take the maximum cycle time from Germany, where usually there are very high levels of the traffic flow quality. Therefore, in this study, the maximum cycle time of 120 s is recommended for MDCs. In exceptional cases, it might be 150 s. In practice, Daewoo intersection in Hanoi is often controlled with the cycle time of 133 seconds in peak hours.

5.2.5. Conclusions

Using the symbols from RiLSA edition 2009, there are following formulas for determining the cycle time:

- The optimal cycle time is determined by the following formula:

$$C_o = \frac{1.5\sum_i t_Z + 5}{1-\sum \frac{1}{f}\cdot\left(\frac{q^{mc}_{FS,maßg}}{q^{mc}_S} + \frac{q^{car}_{FS,maßg}}{q^{car}_S}\right)} \qquad (35)$$

Where: f: adjustment factor for the saturation flow due to a part of the green time used for mixed traffic

$f = 1$	in case 1	(proved)
$f_1 = 0.90 \div 0.95$	in case 2	(recommended)
$f_2 = 0.8 \div 0.9$	in case 3	(recommended)

$q^{mc}_{FS,maßg}$; $q^{car}_{FS,maßg}$: traffic volume of motorcycles and cars, respectively

q^{mc}_{S} ; q^{car}_{S} : saturation flow of motorcycles and cars, respectively

$\sum_i t_Z$: total intergreen time of phases

- The minimum necessary cycle time is determined by the following formula:

$$t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1 - \sum_i \frac{1}{f} \cdot \left(\frac{q^{mc}_{FS,maßg}}{q^{mc}_{zul}} + \frac{q^{car}_{FS,maßg}}{q^{car}_{zul}} \right)} \qquad (36)$$

Where: $q^{mc}_{zul} = g.q^{mc}_{S}$; $q^{car}_{zul} = g.q^{car}_{S}$

g: degree of saturation ($g = 0.8 \div 0.9$).

5.3. Green time

5.3.1. General

In the Traffic Engineering Handbook (ITE, 1950), it is suggested that the least delay to traffic is obtained when the green periods of the phases are in proportion to the corresponding ratios of flow to saturation flow, assuming this ratio to be the same for all arms of the same phase.

This simple rule for the green time division was explained in the Road Research Technical Paper No.39 (Webster, 1957) in which it based on both theory and experiment. From the theory, it was found that the best ratio of the effective green times is achieved when the average delay per vehicle is minimum (see Figure 57 as an example of the relationship between the green time ratio, the average delay per vehicle, and the cycle time).

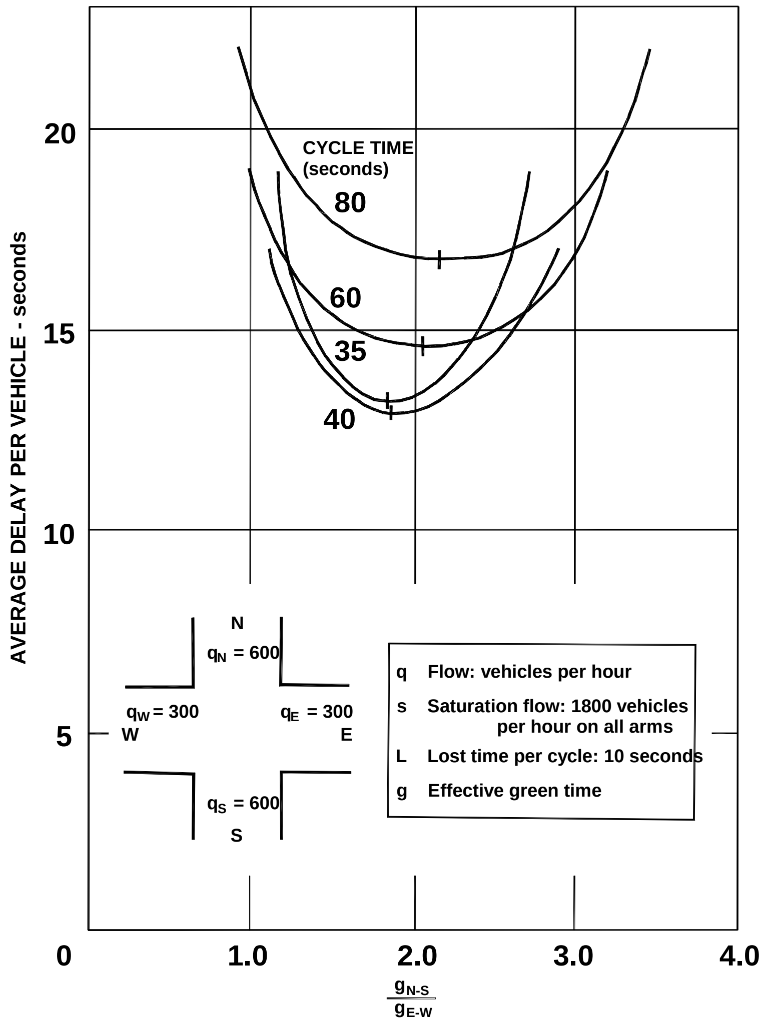


Figure 57: Effect on delay of variation of ratio of green periods

(Webster and Cobbe, 1957)

From the experiment, with a lot of data collected, it was found that the best ratios of the effective green times were approximately in proportion to the ratios of flow to saturation flow.

Therefore, it can be concluded that the conventional rule of the effective green time division based on the condition that the average delay per vehicle is minimum.

In HBS (FGSV, 2001), the optimal cycle time according to Webster was recommended, therefore the green time division is based on the minimum average delay per vehicle as presented above, as well. The green times were calculated as follows:

$$t_{Fi} = \frac{b_{maßg,i}}{B}(t_U - T_Z) \qquad (37)$$

Where t_{Fi} = the green time of the critical lane in phase i,

$b_{maßg,i}$ = ratio of flow to saturation flow of the critical lane in phase i,

$B = \sum_{i=1}^{p} b_{maßg,i}$ (p is the number of phases),

t_U = the optimal delay cycle time ($t_U = \frac{1.5 * T_Z + 5}{1 - B}$),

T_Z = the total intergreen times.

However, there is a question on how the green times should be divided if the cycle time is not calculated based on the optimal delay.

According to RiLSA (FGSV, 2009), if the minimum necessary cycle time is calculated by the following formula:

$$t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1 - \sum_i \frac{q_{FS,maßg}}{q_{zul}}} \qquad (38)$$

Then, the green time is calculated by the assumption that the number of vehicles arriving at the intersection during the cycle time is equal to the number of vehicles releasing during that green time:

$$t_{F,maßg,i} = \frac{q_{FS,maßg,i}}{q_{zul,i}} . t_{U,erf} \qquad (39)$$

Where $t_{U,erf}$ = minimum necessary cycle time,

$q_{FS,maßg,i}$ = traffic flow of the critical lane in phase i,

$\sum_i t_{Z,erf} = T_Z$ (the total necessary intergreen time),

$q_{zul,i} = g.q_{S,i}$,

$q_{S,i}$ = saturation flow of the critical lane in phase i,

g = degree of saturation (g = 0.8 ÷ 0.9).

Formula (38) can be rewritten as follows:

$$t_{U,erf} = \frac{t_{U,erf} - \sum_i t_{Z,erf}}{\sum_i \frac{q_{FS,maßg}}{q_{zul}}} \qquad (40)$$

Setting $b_{zul,i} = \dfrac{q_{FS,maßg,i}}{q_{zul,i}}$; $B_{zul} = \sum_{i=1}^{p} b_{zul,i}$, then substituting (40) in (39), formula (39) becomes:

$$t_{Fi} = \frac{b_{zul,i}}{B_{zul}}(t_{U,erf} - T_Z) \qquad (41)$$

Comparing formula (41) with formula (37), it is seen that the basic difference between HBS 2001 and RiLSA 2009 is that the saturation flow q_S in HBS 2001 is replaced by $(g.q_S)$ in RiLSA 2009, in which g is the degree of saturation. If g is set at the same value at all lanes of the intersection, formula (41) will become formula (37). It means that, in this case, both methods in HBS 2001 and in RiLSA 2009 give the same result.

Furthermore, in HCM 2000 (TRB, 2000), the effective green time is calculated as follows:

$$g_i = \frac{v_i * C}{s_i * X_i} \qquad (42)$$

Where g_i = effective green time

v_i = traffic flow

s_i = saturation flow

C = cycle time

$X_i = \dfrac{v_i * C}{s_i * g_i}$ (called v/c ratio, or ratio of traffic flow to capacity, or degree of saturation, default value suggested is 0.9)

If comparing with RiLSA 2009 ($q_{FS,maßg,i} = v_i$; $q_{zul,i} = g*q_{S,i} = X_i*s_i$; $t_{U,erf} = C$), it has been seen that formula (42) is the same as formula (39). It means that RiLSA 2009 and HCM 2000 have the same methodology for the green time calculation.

5.3.2. Application to MDCs

5.3.2.1. The green time depending on the cycle time

The green time depending on the cycle time is the green time determined based on traffic volume, it is not the minimum green time that is chosen independently from the cycle time calculation.

From the method for determining the minimum necessary cycle time in MDCs, the green time depending on the cycle time is determined as follows:

$$t_{F,maßg,i} = \frac{1}{f_i}.\left(\frac{q^{mc}_{FS,maßg,i}}{q^{mc}_{zul,i}} + \frac{q^{car}_{FS,maßg,i}}{q^{car}_{zul,i}}\right).t_{U,erf} \qquad (43)$$

or

$$t_{F,maßg,i} = \frac{\left(\dfrac{q^{mc}_{FS,maßg,i}}{f_i.q^{mc}_{zul,i}} + \dfrac{q^{car}_{FS,maßg,i}}{f_i.q^{car}_{zul,i}}\right)}{\sum \dfrac{1}{f_i}.\left(\dfrac{q^{mc}_{FS,maßg,i}}{q^{mc}_{zul,i}} + \dfrac{q^{car}_{FS,maßg,i}}{q^{car}_{zul,i}}\right)}.\left(t_{U,erf} - \sum t_{Z,erf}\right) \qquad (44)$$

Where f_i = adjustment factor for saturation flow due to a part of the green time used for mixed traffic

$f = 1$ in case 1 (proved)

$f_1 = 0.90 \div 0.95$ in case 2 (recommended)

$f_2 = 0.80 \div 0.90$ in case 3 (recommended)

$t_{U,erf}$ = minimum necessary cycle time,

$q^{mc}_{FS,maßg,i}$ and $q^{car}_{FS,maßg,i}$ = traffic volume of motorcycles and cars, respectively,

$q^{mc}_{zul,i} = g.q^{mc}_{S,i}$ and $q^{car}_{zul,i} = g.q^{car}_{S,i}$

$q^{mc}_{S,i}$ and $q^{car}_{S,i}$ = saturation flow rate of motorcycles and cars

g = degree of saturation

If the cycle time ($t_{U,gew}$) is chosen longer than the minimum necessary cycle time ($t_{U,erf}$), it is assumed that the total green time contains only the green times depending on the cycle time, then the green time $t_{F,i}$ is determined as formula (44), in which $t_{U,erf}$ is replaced by $t_{U,gew}$. In this case, the actual green time will be longer than the necessary green time. Therefore, the residual green time is used for other aims, for example a co-ordination, or considerations of pedestrian and cycle traffic.

5.3.2.2. The independent green time (the minimum green time)

The independent green time means that this green time is not computed based on the cycle time, but it is chosen independently, for example in case of the minimum green time for motorised traffic.

According to the Road Research Technical Paper No.56 (Webster and Cobbe, 1966), the minimum green time varies between 7 s and 13 s depending on the number of vehicles waiting in a queue length from the detector to the stop-line.

According to RiLSA 2003 (FGSV, 2003), an English version of RiLSA 1992, the minimum green time for motorised traffic is 10 s. But, for main-direction, the value of 15 s is recommended. At low traffic load or in case of traffic-actuated control including green time elongations, the minimum green time can be reduced to 5 s.

According to RiLSA 2009 (FGSV, 2009), the minimum green time is 5 s.

In MDCs, the minimum green time must ensure that the number of motorcycles waiting in the head-start area is completely discharged. Therefore, it depends on the length of the head-start area.

If the lane width is 3 m and the length of the head-start area is 12 m it means that this head-start area can contain theoretically maximum 18 motorcycles (note that the maximum length of a motorcycle is 2 m) as shown in Figure 58 as follows:

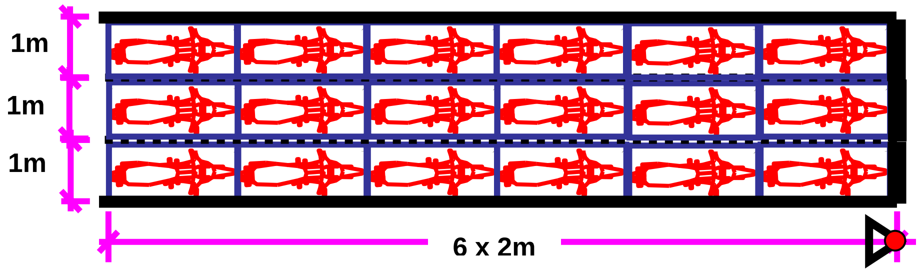


Figure 58: The minimum green time

It is assumed that the start-up lost time for motorcycles is 2.0 s (the same value in HCM 2000). The saturation flow of motorcycles is: 10.000 MCs/h (see section saturation flow of motorcycle). The green time needed to discharge 18 motorcycles is:

$$t_{F,\min} = 2 + 18.\frac{3.600}{10.000} = 8.48 \approx 9 \text{ s}$$

Similarly, if the length of the head-start area is 6 m, then the minimum green time is:

$$t_{F,\min} = 2 + 9.\frac{3.600}{10.000} = 5.24 \approx 6 \text{ s}$$

It is assumed that the length of head-start area is not longer than 12 m (equal to two passenger car units) due to the optical visibility of the car drivers reacting to the signal head; therefore the minimum green time of 9 s (rounded to 10 s) can be taken. This value always ensures that all motorcycles in the head-start area of less than 12 m long can be discharged.

5.4. Transition time

5.4.1. General

At traffic signal systems, when the signal changes from green to red, it is necessary to indicate a transition time (called the amber time) between them to inform moving drivers either to stop safely in front of the stop-line or to pass the stop-line (only for vehicles that do not have enough stopping distance) before the signal turns red. Therefore, the determination of the amber time depends very much on the vehicle‘s movement (speed, deceleration rate), on the location and the psychophysiology of the drivers. In Germany, there is also another kind of the transition time (called the red and amber time), which is used to indicate the signal changing from the red time to the green time.

In Germany, the method to determine the amber time was first developed by Retzko (1966). Then, this research was developed more detailed by Behrendt (1970), and until now this basic method has been seen as the latest one in Germany. The same method was also recommended by ITE (1985). Although the basic methodology for calculating the amber time ***''the drivers are not confronted with the dilemma zone''*** is the same in each country, some values of parameters are different from each other. For example: Germany used V_{zul} (speed limit at traffic signals), but ITE used V_{85} (85^{th} percentile speed); the deceleration rate of vehicle written in the traffic law of Germany StVO (BfV, 2008) is 5.0 m/s^2 (assumed 3.5 m/s^2 when calculating the amber time), but this value in ITE is 3.048 m/s^2 (10 ft/s^2) (ITE, 1985), while the reaction time of the drivers in both countries is 1 second.

To apply the German method to MDCs, firstly this method (Retzko, 1966) must be analysed clearly, then the specific movement characteristics of motorcycles must be taken into account. In the following texts, the sources given to Figure 59, Figure 60, Figure 61, Figure 62, Figure 63, Figure 64, and Figure 65 stem from Boltze (2007), but originally published by Retzko (1966).

5.4.2. German method for determining the transition time

5.4.2.1. Amber time

a. Braking decision

In Figure 59, it is assumed that at the time point t^* the signal starts changing from green to amber, the vehicle is moving at the speed v (m/s) and at the distance $S(t^*)$ from the stop-line. It is obvious that if the distance $S(t^*)$ is equal to or longer than the stopping distance S_H ($S(t^*) \geq S_H$), the vehicle will stop safely in front of the stop-line during the amber time.

The stopping distance S_H of the vehicle includes the reaction distance S_{Re} (the distance driven during the reaction time t_{Re}) and the braking distance S_B (the distance driven while the brake is active until stopping). It is determined by the following formula:

$$S_H = S_{\text{Re}} + S_B = v.t_{\text{Re}} + \frac{v^2}{2.b_v} \qquad (45)$$

Where: b_v is the deceleration rate of the vehicle,

The other parameters were defined in the texts above.

The reaction time t_{Re} was mentioned and researched by Behrendt (1970), and the value of 1 second has been seen as the reaction time.

Before 1988, the minimum deceleration rate b_v written in the StVZO of Germany was 2.5 m/s^2. Nowadays, however, car technology has been developing quickly and getting better. As a result, vehicles have the higher quality and deceleration rate. According to the latest version of the StVZO (BfV, 2008), the minimum deceleration rate is 5.0 m/s^2 written in StVZO § 41 Abs. 4. However, note that this value was given under the condition of a normal braking force, dry and flat pavement surface, and non-longitudinal gradient.

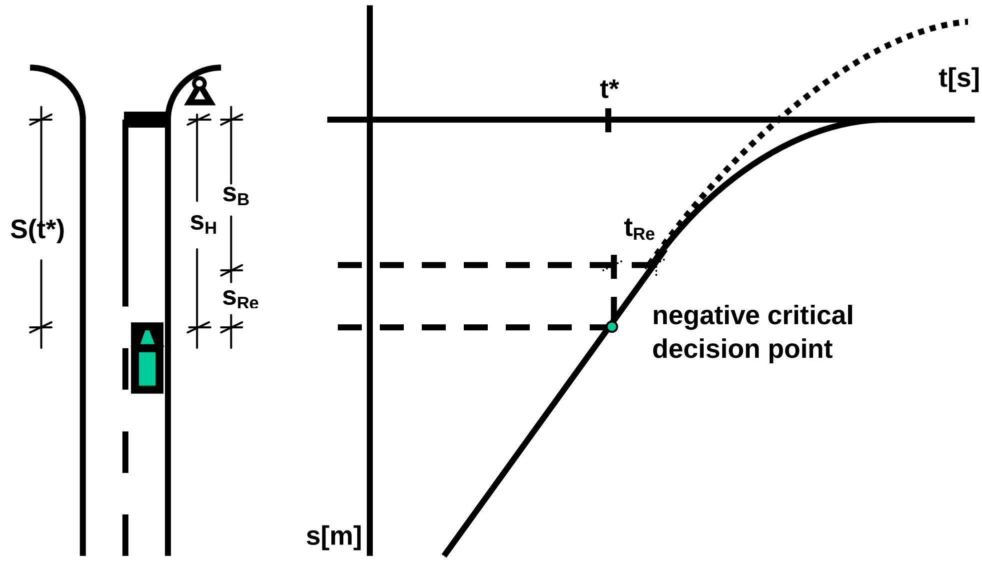


Figure 59: Negative critical decision point

(Boltze, 2007 according to Retzko, 1966)

However, when calculating the amber time at traffic signal systems, the deceleration rate of 3.5 m/s^2 is usually assumed because it refers to a not good pavement surface condition, to the traffic condition, and to the comfort of drivers while braking.

If the speed v and the deceleration rate b_v are given, the **negative critical decision point** is defined as the point at which the stop ends exactly at the stop-line. It means that at the time of the signal changing from green to amber, the vehicle being at the distance $S(t^*) = S_H$ from the stop-line decides to brake and then stops safely in front of the stop-line as soon as the signal turns red. Figure 60 presents the relative curve between the stopping distance S_H [m] and the speed V [km/h] in case the deceleration rate is 3.5 m/s^2. The locations of the negative critical decision point ($S(t^*) = S_H$) are located on the curve, the area above the curve ($S(t^*) > S_H$) is defined as the correct braking decision area, and of course the area under the curve ($S(t^*) < S_H$) is the incorrect braking decision area, it means that if the driver decides to brake, the vehicle will cross the stop-line and stop behind it, which is prohibited by traffic law. To explain more clearly, Figure 60 gives an example: if a vehicle is moving at the speed of 50 km/h, this vehicle can only stop safely in front of the stop-line under the given assumptions for t_{Re} and b_v if the distance from the stop-line is equal to or more than 41.45 m.

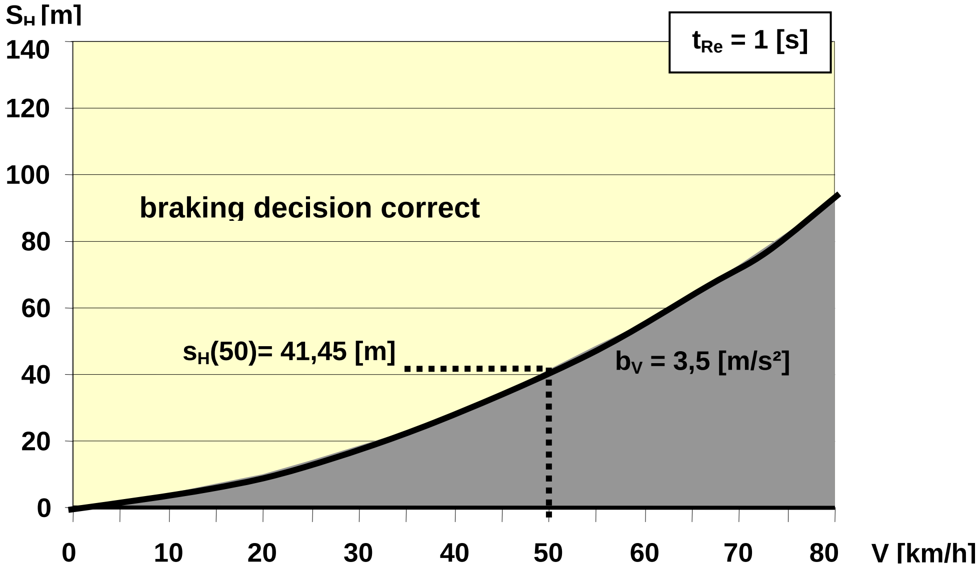


Figure 60: Locations of the negative critical decision point

(Boltze, 2007 according to Retzko, 1966)

b. Crossing decision

As mentioned above, vehicles that could not stop safely in front of the stop-line have to decide to cross the stop-line before the signal turns red.

It is assumed that at the time point t^* the signal starts changing from green to amber, the vehicle is moving at the speed v (m/s), and its position is at the distance of S(t*) from the stop-line. This vehicle can only cross the stop-line safely if it covers this distance $S(t^*)$ before the signal turns red. It means that if the position of the vehicle from the stop-line $S(t^*)$ is equal to or less than $S_F = v^*t_G$ (where t_G is the amber time), this vehicle can cross the stop-line safely. If $S(t^*) > S_F$ and the driver tries to cross the intersection, the vehicle will pass the stop-line during red as presented by the dotted line in Figure 61, and this is not allowed by traffic law.

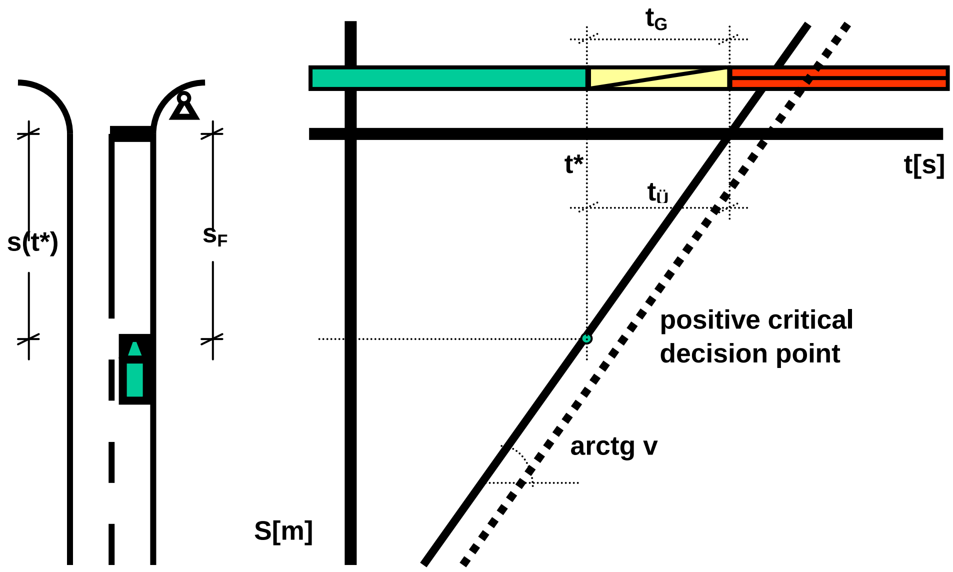


Figure 61: Positive critical decision point

(Boltze, 2007 according to Retzko, 1966)

In Figure 61, the vehicle at the maximum distance from the stop-line that can make a correct crossing decision ($S(t^*) = S_F$) is defined as the **positive critical decision point**.

Figure 62 presents the linear relationship between the speed V [km/h] and the distance S_F [m] by the function $S_F = v*t_G$ in cases t_G = 3s, 4s, and 5s respectively.

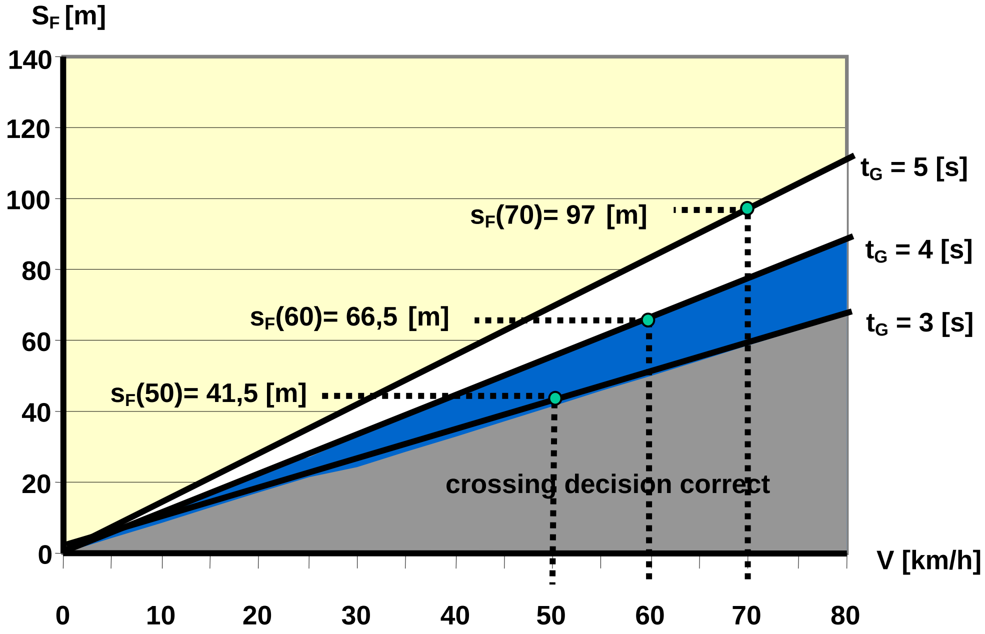


Figure 62: Locations of the positive critical decision point

(Boltze, 2007 according to Retzko, 1966)

In this figure, the locations of the positive critical decision point (at $S(t^*) = S_F$) are located on the linear line depending on the speed. For example, if the vehicle is moving at 50 km/h, 60 km/h, or 70 km/h, the locations of the positive critical decision point are at the distance of 41.5 m, 66.5 m, or 97 m, respectively. The area under the linear line (at $S(t^*) < S_F$) is defined as the correct crossing decision area, and the area above the linear line (at $S(t^*) > S_F$) is the incorrect crossing decision area.

c. Dilemma zone

From the analyses above, the correct braking and crossing decision areas can be summarized in Figure 63 and Figure 64.

Figure 63 presents the relationship between the stopping distance S_H [m] and the speed V [km/h] by the following mathematical function:

$$S_H = \frac{V}{3.6} * t_{\mathrm{Re}} + \frac{V^2}{2*3.6^2*b_v} \quad (46)$$

Figure 64 presents the relationship between the crossing distance S_F [m] and the speed V [km/h] by the following mathematical function:

$$S_F = \frac{V}{3.6} * t_G \quad (47)$$

Where: b_v and t_G were defined above.

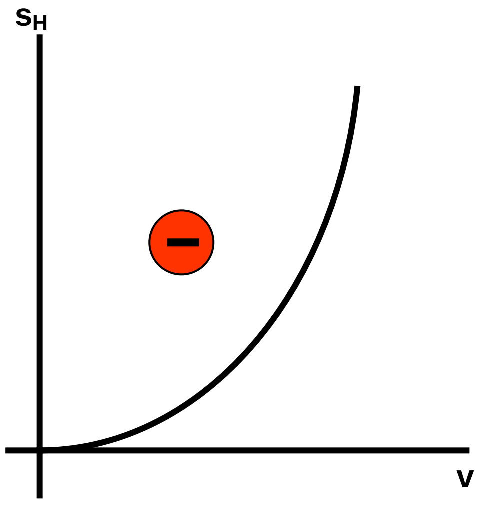


Figure 63: Braking decision correct

(Boltze, 2007 according to Retzko, 1966)

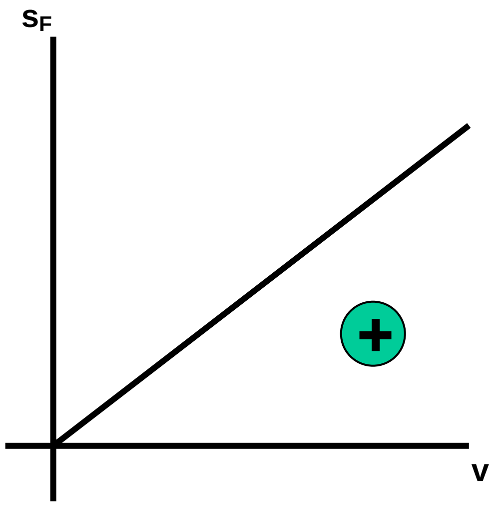


Figure 64: Crossing decision correct

(Boltze, 2007 according to Retzko, 1966)

In practice, as soon as the amber time starts, the drivers can decide either to stop or to cross correctly depending on the distance from them to the stop-line and their speed. Therefore, it is necessary to combine Figure 63 and Figure 64 as follows:

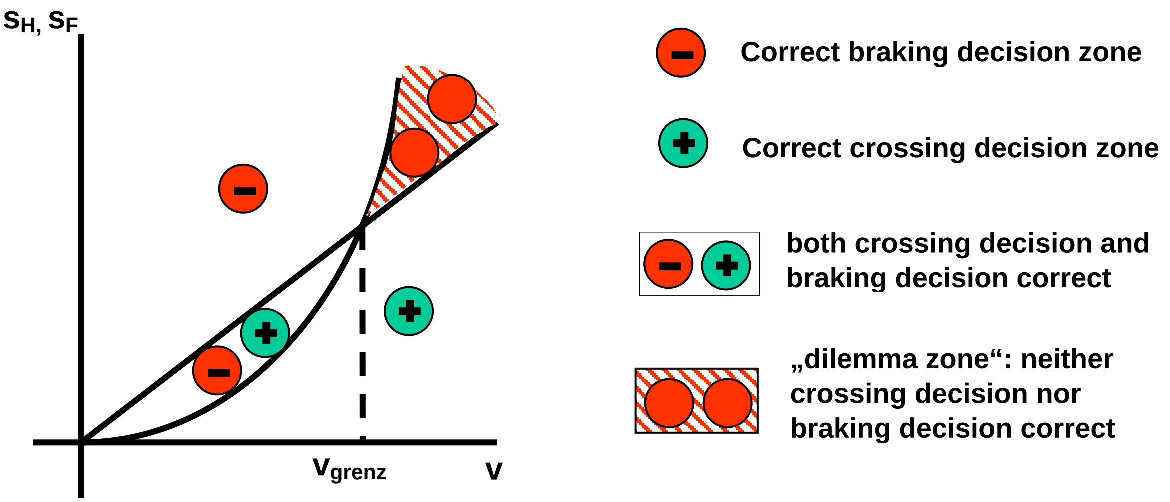


Figure 65: Dilemma zone

(Boltze, 2007 according to Retzko, 1966)

As shown in Figure 65, the dilemma zone is defined as an unsafe area for drivers. When the amber signal starts, the drivers driving in this zone will make neither crossing decision nor braking decision correct. If the driver decides to stop, it means that his/her vehicle will cross the stop-line during the red time and then stop behind the stop-line, inner intersection area. If the driver decides to cross, it means that his/her vehicle will cross the stop-line during the red time. Both cases may lead to accidents that take place in the inner intersection area with entering traffic streams of the successive phase. Therefore, when calculating the amber time, the dilemma zone must be avoided.

From Figure 65, if V_{grenz} is defined as the critical speed, the dilemma zone is always avoided if the drivers are driving at the speed of lower than V_{grenz}. The speed V_{grenz} is derived from the equation of the stopping and crossing distances: $S_H = S_F$ as follows:

$$S_H = S_F \Rightarrow \frac{V}{3.6} * t_{\mathrm{Re}} + \frac{V^2}{2*3.6^2 * b_v} = \frac{V}{3.6} * t_G \Rightarrow V_{grenz} = 2*3.6*b_v*(t_G - t_{\mathrm{Re}})$$

Where: S_H and S_F in unit [m] ; V in unit [km/h].

If V_{zul} is defined as the speed limit at traffic signals, the dilemma zone is always avoided by the following condition:

$$V_{zul} \le V_{grenz}$$

$$V_{zul} \le 2*3.6*b_v*(t_G - t_{\mathrm{Re}})$$

$$\Rightarrow \quad t_G \ge \frac{V_{zul}}{2*3.6*b_v} + t_{\mathrm{Re}} \qquad (48)$$

Finally, if the amber time is satisfied by formula (48), the drivers will not have to confront with the dilemma zone. In addition, the minimum t_G in formula (48) is used because although the higher value is also satisfied formula (48), but it makes the cycle time longer, which should be avoided.

5.4.2.2. Red-and-Amber time

The red and amber time (the red light and the amber light of a signal head are turned on at the same time) was used at first in Great Britain to indicate the signal changing from the red time to the green time. The aim of this transition time is to inform the drivers waiting on the approach to be ready for discharging immediately as soon as the green time starts. Hereby, the start-up lost time is reduced. In Germany, Androsch (1974) did an investigation of the red and amber time, and he achieved the results as follows:

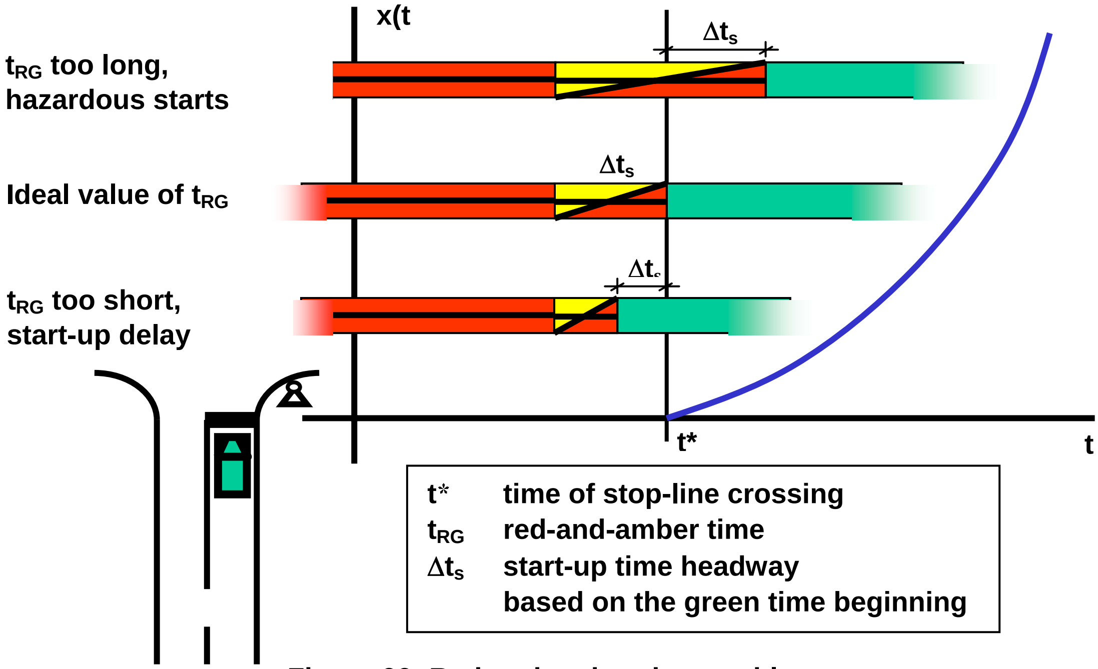


Figure 66: Red-and-amber time problems

(Boltze, 2007 according to Androsch, 1974)

From Figure 66, more details about the effect of the red-and-amber time t_{RG} can be understood as follows:

- If t_{RG} is too long, after the divers gear up the signal is not green yet. Hereby, most of the cases, the drivers start discharging before the green time starts. This may lead to accidents with the last vehicle of the previous phase.
- If t_{RG} is too short, the drivers usually start up when the signal has been already green. It means that a short period of the green time at the beginning is waste.
- The ideal value of t_{RG} is the interval, which is long enough for the drivers to gear up and accelerate immediately as soon as the green time starts.

According to RiLSA edition 2009, the red-and-amber time is 1 s. However, with modern cars (automatically gearing up and down), this value may be lower.

5.4.3. Application to MDCs

In almost all MDCs, there is no signal red-and-amber, therefore this study will analyse and calculate the amber time only.

Because there are both cars and motorcycles driving at traffic signals in MDCs, the amber times for each type of vehicle have to be considered. Then, these amber times will be combined under the traffic safety condition.

5.4.3.1. Amber time for motorcycles

In this case, it is assumed that motorcycles use their own signal head. The amber time for them will be calculated by formula (48). Therefore, three parameters have to be considered.

- ***The reaction time t_{Re} at traffic signals***

  When the signal changes from green to amber, the drivers loss an amount of time to perceive this changing and then make a decision either to brake or to cross. Because, the reaction time t_{Re} in formula (48) is in the case of braking decision; hereby, this reaction time is the interval since the driver perceives and decides until the brake is active. This interval can be investigated in the site by the measurement of the interval since the signal starts changing until the braking red light of vehicle flashes. By this measurement, Behrendt (1970) investigated app. 500 measurement values on two approaches, and the reaction times varied from 0.96 s to 1.23 s on one approach, and from 0.97 s to 1.09 s on the other one. Until now, the German Guidelines for Traffic Signals take 1 s as the reaction time.

  It is assumed that the perception interval of the driver at traffic signals is the same from country to country. Hereby, the reaction times are only different due to vehicle's braking system. Motorcycles have two separate braking systems: hand-brake and foot-brake system, and the experienced motorcycle riders usually have shorter reaction time than the others. Until now, there is no investigation about the reaction time of motorcycle riders at traffic signals in MDCs. In addition, it is not easy to do this investigation by recording the interval since the green signal starts changing until the braking red light of motorcycles flashes under mixed traffic condition.

In Germany (FGSV, 2009) as well as in the United States (ITE, 1985), the reaction time at traffic signals is 1 s. However, in Japan (JSTE, 2006) this value is taken as long as 0.7 s.

In Vietnam, although there is not yet a guideline, or standard for traffic signals, the stopping distance of the vehicles in the Vietnamese standard for road designs is calculated based on the reaction time of 1 s. Therefore, it is acceptable to assume that the reaction time at traffic signals in Vietnam is 1 s.

- ***The speed limit at traffic signals for motorcycles***

The speed limit at traffic signals V_{zul} is the maximum allowed speed at the traffic signal areas. Normally, V_{zul} is less than or equal to the speed limit on the roads to ensure the traffic safety at traffic signals. If V_{zul} is less than the speed limit on the roads, it means that at the computed distance from the traffic signal system, it is necessary to locate a speed limit traffic sign to reduce the speed limit on the roads to V_{zul}. For example, in Germany, according to FGSV (2009), the maximum V_{zul} at traffic signals is 70 km/h, all the roads that have the speed limit of higher than 70 km/h have to reduce to 70 km/h at traffic signals by a speed limit traffic sign.

In Vietnam, the speed limit on the roads is stipulated by the Ministry of Transport (MoT) in article 6 and 7 of the decision No. 05/2007/QD-BGTVT on February 2nd 2007 as follows:

Table 23: Maximum allowed speed inside densely populated areas

Motorised vehicle type	Speed limit on roads (km/h)
Car carrying passengers up to 30 seats; Truck less than 3.500 kg.	50
Car carrying passengers more than 30 seats; Lorry more than or equal to 3.500 kg; Semi-trailer truck; Trailer truck; Car pulling other vehicle; Specialized vehicle; **Motorcycle; Motorbike**.	40

(MoT, 2007)

Table 24: Maximum allowed speed outside densely populated areas

Motorised vehicle type	Speed limit on roads (km/h)
Car carrying passengers up to 30 seats (except bus); Lorry less than 3.500 kg.	80
Car carrying passengers more than 30 seats (except bus); Lorry more than or equal to 3.500 kg.	70
Bus; Semi-trailer truck; Specialized vehicle; **Motorcycle.**	60
Trailer truck; Car pulling other vehicle; **Motorbike.**	50

(MoT, 2007)

From the two tables above, it is acceptable to assign the speed limit at traffic signals V_{zul} for motorcycles at the speed of 40 km/h inside densely populated areas and at the speed of 60 km/h outside densely populated areas. Hereby, it is not necessary to locate any speed limit traffic sign to reduce the speed of motorcycles before traffic signal systems.

- **The deceleration rate b_v of motorcycle**

The deceleration rate of a vehicle depends on the quality of the vehicle (especially, the braking system), on the pavement surface and its condition (normal, wet, or dry), and on the weight of the vehicle. It is usually stipulated in the traffic law of each country.

In Vietnam, the deceleration rate of motorcycles is stipulated by the braking distance in the braking test before allowed importing motorcycle products or producing motorcycles. According to the decision No. 03/2008/QĐ-BGTVT of MoT (2008), the braking test is carried out on the cement concrete or asphalt pavement under a very good condition: pavement surface is flat, dry, and the driver's weight is 75 kg. Under this condition, the braking distance is not allowed to be longer than 7.5 m when braking at the speed of 30 km/h. Hereby, the deceleration rate can be calculated as follows:

$$S_B = \frac{V^2}{2*3.6^2*b_v} \quad \Rightarrow \quad 7.5 = \frac{30^2}{2*3.6^2*b_v} \quad \Rightarrow \quad b_v = \frac{30^2}{2*3.6^2*7.5} = 4.63\,[\text{m/s}^2]$$

This deceleration rate value cannot be taken into the calculation of the amber time because it was tested under good conditions. In reality, it needs to be adjusted by some following elements:

- **Weight element:**

According to the motorcycle producer Yamaha in Vietnam, the wet weight (including fuel, lubricant) of a motorcycle varies from 94 kg to 109 kg depending on the motorcycle style (for example: Sirius 96 kg, Jupiter 100 kg, Exciter 109 kg, Nouvor 108 kg, and Mio 94 kg). Unlike car, the deceleration rate of motorcycle varies much depending on how many persons are carried on it. According to the standard test, there is only one person with the weight of 75 kg, but in reality, the traffic situation with two persons on one motorcycle occurs very often.

It is assumed that the average weight of a motorcycle is 100 kg, the braking force according to the test can be determined as follows:

$$\frac{\text{Braking force}}{\text{Total weight}} = \frac{b_v}{g}$$

$$\text{Braking force} = \text{Total weight} * \frac{b_v}{g} = (100+75)*\frac{4.63}{9.81} = 82.59\,(\text{kg})$$

Where: g is the gravitational acceleration (9.81 m/s^2).

If the motorcycle carries two persons and the braking force is unchanged, then the deceleration rate is determined as follows:

$$b_v = g*\frac{\text{Braking force}}{\text{Total weight}} = 9.81*\frac{82.59}{(100+75+75)} = 3.24\,(\text{m/s}^2)$$

- **Pavement surface condition**

  If the wet pavement surface condition is taken into account, the deceleration rate of 3.24 m/s^2 must be reduced, because under this condition the grip force between the tire and the pavement is impaired.

From the analysis above, it is assumed that when calculating the amber time, the deceleration rate of a motorcycle 2.8 m/s^2 is taken into account in Vietnam. Furthermore, in Germany, the minimum deceleration rate of motorcycle is 2.9 m/s^2 according to BfV (1964).

According to formula (48), the amber time for motorcycles is determined as follows:

- $t_G \geq \dfrac{40}{2*3.6*2.8} + 1 = 2.98\,\mathrm{s}$ $\Rightarrow$ $t_G = 3$ s in case $V_{zul} = 40$ km/h
- $t_G \geq \dfrac{60}{2*3.6*2.8} + 1 = 3.97\,\mathrm{s}$ $\Rightarrow$ $t_G = 4$ s in case $V_{zul} = 60$ km/h

Figure 67 presents the dilemma zone in case of motorcycle traffic. However, this zone is always avoided if the motorcycle riders drive at the speed of lower than 40 km/h inside the densely populated areas, and lower than 60 km/h outside the densely populated areas.

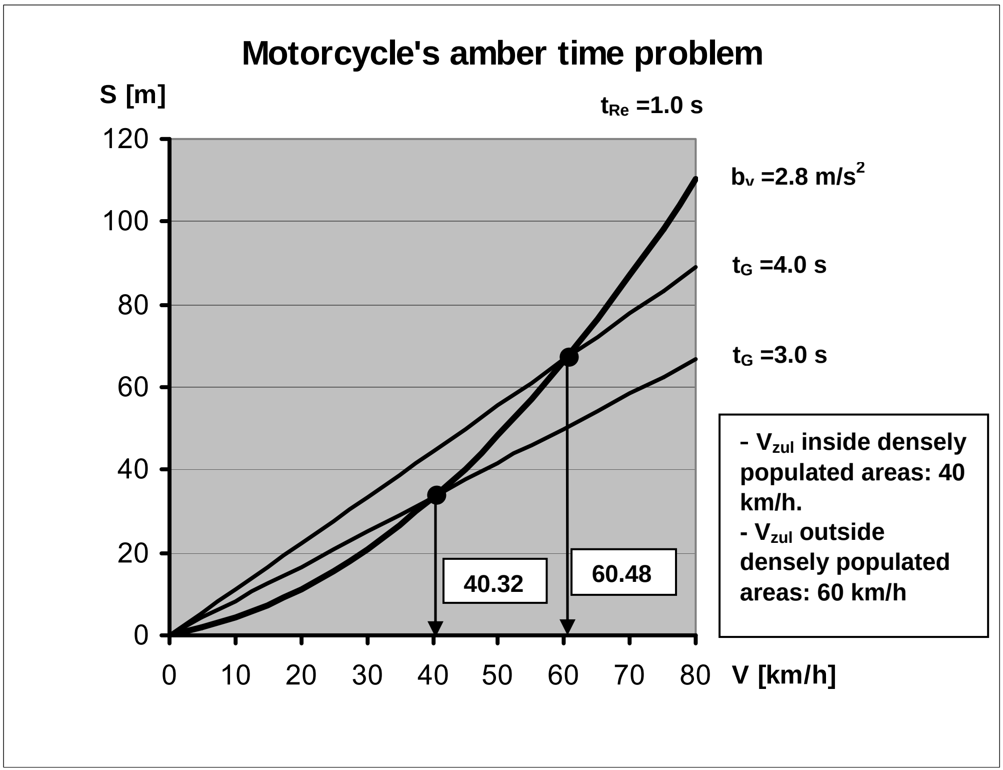


Figure 67: Motorcycle’s amber time problems

5.4.3.2. Amber time for cars

In Vietnam, according to the decision No. 24/2006/QD-BGTVT on October 5th 2006 (MoT, 2006), the deceleration rate of a car is required by the braking test on the asphalt or cement concrete pavement under the flat and dry pavement surface condition with a grip coefficient between tires and pavement of not less than 0.6 as follows:

Table 25: Effectiveness of the braking system without loading

Vehicle type	Initial braking speed (km/h)	Braking distance (m)	Maximum deceleration rate (m/s^2)	Braking corridor (m)
Private car	50	≤ 19	≥ 6,2	2,5
Truck, passenger car less than or equal to 3,5 tons	50	≤ 21	≥ 5,8	2,5
The other car	30	≤ 9	≥ 5,4	3,0

(MoT, 2006).

Table 26: Effectiveness of the braking system with full loading

Vehicle type	Initial braking speed (km/h)	Braking distance (m)	Maximum deceleration rate (m/s^2)	Braking corridor (m)
Private car	50	≤ 20	≥ 5,9	2,5
Truck, passenger car with total weight less than or equal to 3,5 tons	50	≤ 22	≥ 5,4	2,5
The other car	30	≤ 10	≥ 5,0	3,0

(MoT, 2006).

From Table 25 and Table 26, the deceleration rate of 5.0 m/s^2 can be taken into account. However, this value is achieved under a very good condition of pavement surface. Therefore, it is assumed that the value of **3.5 m/s^2** is taken into the calculation of the amber time. This value was also assumed in Germany (Boltze, 2007).

According to Table 23 and Table 24, the maximum allowed speed for car traffic is 50 km/h inside densely populated areas, and 80 km/h outside densely populated areas. Therefore, it is reasonable to assign the speed limit at traffic signals V_{zul} of 50 km/h inside densely populated areas. But, outside densely populated areas, where the speed limit on roads is 80 km/h, V_{zul} can be reduced to 70 km/h at traffic signals as introduced in Germany due to safety reason.

If the reaction time t_{Re} of car drivers is 1 s, then the amber time is determined as follows:

- $t_G \geq \frac{50}{2*3.6*3.5} + 1 = 2.98\,\text{s}$ $\Rightarrow$ $t_G = 3$ s in case $V_{zul} = 50$ km/h.
- $t_G \geq \frac{70}{2*3.6*3.5} + 1 = 3.78\,\text{s}$ $\Rightarrow$ $t_G = 4$ s in case $V_{zul} = 70$ km/h.

Figure 68 presents the dilemma zone for car traffic.

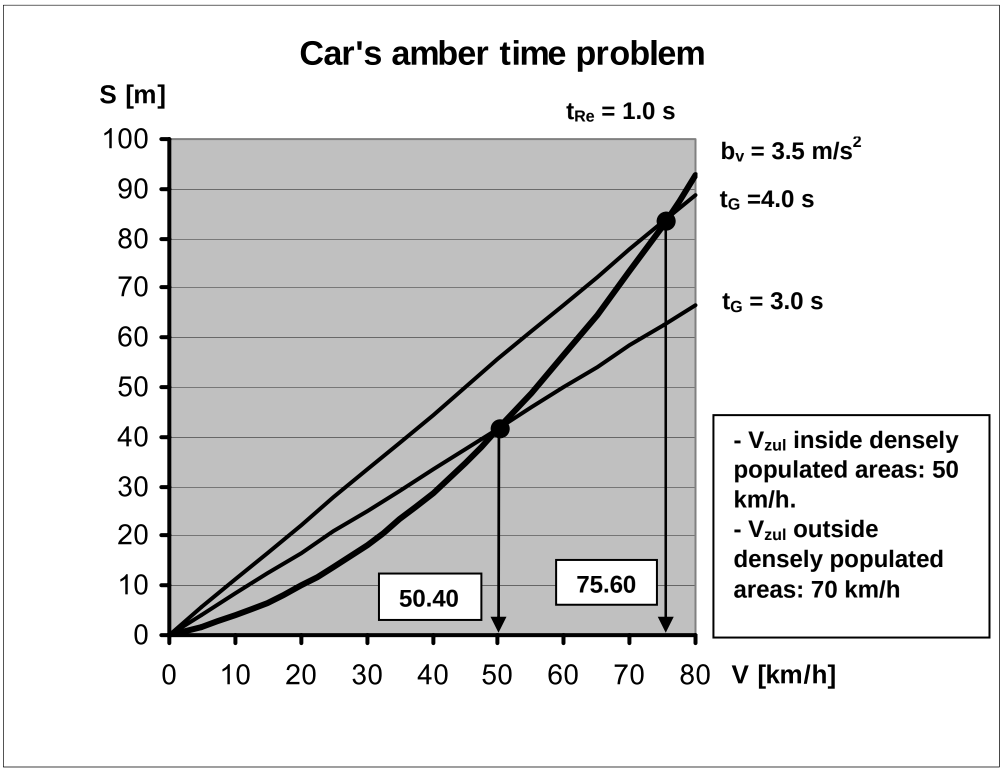


Figure 68: Car's amber time problems

5.4.4. Conclusions

In MDCs, depending on the intersection layout design, motorcycles and cars can either share the signal head or not. If they share the signal head, they have to use the same amber time. However, with the results analysed above, the amber time of 3 s and 4 s can be used for both types of vehicle in MDCs depending on their V_{zul} stipulated in the traffic law. Therefore, it can be concluded that:

- t_G = 3 s in case V_{zul} = 50 km/h (in which motorcycles are only allowed to drive at the maximum speed of 40 km/h according to the traffic law),
- t_G = 4 s in case V_{zul} = 70 km/h (in which motorcycles are only allowed to drive at the maximum speed of 60 km/h according to the traffic law)

Figure 69 and Figure 70 present the dilemma zone of motorcycles and cars in case the amber time is 3 s and 4 s, respectively.

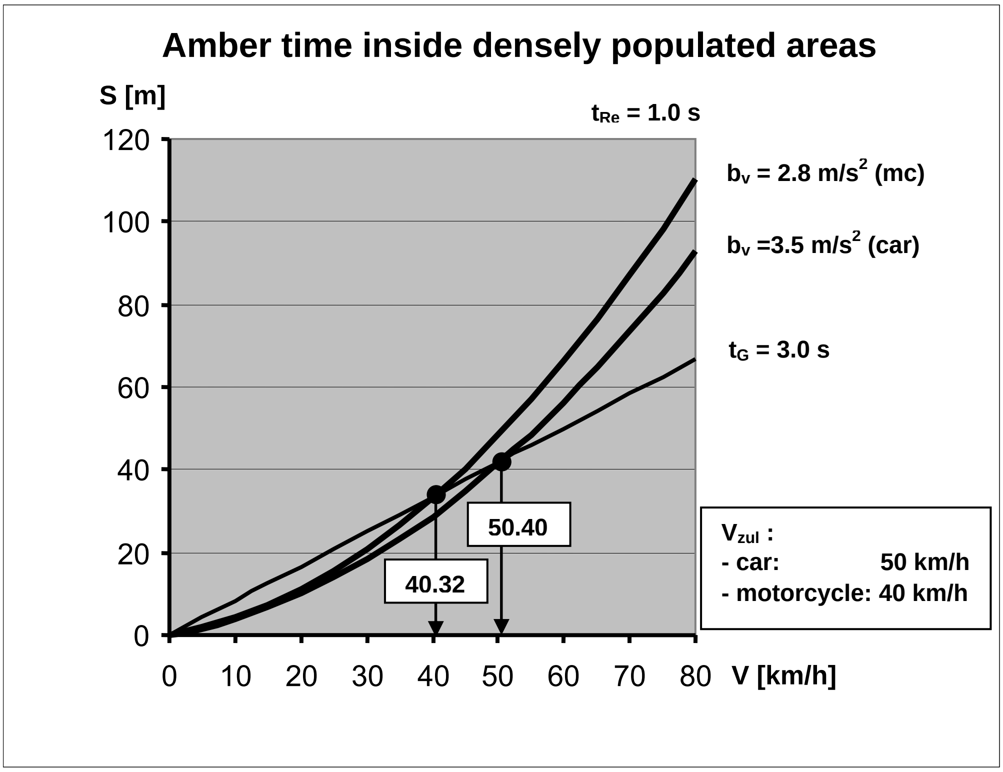


Figure 69: Amber time inside densely populated areas in MDCs

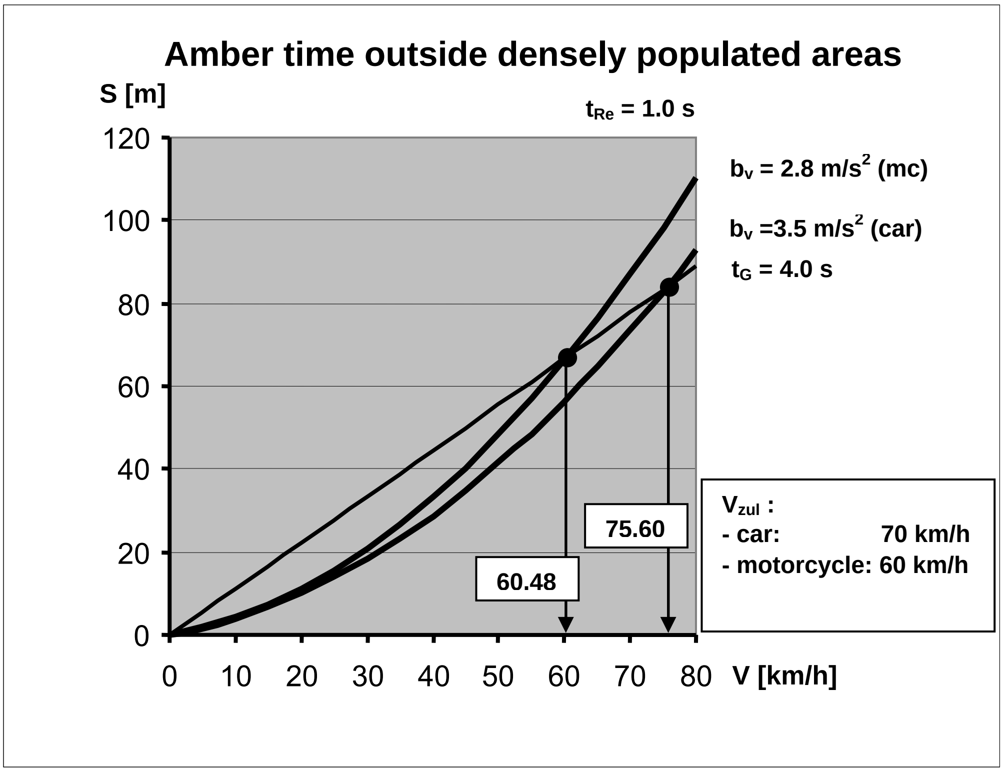


Figure 70: Amber time outside densely populated areas in MDCs

5.5. Intergreen time

5.5.1. General

The intergreen times have been discussed a lot in the articles coming from different countries in ITE Journal, Traffic Engineering & Control, Straßenverkehrstechnik, such as: Jourdain (1986); Zador (1985); Lin (1986). Most of these articles discussed in trend with the concept of the intergreen time proposed by the method of ITE (ITE, 1985), (ITE, 1989). Although each country has its own philosophy, it may be said that the goal is to ensure the traffic safety when changing from one phase to another.

ITE (1985) and ITE (1989) used the terms ''change interval (yellow)'' and ''clearance interval (all-red)'' to express and calculate the total lost time as written in Highway Capacity Manual HCM 2000 (TRB, 2000): "the total lost time = yellow time + all-red time" (note that this formula is achieved only if the start-up lost time l_1 is equal to the extension of effective green time e. According to HCM 2000, l_1 = e = 2 s).

In Germany, Jakob (1982) proposed one more simple method to determine the intergreen time based on a probabilistic approach. However, it was not allowed to be used in Germany because of juridical consideration.

The German method according to the German Guidelines for Traffic Signals (RiLSA edition 1981) was discussed in the ITE journal by Retzko and Boltze (1987). Up to now, in the latest RiLSA (edition 2009), this method is basically kept the same. This method will be discussed below in order to apply to MDCs.

In addition, comparing with the German method, the United States does not use the entering time (t_e = 0) when calculating the intergreen time. According to this concept, the last vehicle of the current phase has already cleared the intersection, then the first vehicle of the next phase starts to enter the intersection. But, Germany uses the entering time to save the intergreen times. Therefore, the intergreen time is usually shorter while traffic process is still safe.

5.5.2. German method for determining intergreen times

According to the German method (FGSV, 1992), the intergreen time is defined as follows:

''The intergreen time is the interval between the end of the green time for one traffic stream and the beginning of the green time for the next one (the conflicting traffic stream)''.

By this definition, the last vehicle of the ending green time (traffic stream A) must have cleared the conflict area at the latest when the first vehicle of the beginning green time (traffic stream B) arrives at the conflict area (see Figure 71). During the intergreen time, different movements occur: crossing and clearing movements of the last vehicle of the traffic stream A, and the entering movement of the first vehicle of the traffic stream B.

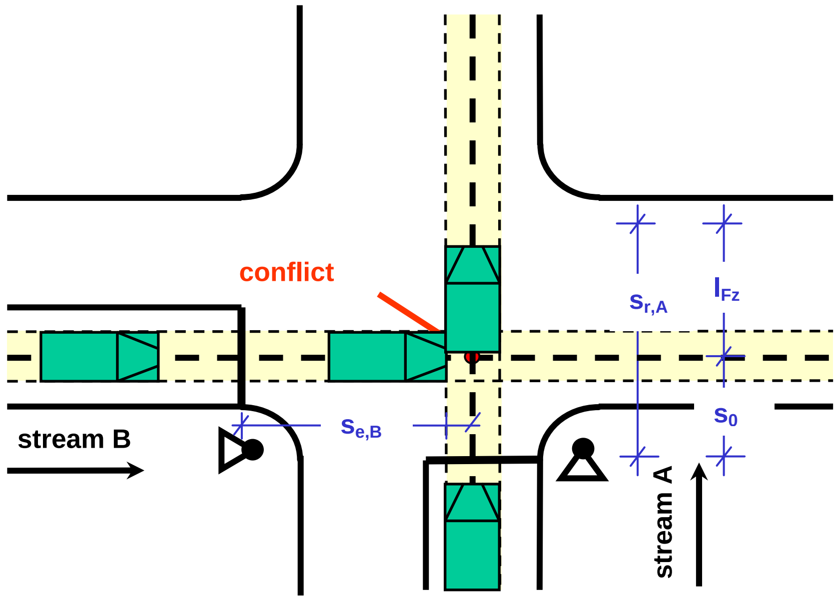


Figure 71: Clearing distance and entering distance

(Boltze, 2007 according to the German method)

All the movements occurring during the intergreen time are illustrated in Figure 72.

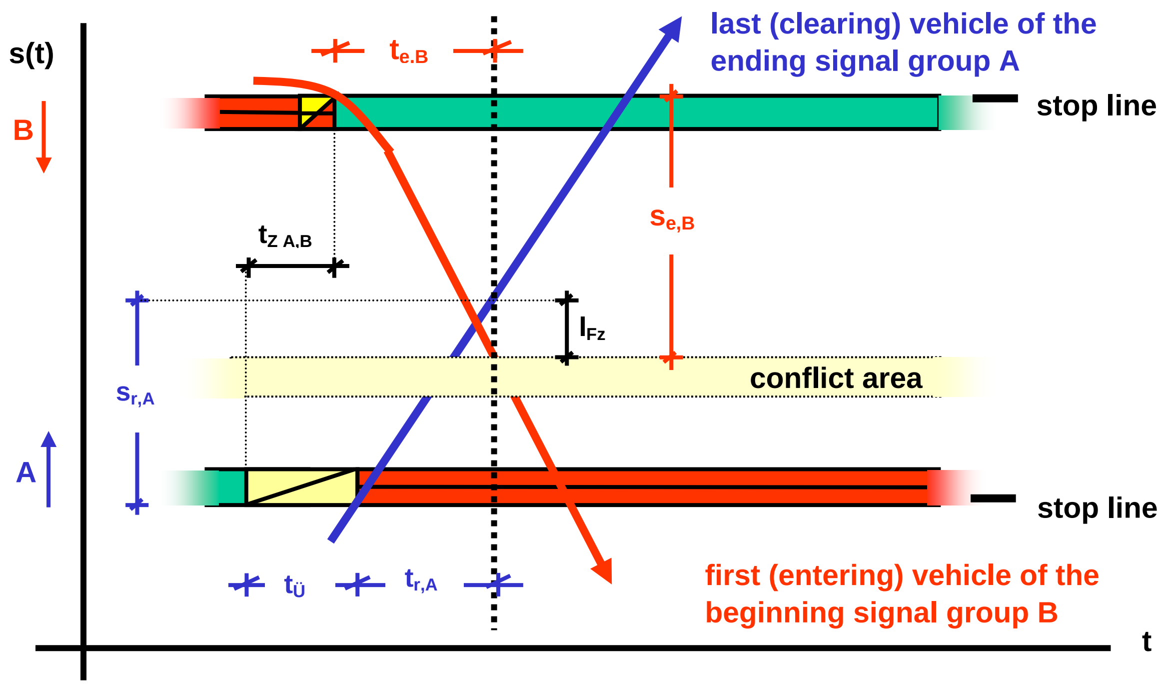


Figure 72: Movements of vehicles during the intergreen time

(Boltze, 2007 according to the German method)

Where:	s_r = clearing distance;	v_r = clearing speed;	t_r = clearance time
	s_e = entering distance;	v_e = entering speed;	t_e = entering time
	$s_r = s_o + l_{Fz}$;		$t_ü$ = crossing time
	s_o = basic clearing distance;		t_Z = intergreen time
	l_{Fz} = fictitious length of vehicle;		

From the figures above, the intergreen time t_Z is calculated as follows:

$$t_Z = t_ü + t_r - t_e \qquad (49)$$

Therefore, to determine the intergreen time t_Z, each term of equation (49) needs to be determined.

a. Crossing time $t_ü$

According to RiLSA (FGSV, 2009), the crossing time is the interval between the end of the green time and the beginning of the clearance time. The clearance time is the interval needed to cover the clearing distance. The clearing distance is that distance from the stop-line to the front-top of the vehicle at the latest position that has just cleared the conflict point.

Therefore, it can also be said that the crossing time is the interval since the end of the green time until the vehicle reaches the stop-line. The crossing time exists only if the driver decides to cross the intersection at the moment of the green time ending (the amber time starts). Hereby, the crossing time is a part of the amber time, and the maximum crossing time is equal to the amber time (see Figure 73). In addition, the lower the speed limit is, the lower the clearing speed will be, and therefore the shorter the crossing time will be, because slowly driving vehicles are better able to react on the green time ending. In RiLSA 2009, $t_ü$, therefore, was set depending on the clearing speed, for example, at 3 s for straight-on moving vehicles, and at 2 s for slowly moving vehicles (because e.g. turning vehicles normally approach the intersection at the slower speed).

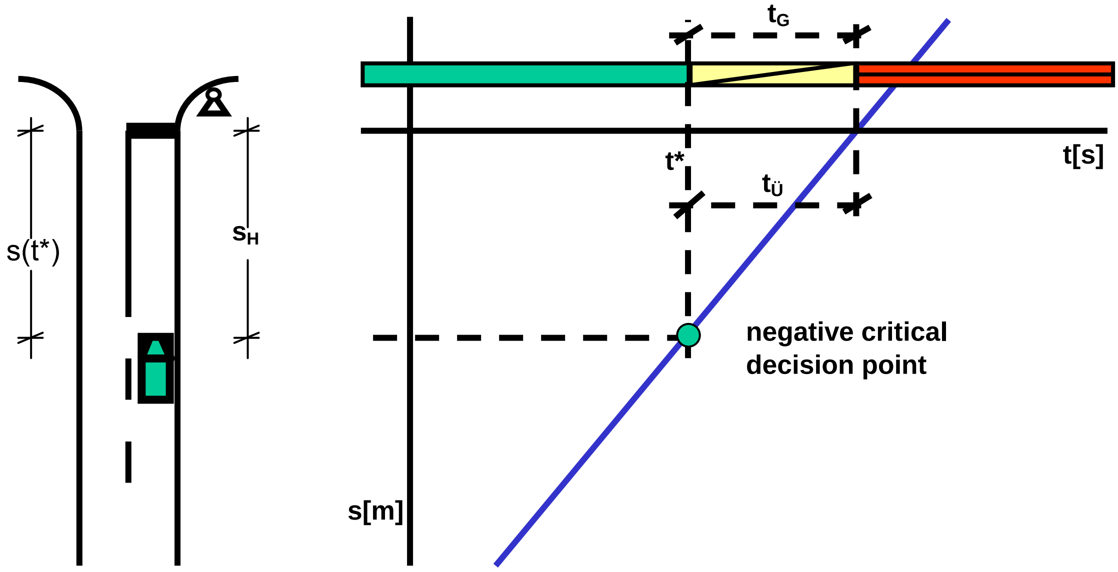


Figure 73: Maximum crossing time

(Boltze, 2007 according to the German method)

b. Clearance time

As defined above, the clearance time is the interval needed to cover the clearing distance s_r, and determined as follows:

$$t_r = \frac{s_r}{v_r} \qquad (50)$$

Normally, the clearing distance s_r in formula (50) is fixed by the geometry of the intersection and the fictitious length of the vehicle. The clearance time t_r, therefore, depends much on the clearing speed v_r. In terms of safety, the lower clearing speed v_r is critical. According to ITE (1985), to provide a reasonable clearance time, the use of the same value for the speed limit ($v_r = v_{zul}$) is not always valid. This is especially true for protected turning movements. The preferable method for identifying the vehicle speed involves speed sampling, but estimation methods are also available.

For example, ITE (1989) proposed v_{85} to determine the amber time, but v_{15} for determining the clearance interval and assumed that v_{15} is 10mph (16.09 km/h) less than v_{85}.

However, the German Guidelines for Traffic Signals (FGSV, 2009) proposed v_{zul} = 50 km/h to determine the amber time inside urban areas. But, to determine the clearing time, FGSV (2009) proposed the clearing speed at v_r = 10 m/s (36 km/h) for straight-on vehicles, at v_r = 7 m/s (25,2 km/h) for turning vehicles, and at v_r = 5 m/s (18 km/h) in case the radius of the inner lane edge R < 10 m. In other words, it may be said that for the amber time the maximum speed is critical, but for the clearing time the minimum speed is critical.

c. Entering time

According to RiLSA (FGSV, 2009), the entering time is the interval needed for the first entering vehicle to cover the entering distance s_e. This entering time is determined by the following formula:

$$t_e = \frac{3.6 * s_e}{40} \text{ (s)} \qquad (51)$$

This formula is based on the assumption that the first entering vehicle is entering the stop-line at the speed of 40 km/h and keeps this speed until reaching the conflicting area; this situation has been seen as the critical case. This is quite reasonable because if it is assumed that the first vehicle is approaching the intersection at the speed of v_{zul} = 50 km/h, this driver tends to decelerate when approaching the red signal, but still keeps the relatively high speed. When the red-and-amber time starts and remains 1 s, the driver will accelerate again and enter the stop-line as soon as the green time begins. Therefore, the speed at the moment of entering the stop-line is usually lower than the speed limit v_{zul} (40 km/h comparing to 50 km/h). In other words, it may be said that the maximum entering speed is critical, but must be lower than v_{zul}.

Cases for determining the intergreen times in Germany:

Depending on the traffic situations at traffic signals in each country, the intergreen time must be considered and determined for individual cases. In Germany, the intergreen times are classified into 6 cases:

Case 1: Straight-on vehicles are clearing

Case 2: Turning vehicles are clearing

Case 3: Trams are clearing without stop before the intersection

Case 4: Public transport vehicles are clearing with stop before the intersection

Case 5: Cyclists are clearing

Case 6: Pedestrians are clearing

These six cases are presented in detail in RiLSA edition 2009 (FGSV, 2009).

5.5.3. Application to MDCs

5.5.3.1. Setting values for determining the intergreen time

Going from the geometry elements of the intersection in MDCs, especially the positions of the stop-lines for motorcycles and cars as illustrated in Figure 74 below:

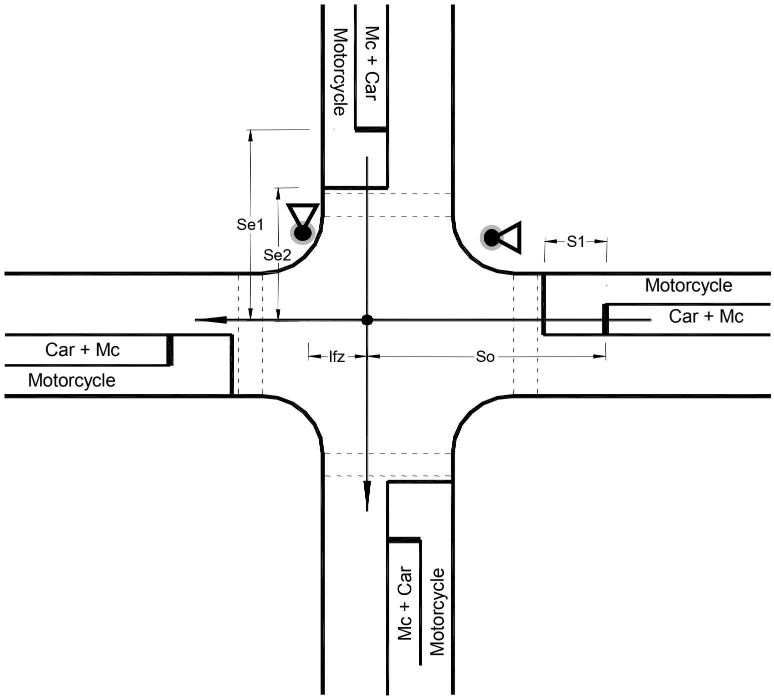


Figure 74: The intergreen time in MDCs

Some traffic models were already introduced in chapter 4 depending on the intersection layout. However, to analyse for determining the intergreen time, all these traffic models in MDCs can be generally summarized as shown in Figure 74. It means that either a motorcycle or a car can be the clearing vehicle or the entering vehicle. However, during the red time, cars must stop in front of their own stop-line, and motorcycles have an opportunity to move up and stop in front of their own stop-line ahead.

According to the German method, individual elements for determining the intergreen time in MDCs must be analysed clearly as follows:

a. Crossing time

In the critical case, the stop-line of cars is referred for determining the intergreen times. Because the speed limit v_{zul} at traffic signals for car traffic in MDCs is the same as in Germany, the crossing time value can be taken from RiLSA edition 2009. These crossing time values are still safe for motorcycles because the speed limit of motorcycles is lower than that of cars (note that lower speed limit gives lower crossing time). In addition, these values of the crossing time are also reasonable according to the investigation under the mixed traffic condition at Daewoo intersection in Hanoi (see Annex B).

b. Clearance time

As discussed above, the clearance time is determined by the formula: $t_r = \frac{s_r}{v_r}$, where $s_r = s_o + l_{Fz}$ as shown in Figure 74. While the basic clearing distance s_o is fixed by the geometry elements, the fictitious length of vehicle l_{Fz} can be either of car or of motorcycle. It is assumed that l_{Fz} of car is 6 m, of motorcycle is 2 m, the clearance time t_r of the car will be longer than that of the motorcycle in case of the same clearing speed v_r (because the fictitious length of the car is longer than that of the motorcycle). Hereby, the calculation of the intergreen time for car is longer than for motorcycle. Therefore, in order to make it simple, the fictitious length of vehicle is always chosen as equal as 6 m for determining the intergreen time in MCDs.

Now, the clearing speed v_r in MDCs has to be analysed. Since the speed limit in MDCs in Vietnam are stipulated as high as 40 km/h for motorcycles and 50 km/h for cars inside densely populated areas, it is still reasonable to assume the clearing speed v_r of 10 m/s (36 km/h) for straight-on cars, and of 7 m/s (25,2 km/h) for turning cars as introduced in the German method. However these values are too high for motorcycles because v_{zul} of motorcycles is lower. It means that, in practice, if a straight-on motorcycle is clearing at a speed of less than 36 km/h, it cannot clear the conflict area before the entering vehicle comes up. This can definitely occur when the speed limit at traffic signals for motorcycles is as high as 40 km/h (very close to the value of 36 km/h). Therefore, the clearing speed according to the German method must be reduced for MDCs due to v_{zul} of motorcycles is lower. If the speed limit at traffic signals for motorcycles is 40 km/h, it is reasonable to assume the clearing speed v_r of 8 m/s (28.8 km/h) for straight-on motorcycles and of 5 m/s (18 km/h) for turning motorcycles. Obviously, these values are still safe for cars because the speed limit of cars is 50 km/h. In addition, these values of the clearing speed are also reasonable according to the investigation under mixed traffic conditions at Daewoo intersection in Hanoi (see Annex B).

c. Entering time in MDCs

As shown in Figure 74, in the critical case, the stop-line of motorcycles is referred for calculating the entering time because the stop-line of motorcycles is closer to the conflict point than that of cars.

Now, the entering speed v_e has to be discussed. As discussed in the German method for the entering time, the entering speed in RiLSA 2009 is set at 40 km/h. This value shows that, in practice, if the vehicle is entering the stop-line at a speed of more than 40 km/h, the high risk of accident may happen because this vehicle will reach the conflict point before the clearing vehicle comes up. Therefore, the higher the value of entering speed is set in the guidelines, the safer the traffic process in practice is, and of course the longer the intergreen time will be.

In MDCs, there is no red-and-amber signal as in Germany. Hereby, the value of the entering speed may be lower than 40 km/h. However, in respect of safety, the value of 40 km/h for the entering speed is still safe for both cars and motorcycles (note that v_{zul} of cars is 50 km/h, v_{zul} of motorcycles is 40 km/h). Therefore, this value should be kept the same as in RiLSA.

Finally, from the referred stop-line for motorcycles in Figure 74, and the entering speed of 40 km/h, the entering time is determined by the following formula:

$$t_e = \frac{3.6 * s_{e2}}{40} \qquad (52)$$

5.5.3.2. Cases for determining the intergreen time in MDCs

Since MDCs did not have special traffic situations as in Germany, especially situations of trams, the following cases for determining the intergreen times in MDCs are recommended:

Case 1: Straight-on vehicles are clearing,

Case 2: Turning vehicles are clearing,

Case 3: Cyclists are clearing,

Case 4: Pedestrians are clearing,

Case 5: Buses are clearing with and without stop before the intersection.

These cases will be presented in detail in the draft of ''Guidelines for Traffic Signals in MDCs'' (see Annex A).

5.5.4. Conclusions

The intergreen time is determined by the following formula:

$$t_z = t_ü + t_r - t_e$$

For motorised traffic, all the values are kept the same as in RiLSA, except:

- The referred stop-line: for the clearing vehicle, the stop-line of cars is critical. For the entering vehicle, the stop-line of motorcycles is critical.
- The clearing speed: the clearing speed in MDCs is lower than that in RiLSA due to the mixed traffic condition as well as the lower speed limit for motorcycles (40 km/h).
- The following formula is used to calculate the intergreen time for motorised traffic:

$$t_z = 3 + \frac{s_o + 6}{8} - \frac{3.6 * s_{e2}}{40}$$ for straight-on vehicles,

$$t_z = 2 + \frac{s_o + 6}{5} - \frac{3.6 * s_{e2}}{40}$$ for turning vehicles,

where s_{e2} and s_o are shown in Figure 74.

For buses, pedestrians, and cyclists, the intergreen time is calculated the same as in RiLSA.

6. Traffic Signal Control Strategies in MDCs

6.1. Overview on control strategies in MDCs

As analysed in detail in chapter 3, some control strategies (A2, A3, and B6) proposed in RiLSA by FGSV (2009) cannot be applied to MDCs due to infeasibility in collecting traffic data online, especially traffic volume. The following table shows the overview on control strategies in MDCs.

Table 27: Overview on the control strategies in MDCs

<table>
<tr><td rowspan="2"></td><td colspan="2">control strategy</td><td rowspan="2">number</td><td colspan="2">activation</td><td colspan="5">traffic-dependent variable elements of the signal programs</td></tr>
<tr><td>general term</td><td>main feature of signal program modification</td><td>time-dependent</td><td>traffic-dependent</td><td>cycle time</td><td>phase sequence</td><td>number of phases</td><td>green time</td><td>time offset</td></tr>
<tr><td rowspan="3">A:macroscopic control level</td><td rowspan="2">signal program selection</td><td>time-dependent signal program selection</td><td>A1</td><td>X</td><td></td><td colspan="5" rowspan="3">in combination with the variable elements of the signal programs from a control strategy of the group B</td></tr>
<tr><td>traffic-dependent signal program selection</td><td>A2</td><td></td><td>X</td></tr>
<tr><td>signal program formation</td><td>forming signal programs by traffic-dependent selection</td><td>A3</td><td></td><td>X</td></tr>
<tr><td rowspan="6">B: microscopic control level</td><td>fixed-time signal program</td><td></td><td>B1</td><td colspan="2" rowspan="6">activation according to control strategies of group A</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="4">signal program adaptation</td><td>green time adjustment</td><td>B2</td><td></td><td></td><td></td><td>X</td><td></td></tr>
<tr><td>phase swapping</td><td>B3</td><td></td><td>X</td><td></td><td></td><td></td></tr>
<tr><td>demand phase</td><td>B4</td><td></td><td></td><td>X</td><td>X</td><td></td></tr>
<tr><td>time-offset adjustment</td><td>B5</td><td></td><td></td><td></td><td></td><td>X</td></tr>
<tr><td>signal program formation</td><td>free modification possible</td><td>B6</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td></tr>
</table>

(based on FGSV, 2009) Remark: control strategies usually not applicable in MDCs

FGSV (2009) has described in detail the individual control strategies in RiLSA already. Therefore, this chapter will go through on how to develop the applicable control strategies (A1, B1, B2, B3, B4, and B5) in MDCs.

The macroscopic control level A1 (time-dependent signal program selection) has been presented in RiLSA already (see the annex “draft of Guidelines for Traffic Signals in MDCs”), and it does not need any modification for MDCs.

All the microscopic control levels in MDCs will be activated from the macroscopic control level A1, in which the microscopic control level B1 (fixed-time signal program) does not need any modification. However, the other microscopic control levels (B2, B3, B4, and B5) need to be clearly analysed whether motorcycles can influence on the control parameters or not. Therefore, the following sub-chapters will focus on the substantial issues of the control parameters to develop traffic-actuated control in MDCs (control level: B2, B3, B4, and B5).

6.2. Parameters for traffic-actuated control in MDCs

6.2.1. Green time request

Green time request for pedestrians, cyclists and buses can be applied from RiLSA without any modification. However, green time request for motorised traffic must be modified for locating positions of the inductive loop due to the head-start area for motorcycles. The following figures show positions as well as shapes of the inductive loops.

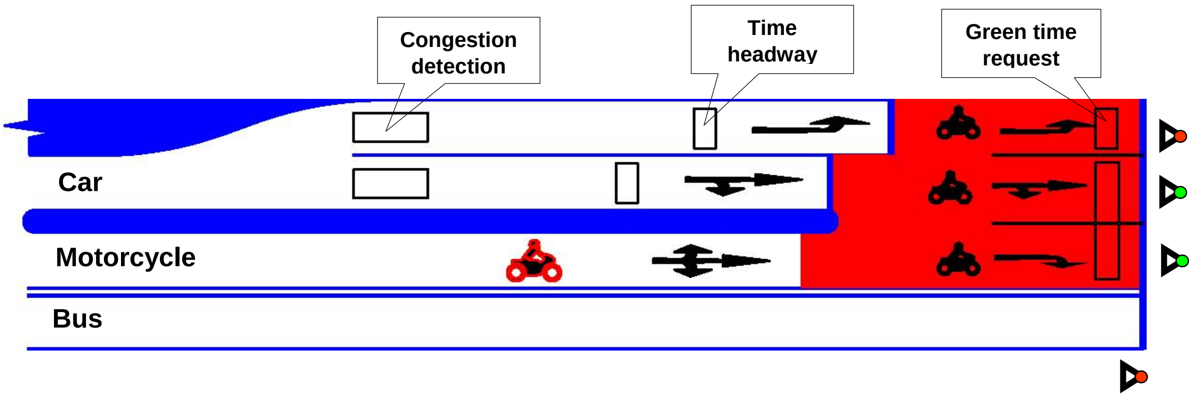


Figure 75: Positions of inductive loops in traffic model 1

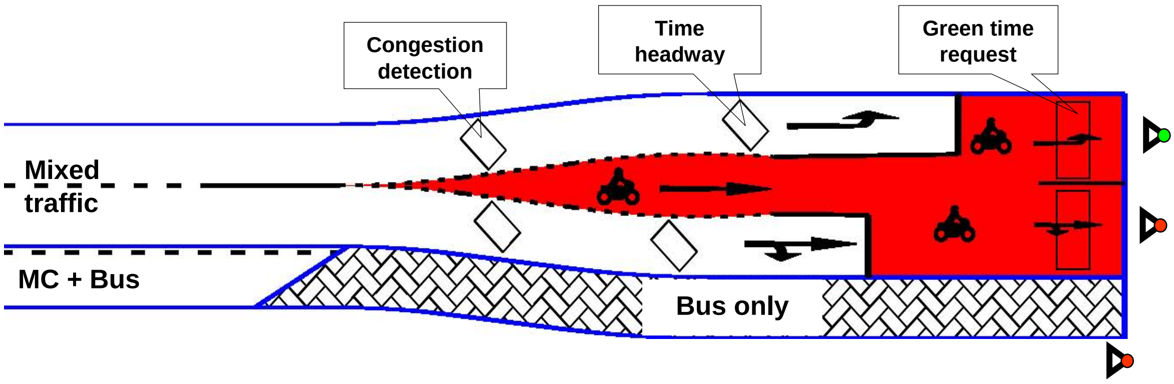


Figure 76: Positions of inductive loops in traffic model 2

The inductive loops, here, are designed for the green time request of motorcycle vehicles. As the maximal length of a motorcycle is 2 m, the inductive loop is located approximately 1 m backward from the stop-line of motorcycle. The length of the inductive loop is from 1 m to 1.5 m, and its width should be 0.5 m narrower than the lane width due to the small size of motorcycle (see Figure 77).

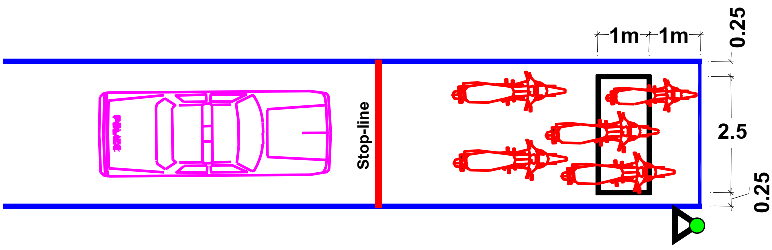


Figure 77: An example of green time request for motorcycles

The function of these detectors is to detect the presence of waiting motorcycles. However, because a motorcycle has an amount of metal less than that of a car, therefore if the inductance of the motorcycle is not strong enough for the detection, it is necessary to rotate the inductive loop an angle in order to increase the inductance (an angle of 45 degrees will give the maximum inductance) (FGSV, 1991). But the distance from the loop to the stop-line has to be proper for the length of motorcycle to ensure that the waiting motorcycle is not out of the magnetic flux produced by the electrical current in the wires of the detector (see principles of the inductive loops in Merkblatt über Detektoren für den Straßenverkehr (FGSV, 1991), and in Traffic Detector Handbook (ITE, second edition)).

Finally, before operating, the inductive loops have to be tested to ensure the detection for the presence of motorcycle vehicles.

6.2.2. Time headway

Time headway can also be applied to MDCs. It is used to extend the green time for arriving vehicles based on the given time headway value ZL between vehicles (see details in annex "draft of Guidelines for Traffic Signals in MDCs").

The function of the inductive loop in this case is detecting the passage of vehicles (see Figure 75 and Figure 76). The dimensions of this inductive loop are also similar to the case of green time request. This kind of inductive loop can easily detect the passage of a car due to high amount of metal. However, in case of motorcycles with lower amount of metal, it is necessary to rotate the detector with an angle of 45 degrees to increase the inductance of motorcycles in the magnetic flux area.

In Figure 75, motorcycles are separated from car traffic, the time headway is applied to car traffic only, and it does not need to rotate the detectors. However, it is important that the position of the car's stop-line is referred to locate the position of detectors according to RiLSA.

In Figure 76, under the mixed traffic condition, the detectors need to rotate an angle of 45 degrees as mentioned above.

If the distance between the detector and the stop-line is fixed, the minimum green time has to be so long that all cars and motorcycles in Figure 78 can discharge. This is presented by formula (53).

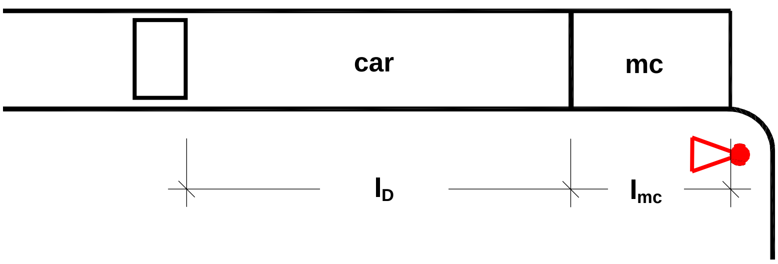


Figure 78: Determination of the minimum green time in case of using time headway

$$\min t \geq \frac{l_D}{l_{Fz}}.t_B + t_{mc} \qquad (53)$$

where t_B = mean time required between two cars passing the stop-line,

l_{Fz} = length of car (6m),

t_{mc} = necessary green time for motorcycles discharging (see section 5.3).

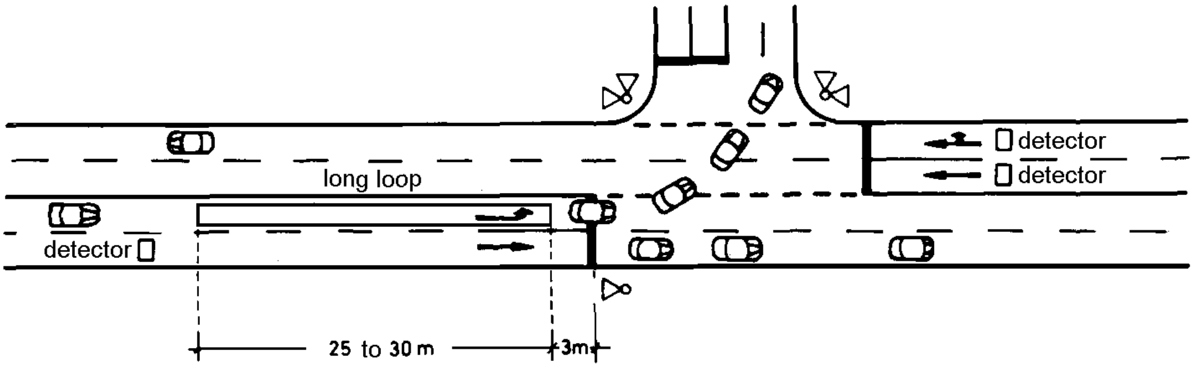


Figure 79: Example for the arrangement of a long loop

(FGSV, 2009)

In addition, RiLSA proposed another kind of detector to detect time headway, which is the long loop as illustrated in Figure 79. However, this kind of long loop ***cannot be applied under the mixed traffic condition***, especially in case of motorcycle with smaller size and lower amount of metal than that of car. For example, the long loop cannot detect a motorcycle that is slowly moving or standing inside it, because in this case there is no or very little inductance from the motorcycle. Therefore, it may be said that the long loop is unsuitable for motorcycle vehicles.

6.2.3. Degree of occupancy

According to FGSV (2009), the degree of occupancy is used to assess traffic flows taking traffic volume, speed, and vehicle length into account. Therefore, ***this control parameter cannot be applied to MCDs*** because it relates to counting the number of vehicles, which is impossible in case of motorcycles.

In reality, many motorcycles can arrive at one detector at the same time on a lane. Therefore, the same degree of occupancy can sometimes give different traffic volume. It means that the degree of occupancy does not reflect truly the characteristics of traffic flow under mixed traffic condition. Consequently, the control strategy does not achieve the effects as expected.

6.2.4. Congestion and queue length measurement

In Figure 75, detectors are arranged to detect congestion of car traffic as mentioned in RiLSA. These detectors are at least 6 m long in order to ensure that they are really occupied in case of congestion (see Figure 80). It means that the traffic situation in which the detector completely lies between two cars is avoided (in this case, even congestion is occurring, but the detector does not recognize).

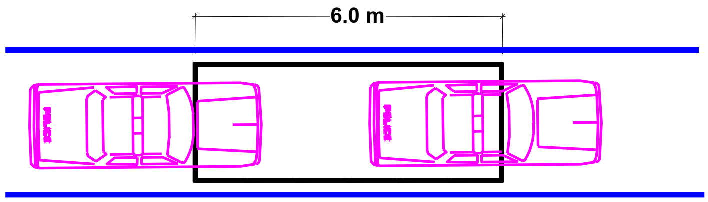


Figure 80: Congestion detection for car traffic

However, such dimension of detector is unsuitable for detecting congestion under the mixed traffic condition. For example, this following traffic situation (Figure 81) shows that even congestion is occurring, but the detector does not recognize.

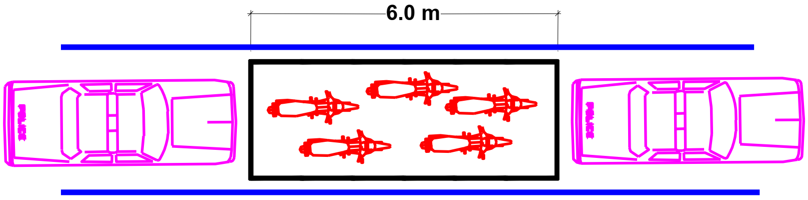


Figure 81: An example of unsuitable dimensions of detector for congestion

If the detector is shortened to 1.0 m (short detector) and rotated an angle of 45 degrees to increase the inductance of two-wheel vehicles, the above situation can be avoided due to flexible manoeuvres of motorcycles (see Figure 82).

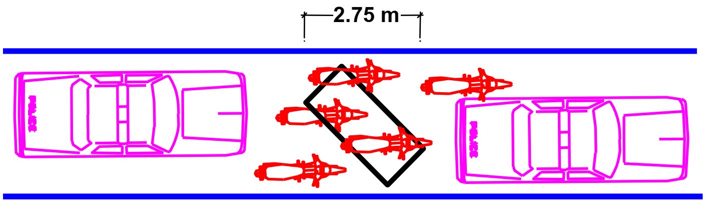


Figure 82: An example of congestion detection

This kind of detectors is arranged in the intersection layout as presented in Figure 76.

6.3. Conclusions

In general, the biggest problem in collecting traffic data online in MDCs is traffic volume. Therefore, some macroscopic control strategies related to this such as traffic-dependent signal program selection, signal program formation, cannot be applied to MDCs. The only possible one is time-dependent signal program selection.

For microscopic control level, except signal program formation (B6), all the other control strategies can be applied to MDCs because they require only the detective function of the detectors. Therefore, it is necessary to improve the detective ability of detectors for two-wheel vehicles according to individual control parameters. Then, the basic principles for each microscopic control strategy can be taken from RiLSA.

7. Draft of Guidelines for Traffic Signals in MDCs

This study aimed at developing a first draft of Guidelines for Traffic Signals in MDCs. Comprehensive efforts have been spent to verify all proposed regulations for the specific situations in MDCs. Nevertheless, it will be necessary to gain co-operations with national experts in those countries, who intend to introduce these guidelines as a national standard.

These Guidelines for Traffic Signals in MDCs have been developed based on the German standard “Guidelines for Traffic Signals” (Richtlinien für Lichtsignalanlagen – RiLSA 2009), which have been developed by the German Forschungsgesellschaft für Straßen- und Verkehrswesen FGSV (Road and Transport Research Association) and reflect the latest state of developments.

The first version of Guidelines for Traffic Signals in MDCs includes the basic aspects of traffic engineering. From RiLSA 2009, four chapters (chapter 1: Basic Principles, chapter 2: Signal Program Design, chapter 3: Inter-relations between Traffic Signal Control and Road Design, and chapter 4: Control Strategies) are modified to reflect the specific situations in MDCs. Chapter 6 (Technical Design) are kept the same as in RiLSA 2009. The other four chapters (chapter 5 Special Forms of Signalisation, chapter 7: Technical Acceptance and Operation, chapter 8: Quality Management, and chapter 9: Specification and Technical Regulations) have not been included.

Furthermore, some contents of the German Highway Capacity Manual (Handbuch für die Bemessung von Straßenverkehrsanlagen – HBS 2001), which are related to traffic signals are presented in the annexes of these Guidelines. These include details on the traffic load, traffic flow quality, and traffic engineering calculations of a single signalised intersection.

The users of these guidelines are expected to follow these basic principles in the sense of the generally aspired standardisation. Since not all problems occurring in practical application can be completely covered by guidelines and technical progress as well as specific local characteristics having to be considered individually in a suitable way, it is assumed that experts are willing to and capable of modifying the determined principles autonomously, if necessary, based on their expertise and knowledge. The guidelines, therefore, provide recommendations and suggestions on a series of problems, giving a framework for autonomous engineering action.

Besides these present guidelines, it is necessary to consider other available regulations, instructions, and guidelines.

The full text of the drafted Guidelines for Traffic Signals in MDCs is presented in Annex A of this study.

8. Conclusions and Recommendations

8.1. Conclusions

8.1.1. Statement on problems at traffic signals

At most of the signalised intersections in MDCs, traffic process occurs disorderly and ineffectively. Under traffic engineering aspect, three major problems at traffic signals in MDCs still exist, which are traffic safety, intersection layout design, and signal program design. Of these three problems, traffic safety is a consequence of the other two problems under the mixed traffic condition.

Therefore, this study focuses on solving the problems of intersection layout and signal program design including traffic signal control strategies in order to improve traffic safety and increase traffic flow quality.

8.1.2. Measures for solving problems

8.1.2.1. Applicability of RiLSA in MDCs

The analysis of RiLSA in chapter 3 confirmed that RiLSA could be applied to MDCs with some modifications of intersection layout, signal program elements as well as control strategies. These modifications mainly based on the criteria of traffic safety and traffic flow quality.

8.1.2.2. Intersection layouts

The intersection layouts have to consider all types of vehicles: motorised traffic (car and motorcycle), public transport (buses), pedestrians, and cyclists. Measures for the layout of pedestrian and cyclist facilities can be taken from RiLSA, therein.

For motorised traffic, going from the observations at intersections, during the red time the motorcycle drivers always try to get in front of cars. Therefore, the main ideal in designing the intersection layout is to give the motorcycle riders an opportunity to get in front of cars. This solution utilized the red time to make traffic process stable on the approach before the green time starts. Hereby, it improves traffic safety, increases traffic flow quality and enhances capacity of intersections, as well, this solution of the intersection layout also allows to apply the leading green time for motorcycles waiting ahead in order to reduce the number of conflicts between motorcycles and pedestrians. However, on the exclusive right-turning lanes as well as right-turning carriageways, it is not necessary to use this solution because right-turning movements are not as critical as left-turning or go-through movements.

For public transport, if the road has more than 2 lanes and a high frequency of buses, an exclusive lane for buses should be established. Hereby, signalisation of buses can be taken from RiLSA.

However, it is noted that all the existing intersection layouts have been designed for car-oriented traffic already. Therefore, the adjustments of intersection layouts should only be implemented by marking. Only exceptional cases, the constructional measures will be used.

8.1.2.3. Signal program elements

- **Saturation flow**

Unlike the conventional approach on saturation flow under the mixed traffic condition by using the equivalent number converting motorcycles into passenger car unit, this study gave the result of saturation flow depending on traffic volume of cars and motorcycles as well as homogeneous saturation flow of car and motorcycle. This is completely reasonable, because:

- firstly, saturation flow depends on the composition of traffic flow (proportion of motorcycles and cars in the traffic flow). Different proportions of motorcycles will give different results of saturation flow.
- secondly, saturation flow depends on “degree of mixture” of traffic. The more mixture of traffic occurs, the lower is the saturation flow (in this study, the adjustment factor f represents for degree of mixture of traffic).

In general, at each intersection, it is sufficient to determine saturation flow in peak hours, normal hours, and off-peak hours to develop the signal programs.

- **Cycle time**

From the concept of saturation flow, this study provided the new formulas to calculate the optimal delay cycle time and the minimum necessary cycle time. The result shows that the higher the degree of mixture of traffic, the longer is the cycle time. And the more motorcycle traffic volume occurs, the longer cycle time is.

However, the maximum cycle time should be 120 (150) s. Otherwise, the waiting time will be too long.

- **Green time**

Depending on the method for calculating the cycle time, the green time will be correspondingly calculated. This study provides the formula to calculate the green time according to the method for calculating the minimum necessary cycle time.

The minimum green time in the fixed-time signal program is 10 s to ensure that all motorcycles waiting in the head-start area can be discharged during this period of time.

- **Amber time**

Two major elements affecting the amber time are the speed limit at traffic signals, and the deceleration rate of vehicles. Therefore, the amber time is determined depending on these two elements that were stipulated in the traffic law of each country.

In MDCs, the amber time 3 s is determined for the case of intersections inside densely populated areas, and 4 s for the case of intersections outside densely populated areas.

- **Intergreen time**

Intergreen time in this study is determined according to the German method, in which the clearing speed of a vehicle must be modified depending on the speed limit at traffic signals. The lower the speed limit at traffic signals, the lower is the clearing speed.

In Vietnam, the speed limit for motorcycles inside densely populated areas is 40 km/h, therefore the clearing speed must be lower than that in RiLSA. Depending on cases of the go-through vehicle or the turning vehicle, the clearing speed is 8 m/s or 5 m/s, respectively.

8.1.2.4. Control strategies in MDCs

All the control strategies related to counting traffic volume usually are not possible in MDCs because of motorcycle traffic. Therefore, only the macroscopic control level A1 (time-dependent signal program selection) could be applied to MDCs.

Regarding the microscopic control level, fixed-time signal control and traffic-actuated control can be applied to MDCs, in which traffic-actuated signal control includes green time request, time headway, demand phase, and time-offset adjustment.

According to the German control strategies, all the traffic-actuated control strategies above use only the detective function for vehicles. Therefore, for two-wheel vehicles (motorcycles), it is necessary to have measures to enhance the detective ability of the detectors. The proposed measure is rotating the inductive loops to an angle of from 30 to 45 degrees.

8.1.3. Draft of Guidelines for Traffic Signals in MDCs

Based on the German Guidelines for Traffic Signals RiLSA edition 2009 and some necessary modifications above, the first draft of Guidelines for Traffic Signals in MDCs was compiled. However, it should be noted that this draft is basically written based on the Vietnamese traffic law as well as on the traffic data collection in Vietnam, where many typical motorcycle dependent cities exist.

8.2. Recommendations

With the results of this study, there are following recommendations:

- Testing this study at some signalised intersections for fixed-time signal control in Hanoi or Ho Chi Minh City in Vietnam.
- During the period of time for testing (several months), all of the relevant elements must be checked carefully and collated with the theory to correct if necessary.
- Testing one of these intersections with traffic-actuated signal control to collate with theory.
- Establishing the national Vietnamese standard based on these Guidelines for Traffic Signals in MDCs.

Other motorcycle dependent cities can consider these guidelines to modify and apply according to the traffic law of the specific nation.

Further research related to traffic signals in MDCs should be considered:

- Impacts of traffic signal control on environment in MDCs.
- Quality management for traffic signals in MDCs.
- Improvement and application of detection devices for motorcycle traffic.

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11/1985.

Annexes

Section Transport Planning and Traffic Engineering
Technische Universität Darmstadt

Guidelines
for Traffic Signals in
Motorcycle Dependent Cities

Based on RiLSA Edition 2009
(Richtlinien für Lichtsignalanlagen)
of
The Federal Republic of Germany

Adapted from FGSV, 2009

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0. Introduction

The first draft of Guidelines for Traffic Signals in Motorcycle Dependent Cities (MDCs) has been developed in a doctoral dissertation by Mr. Do Quoc Cuong at the Institute of Transport Planning and Traffic Engineering at Technische Universität Darmstadt, Germany. Comprehensive efforts have been spent to verify all proposed regulations for the specific situation in MDCs. Nevertheless, it will be necessary to gain co-operations with national experts in those countries, who intend to introduce these guidelines as a national standard.

Traffic signals have become a very important operational measure in traffic management in developed as well as in developing countries. The implementation and operation of traffic signals must consider the specific characteristics of all road user groups as well as all different types of vehicles.

In developed countries, where traffic is usually dominated by cars, very comprehensive experience with traffic signals is available. But this experience cannot be transferred to other countries and cities without verification and necessary adaptations to consider the different conditions. Consequently, those cities which have traffic with a very high proportion of motorcycles need special considerations regarding the implementation and operation of traffic signals.

Therefore, the experiences gained with traffic signals in developed countries with a domination of car traffic can be a sound foundation for establishing guidelines for traffic signals in motorcycle dependent cities (MDCs). But specific regulations are necessary to consider the different characteristics of motorcycle dominated traffic.

These Guidelines for Traffic Signals in MDCs have been developed based on the German Guidelines for Traffic Signals (Richtlinien für Lichtsignalanlagen – RiLSA 2009), which have been published by the German Forschungsgesellschaft für Straßen- und Verkehrswesen FGSV (Road and Transport Research Association) and reflect the latest state of developments.

The first version of Guidelines for Traffic Signals in MDCs includes the basic aspects of traffic engineering. From RiLSA 2009, four chapters (chapter 1: Basic Principles, chapter 2: Signal Program Design, chapter 3: Inter-relations between Traffic Signal Control and Road Design, chapter 4: Control Strategies) are modified to reflect the specific situation in MDCs. Chapter 6 (Technical Design) are kept the same as in RiLSA 2009. The other four chapters (chapter 5: Special Forms of Signalisation, chapter 7: Technical Acceptance and Operation, chapter 8: Quality Management, and chapter 9: Specification and Technical Regulations) have not been included in this first draft version of Guidelines for Traffic Signals in MDCs.

Furthermore, some contents of the German Highway Capacity Manual (Handbuch für die Bemessung von Straßenverkehrsanlagen – HBS 2001), which are related to traffic signals are presented in the annexes of these guidelines. This includes details on the traffic load, traffic flow quality, and traffic engineering calculations of a single signalised intersection.

The users of these guidelines are expected to follow these basic principles in the sense of the generally aspired standardisation. Since not all problems occurring in practical application can be completely covered by guidelines and technical progress as well as specific local characteristics having to be considered individually in a suitable way, it is assumed that experts are willing to and capable of modifying the determined principles autonomously, if necessary, based on their expertise and knowledge. The guidelines, therefore, provide recommendations and suggestions on a series of problems, giving a framework for autonomous engineering action.

Besides these present guidelines, it is necessary to consider other available regulations, instructions, and guidelines.

1. Basic Principles

1.1. General Remarks

Traffic signal systems are installed to improve traffic safety and traffic flow quality.

Traffic signal control substantially determines the traffic management in road networks, along arterials, and at isolated intersections. It is an important tool within the framework of a higher-level traffic concept, in which measures aiming at speeding up public transport, safe guidance of pedestrians and cycle traffic, bundling of motorised traffic on certain routes etc. fit into each other. As a dynamic element, traffic signal control is an important component of traffic management.

Since traffic signal systems directly intervene in traffic by alternatively stopping or releasing traffic flows which share conflict zones, they have to be designed, implemented and operated very carefully.

The project planning of a traffic signal system covers the road traffic engineering design, the design and calculation of signal programs, the traffic engineering description of control as well as their integration in the control of other networks.

Road space layout, traffic layout and signalisation have to form an integrated whole.

The individual design components of the road, for example, the division of approaches into lanes and head-start areas for motorcycles, the guidance of pedestrians and cyclists, and the signalisation of the individual traffic streams have to be coordinated in a way that the preconditions for safe traffic flow are given under all operational conditions and for all traffic loads.

1.2. Criteria for the Use of Traffic Signal Systems and the Effects to be Achieved

1.2.1. Traffic Safety

The set-up of a traffic signal system has to be considered if accidents have to be expected or have occurred repeatedly which might be prevented by traffic signal control, and if alternative measures (such as speed limits, overtaking prohibitions or constructional crossing aids for pedestrians or cyclists) have proved to be ineffective or not promising. For example, noticeable characteristics of such situations are:

- clustering of priority accidents
    - due to too high traffic volume or too high speeds on the higher-level road,
    - due to insufficient visibility at the intersection or unclear priority,
    - due to insufficient capacity,
- clustering of accidents between left-turners and opposing traffic or
- clustering of accidents between motorised vehicles and crossing cyclists or pedestrians.

If persons are in danger (e.g. old people, handicapped, and children) who are in particular need of protection and who regularly have to cross the road at a certain site and if within reasonable distance no safe crossing is possible, a traffic signal system has to be implemented disregarding the number of persons to be protected or the accident situation, if the protection may not be achieved otherwise.

Traffic signal systems can also be installed in case of special requirements of police, ambulance and emergency vehicles.

1.2.2. Traffic Flow Quality

The quality of traffic flows in networks, in subsections and at intersections can be improved by traffic signal systems. In many cases, the implementation of traffic signals can avoid a constructional enlargement of the road facilities.

The quality of public transport and for non-motorised road user groups can also be improved significantly by suitable priority measures.

Traffic signal systems can also be used in order to guide traffic in the whole road network in a favoured way.

For the purpose of access control, traffic signal systems can be implemented:

- to protect subsections or network areas from an overload,
- to keep priorities for public transport vehicles in subsections and
- to keep sections with intensive roadside use free of congestion.

1.2.3. Fuel Consumption and Emissions

Basically, all measures aiming at a smooth speed level for traffic flows within the permissible speed reduce fuel consumption, noise emissions, and air pollution. A low number of stops, a smooth passing through several intersections and influencing the route choice can decrease fuel consumption and emissions. This is particularly important for subsections featuring great pedestrian and cyclist streams and intensive roadside use. Signal programs adapted to traffic flow variations equally contribute to reduce fuel consumption and emissions.

1.2.4. Balancing of Conflicting Objectives

The objectives of traffic signal control are mainly determined by the demands, interests, and requirements of the authorities, the individual road user groups, the operators, and the residents concerned. Since all expect that traffic signal systems are to ensure safe, fast, and comfortable traffic flow, their objectives are frequently conflicting, because the legitimate objectives of the individual groups generally cannot all be fulfilled at the same time. Even between the aspired impacts with regard to improved traffic safety, high traffic flow quality, priority for public transport, reduced fuel consumption and less environmental pollution by emissions as low as possible, conflicts may come up, too.

When planning a project of traffic signal systems, all basic requirements have to be balanced. Usually, only combined measures reflecting the conflicting objectives allow a good compromise.

1.3. Traffic Signals and Signal Sequences

Traffic signals are light signals. For traffic signals controlling traffic flows at intersections, on approaches and other road sites, the term of “variable light signals” is used in traffic law.

In these guidelines, further descriptions of signal lamps, and constructional guidelines will be presented in chapter 5 “Technical Design”.

Traffic signals for motorised traffic usually have the following signal sequence: GREEN – AMBER – RED – GREEN. In some cities, the signal sequence GREEN – AMBER – RED – RED/AMBER – GREEN is also used for motorised traffic. When traffic signal systems are operated after longer intervals only, or when there is confusion due to successive signal heads, the sequence DARK – AMBER – RED – DARK is permitted. For left-turners, a green arrow can be shown on the left after the intersection if all conflicting traffic streams are stopped by RED (diagonal green). The signal sequence is then DARK – GREEN - DARK. To switch an additional green time for right-turners, usually, a two-lens signal head with the sequence DARK – GREEN – AMBER – DARK is chosen. In some cases, it may be sufficient to indicate by a one-lens signal head depicting a green arrow. Signals for motorised vehicles also address all other road users on the lanes, if not signalised separately. Green arrows in the signal lens must only be shown when all conflicting and permitted traffic streams are stopped.

If not jointly signalised with motorised traffic, buses should be signalised by their own special traffic signals, especially on ring-roads or arterials where the frequency of buses is high.

For exclusive signals of buses, the signal lens showing a horizontal white luminous bar is used to indicate for red time. The signal lens shows a white luminous bar, either vertically or rising diagonally to the left or right is used to indicate green time for a protected movement, where public transport has priority to pass the intersection. A white luminous triangle whose top points downward is used to indicate permissive green time in which public transport has to give way to other priority traffic streams simultaneously released (permissive signal).

A transition period is to be indicated by a white luminous spot meaning “stop to be expected”.

The signal sequences STOP – GO (permissive) – STOP and STOP – GO (permissive) – STOP TO BE EXPECTED - STOP are shown.

Further signals mainly serve in operation control, for example the door-closing signal might be used.

If the motorised traffic lanes are routed across a public transport lane in the lateral side or on the central reservation and outside a junction, a two-lens signal head with the signal sequence DARK – AMBER – RED - DARK can be implemented for motorised traffic, and the exclusive signals for public transport can also be implemented. The two-lens signal head must not be a part of full signalisation.

Traffic signals for pedestrians follow the sequence: GREEN – RED – GREEN.

The three-lens signal heads for cyclists have the signal sequence GREEN – AMBER – RED – GREEN.

An amber flashing light may be used to warn of danger. If symbols of a vehicle type (black pictograms on yellow luminous ground) are added, it means that other types of vehicles have to pay attention and give way to that type of vehicles. Possible symbols are described in chapter 5 “Technical Design”.

Basically, it is not necessary to use the countdown clock signals at intersections. However, at fixed-time signal program intersections, when using the countdown clock signals, some elements of the signal program (for example: the amber time, the intergreen time, etc.) must be considered carefully to eliminate potential accidents.

2. Signal Program Design

2.1. Terms and Definitions

The term "**signal program**" includes the signal timing of a traffic signal system which is fixed with regard to duration and assignment.

A signal program is designed in several steps, which have to be developed interdependently. The required documents and steps presented in the following sections are considered to be an example of an intersection.

The term "**signal group**" is used to imply one or more signal heads, which jointly control the assigned traffic streams and show the same signal at any time.

A phase is that part of a signal program during which a certain basic signalisation stage does not change, whereby the green times of the released traffic streams may begin and end at different points of time.

2.2. Documents and Pre-studies

A layout plan (scale 1:200 to 1:500) including the relevant local characteristics (for example roadside bordering, sidewalks and cycle paths, buildings, entries and exits, trees, masts and poles, hydrants, shafts, switchboards, longitudinal gradients, signing, markings and traffic installations) is considered to be the basis of the design task. The data on the layout plan have to be checked on site.

The layout plan in progress builds the basis for designing the signal plans.

Details on the traffic load are used to select the control strategy, to design signal programs as well as to monitor traffic flows. These contents are presented in the annex 1 of these guidelines.

Traffic volume must be counted by direction, vehicle type (especially motorcycles and cars), and individual related lane. If there are several lanes for one direction and the available data are not for individual lanes, equal distribution onto all lanes can be approximatively assumed.

The general map should show the location of the intersection in the road network as well as the neighbouring traffic signal systems. If necessary, documents on the control of these traffic signal systems must be considered.

Furthermore, besides the local planning, cycle traffic planning, use of environment surrounding, which suggest traffic volume and direction of weak road user groups such as school-children or mobility-handicapped, are relevant.

The results of accident studies, especially in the case of re-designing an existing traffic signal system have to be considered.

Starting point for local accident studies generally is an accident pin board. Many similar accidents at one intersection usually give a clear indication of a systematic shortcoming.

For detailed information on increased accident numbers and accident types, accident diagrams reflecting the accident occurrence of several years have to be consulted. If they confirm the increased number of similar accidents, it has to be checked according to the criteria mentioned in section 1.2.1, whether signal control is to be considered or whether modifications of the existing traffic signal system are required.

2.3. Signal Program Structure

2.3.1. Signal Phasing

2.3.1.1. General Remarks

When phasing signals non-conflicting, conflicting, and permitted traffic flows have to be distinguished. Non-conflicting traffic flows do not share any conflicting area and can be combined in one single phase. Conflicting traffic flows have a conflicting area and must be released separately.

Turning traffic flows that have conflicting areas with opposing traffic or with parallel pedestrians or cycle traffic flows can be released as permitted traffic flows, but they must obey prioriry rules.

The bundling of traffic streams on lanes leads to compulsory conditions to be fulfilled with regard to signal phasing.

A separate signalisation for turning traffic usually requires a separate lane marked with directional arrows.

Successively, i.e. in different phases, traffic streams of different directions may release only if they are led separately on different lanes, (e.g. straight-ahead and turning vehicles each on their own, separate lanes).

A priority traffic flow must not be added to a permitted traffic flow which has already been released, (e.g. pedestrians to an already running phase of permitted turning vehicles). This may be disregarded in exceptional cases when signalling leading green to left-turners if an auxiliary signal (amber flashing light) indicates them released priority traffic streams.

If a traffic flow is released by a direction arrow, all other streams sharing its conflict areas have to be shown red. This equally applies to green indicated by a combination arrow.

If not all lanes of an intersection approach receive green at the same time, the direction arrows on the signal lenses may only be omitted if the different lanes are separated by constructional measures. There must be no doubt about which signal head is assigned to which direction.

If an intersection approach has a separate turning lane which is signalised separately by direction arrows, all other directions of this intersection approach can then be signalised without direction arrows (full signal lens).

If all traffic flows of a multi-lane approach have to turn right or left (turning enforcement sign), parallel traffic flows, e.g. pedestrians or cyclists, will not be released as permitted traffic flows.

2.3.1.2. Left-turning Movements

Protected left-turning movements must be aspired for reasons of traffic safety, especially in non-urban areas. This is more urgent,

- the faster opposing traffic,
- the more rapidly left-turning traffic flow is led,
- the heavier left-turning traffic or a conflicting traffic flow to be crossed,
- the more restricted the visibility of permitted traffic streams and
- the more attentions of left-turning drivers are demanded due to the increasing number of possible conflicts (multi-lane opposing traffic, jointly right-turning vehicles, and parallel released pedestrian and cycle traffic).

If being allocated two or more exclusive lanes on an approach, left-turning vehicles have to be protected by signalisation.

Temporarily protected left-turning movements arise by means of lagging and leading green time if the green times of opposing traffic flows are offset. They allow left-turning vehicles to clear the intersection unimpeded by opposing traffic after their green time has ended or before the green time of the respective opposing direction starts.

Lagging green can always be used and does not pose any problems at whatever duration because left-turners can quickly recognize opposing traffic that has stopped. It is recommended that a one-lens signal head (diagonal green) located beyond the intersection should be used to indicate the lagging green time for left-turners. However, a two-lens signal head can also be used in which an amber-flashing light mounted above the diagonal green is used to warn left-turners against released opposing traffic flows.

Leading green, for safety reasons, must be always indicated by a two-lens signal head. After the diagonal green's extinction, left-turning vehicles have to be warned against starting opposing traffic and of simultaneously released priority pedestrians and cyclists by an amber flashing light mounted above the directional signal.

It is recommended to use the leading green time for head-start left-turning motorcycles because motorcycles often cause a high number of the critical conflicts, especially with simultaneously released priority pedestrians and cyclists.

If a leading and lagging green time of left-turning vehicles is indicated by a one-lens or two-lens signal head, the beginning and ending of the green arrow have to be determined from the intergreen time between left-turners and opposing traffic, parallel pedestrians, and cyclists. If the driver's misconception on the same approach at the diagonal green cannot be precluded, the green time of the green arrow must be within the green time of the approach.

Permitted left-turning movements should only be applied if at least one of the two conflicting traffic streams is of low volume. The queuing left-turners have to be given an opportunity to clear the intersection.

If the left-turning vehicles do not discern that they have to give way to pedestrians and simultaneously released cyclists, an auxiliary signal should be installed directly at the crossing. Its flashing light has to be activated while pedestrians and cyclists are clearing the intersection.

2.3.1.3. Right-turning Movements

On the approaches without triangular islands, normally, right-turners do not require any signalisation by directional signals. Signal control by directional signals should be considered in cases of heavy lateral traffic flows, or when it results in special advantages for the signal phasing.

If directional signals are not used, an auxiliary one-lens signal (amber flashing light) may warn against possible conflicts with parallel priority pedestrians and cyclists if the right-turners do not clearly recognize the obligation to wait. Then, the auxiliary signal head has to be arranged directly at the pedestrian or cycle crossing. The amber flashing light has to be activated even during the pedestrians' and cyclists' clearance time.

The auxiliary signal may also be used in exceptional cases at intersections (for example, the stop-line is far away from the pedestrian crossing), turning vehicles do not reckon any more with pedestrians and cyclists. It should be used if the pedestrian crossing is not released in every cycle time.

At intersections featuring right-turning lanes, the signal program structure sometimes provides additional green times for right-turning vehicles.

These are usually indicated by a two-lens signal head showing two directional arrows with the signal sequence GREEN – AMBER – DARK (see **Figure 5.18** in chapter 5).

The beginning and end of the additional green time must be determined by intergreen time calculation to conflicting traffic flows released previously and afterwards.

When added to the main phase as leading or lagging green, right-turning vehicles usually undergo a short green time disruption. In the first case (leading green), an intergreen time between clearing right-turners and pedestrians is required, who reach the conflict area with a time lead to the right-turning vehicles released again during the general green time. In the second case (lagging green), an intergreen time between clearing left-turners of opposing traffic (at intersections) and right-turners released by directional signals entering afterwards is required (see sections 2.7.5 and 2.7.6). In these cases, the period of green time on the green arrow should directly follow the green time of the entire approach. Therefore, a one-lens signal head with the sequence DARK – GREEN – DARK is enough.

Like left-turning motorcycles, it is recommended to use leading green time for head-start right-turning motorcycles.

On the approaches with triangular islands, if right-turners are routed on a carriageway, they can be controlled without signalisation, and are led as waiting obligation vehicles to the crossing roads (minor and major road signs). In order to increase attentions to the priority rule, crossing pedestrians and cyclists, a one-lens auxiliary signal head (amber flashing) should be used.

On the right-turning carriageway, it is not necessary to separate motorcycles from car traffic because right-turning traffic flows do not cause as many critical conflicts as go-through or left-turning traffic flows do.

If road signs and markings of the carriageway are insufficient to ensure traffic safety for pedestrians and cyclists crossing the carriageway, the signalisation with the sequence DARK – AMBER – RED – DARK for vehicles at the crossing may be used. This signalisation can be controlled independently from the other parts of the intersection (for example: also traffic-actuated by requests of pedestrians).

In these cases, an auxiliary signal can be added to increase attention for the priority rules. In cramped cases, the auxiliary signal may be combined with the signal head at the pedestrian crossing.

The separated signalisation with a three-lens signal head is required for right-turning movements if:

- there are two right-turning lanes,
- vehicles turn too rapidly. Consequently, pedestrians and cyclists are not paid enough attention,
- visibility is impeded and
- pedestrian and cycle traffic flow is too heavy.

Hereby, signal phasing has to ensure that opposing left-turners do not turn up at the exit of the right-turning carriageway during the right-turners' green time.

The right-turning movements with a green arrow plate (a green arrow on the dark background) are allowed during the red time at signalised intersections if released traffic flows are not impeded or endangered.

By this possibility of the right-turning movements, it will:

- drop the waiting time for right-turners,
- enhance the capacity of right-turners, especially motorcycles
- require a short queuing space for right-turners.

For the traffic safety reason, the green arrow plate must not be used if:

- the protected left-turning movement of opposing traffic is signalised,
- the temporarily protected left-turning movement of opposing traffic is led by a diagonal green arrow,
- arrows stipulating the direction for right-turners are valid on lens of the signal head,
- many marking lanes are available for the right-turning movements,
- the traffic signal system mostly serves traffic safety on the roads to school.

A precondition for applying the green arrow rule is a sufficient visibility on all released traffic streams. This must be given at the stop-line of right-turners. Hereby, according to the green arrow rule, the driving vehicles do not block the way of released traffic streams if they drive up to the stop-line and must stop there again.

At intersections, if the blind, visual- or mobility-handicapped cross very often, the green arrow rule must not be used. In exceptional case, if the green arrow rule is designed, and the blind, visual- or mobility-handicapped cross very often, the traffic signal system must be equipped with acoustic devices or other suitable devices.

2.3.1.4. Public Transport (Bus)

Regularly scheduled buses can use their own special signals (compare chapter 1.3), if separate phases are provided to give priority to public transport vehicles. This avoids the danger of misconception by car drivers.

When using a bar signal light for turning public transport vehicles, normally a separate phase including the necessary intergreen times is required. Hereby, public transport may only turn during very short green periods, long waiting time affecting private traffic may arise.

If a permissive signal is used and if the opposing traffic flow is not too heavy, part of the general green time can often be used for turning, thus waiting times are reduced. When ending the permissive signal, the possibly longer clearance times of public transport have to be taken into account. Especially, in case of higher traffic volumes of the permitted traffic streams, the application of permissive signals may be restricted.

2.3.1.5. Pedestrian Traffic

Traffic signal systems for pedestrians are installed according to the recommendations given for pedestrian facilities.

They are usually operated by requests, i.e. pedestrians can request their own green time. The waiting time until release should be as short as possible. An information signal (e.g. text: "signal activated") can indicate to pedestrians that their request has been registered.

The vehicle signals have to be switched so that all vehicle streams passing the crossing are given RED simultaneously. So pedestrians who watch the vehicles of one direction stopping do not step onto the carriageway while the opposing traffic flow has still got GREEN.

For Green Wave road sections, the signal programs of the pedestrian traffic signal systems have to be integrated into coordination. Hereby, pedestrians can be considered either once in every cycle or in case of low pedestrian volumes only in those cycles when a pedestrian green time is requested. The abortion of the Green Wave for motorised traffic has to be accepted to protect pedestrian traffic if long cycle times entail long waiting times. Green times not needed for motorised traffic have to be used to extend the pedestrian green time.

At traffic signal systems for pedestrians, usually it is recommended to have GREEN for vehicles and RED for pedestrians at the basic stage (see **Figure 2.1**).

The system uses the complete signal sequence. After a pedestrian green time has been requested, the vehicle signals turn from GREEN via AMBER to RED. The pedestrian phase being finished, the vehicle signals return to GREEN (basic stage). In case of repeated requests, pedestrians are given green at the earliest after the intergreen time and a fixed minimum time called reservation time. This reservation time must not be shorter than the minimum green time for vehicles.

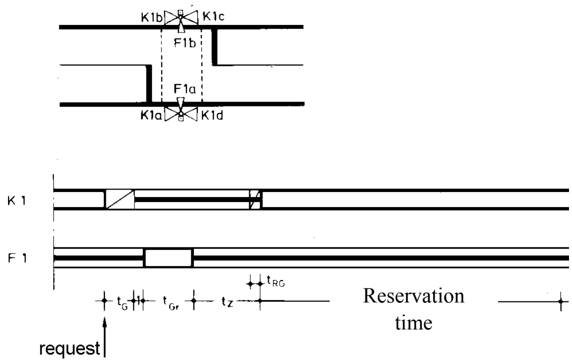


Figure 2.1: Example of a pedestrian traffic signal system, basic stage: GREEN for vehicles and RED for pedestrians

Furthermore, an operating mode may be selected, the basic signalisation stage of which shows DARK for vehicles, RED, however, for pedestrians. After a green time request from pedestrians, the vehicle signals turn from DARK via AMBER to RED. When the pedestrian phase is being terminated, the vehicle signals fall back to the basic stage (DARK) (see **Figure 2.2**).

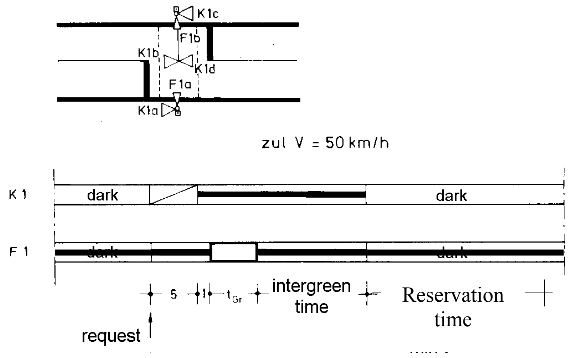


Figure 2.2: Example of a pedestrian traffic signal system, basic stage: DARK for vehicles and RED for pedestrians

The signalisation of pedestrians and turning vehicles is basically provided for the permitted or protected traffic flows.

Parallel pedestrians have to be always signalised separately if vehicles turn on more than one lane.

Separate signalisation is recommended if:

- turning traffic is routed quickly, is heavy or has to take into account several possible conflicts,
- the visibility between vehicles and pedestrians is impeded,
- the pedestrian streams are heavy,
- the left-turning vehicles on high-speed roads have a difficulty in estimation for time gaps between opposing traffic vehicles.

Separate signalisation offers the full signal protection. However, it leads to longer waiting times for all road users than designing a signal phasing with permitted traffic streams.

The green times of pedestrians must not be added to a permitted traffic stream which has already been released. Uncertainties and hazards may come up if pedestrians cannot use their right of way and if turning vehicles are taken by surprise when priority pedestrians occur unexpectedly. This requirement has to be considered especially in case of traffic-adaptive control. Leading green for left-turners is the only exception.

Signalisation at successive crossings are operated by either coordinated or separate signalisation with central reservations or separating strips, depending on local boundary conditions or other given determinations of traffic operation.

For simultaneous signalisation, on the edges of the carriageway and on separating strips the same signal is shown simultaneously. Hereby, the green time duration of pedestrians should allow them starting at the green time beginning and crossing at the arithmetical clearance speed to reach at least the centre of the second carriageway before the green time ends.

However, such a signalisation cannot avoid the fact that pedestrians having started during the second half of the green time have to wait on the separating strip or the central reservation (see **Figure 2.3**).

For progressive signalisation, if pedestrians are not to stop on the central reservation or the separating strip, especially in case of lacking space for them to stop, the pedestrian signal here may turn from GREEN to RED earlier than the signal on the opposing side of the carriageway.

But the disadvantage is that pedestrians, who first stop because the signal arranged on the central reservation shows RED, may be tempted to violate RED as oncoming pedestrians are still given GREEN. Furthermore, for permitted conflicting signalisation it cannot eliminate that right-turning drivers misconceive the red signal of pedestrians on the central reservation, and try to impose their wrongly assumed priority. This can be avoided by using suitable visors or signal optics for the pedestrian signals or by a flashing auxiliary signal.

Figure 2.4 shows an example of progressive signalisation with three signal groups.

Separate Signalisation must be considered if one of two successive crossings has to be given GREEN or RED earlier than the other due to motorised traffic operation. It may be useful for reasons of clarity to release pedestrian traffic only if the signals of both crossings can show GREEN at the same time.

However, because of the following reasons, it is usually better to release earlier the pedestrian streams on that intersection approach on which motorised traffic has already been shown RED.

- Pedestrians are not tempted to start on RED, as in the case when it is still shown RED, although the vehicles on the approach concerned have already stopped.
- An earlier release may allow pedestrians starting at the green time beginning to cover the first crossing and to step on the second crossing before right-turning vehicles have arrived.

Possible longer green time at one crossing should not be given, if this led to waiting time on a small separating strip.

If the green times at both crossings are offset so that pedestrians always have to wait on the central reservation or the separating strip, the following measures can be recommended to improve the situation:

- expansion of the queuing space, e.g. by narrowing the lane width or by widening the crossings,
- possible arrangement of barriers which must be walked around in case of slightly staggered crossings
- reduction of waiting time by traffic-adaptive control.

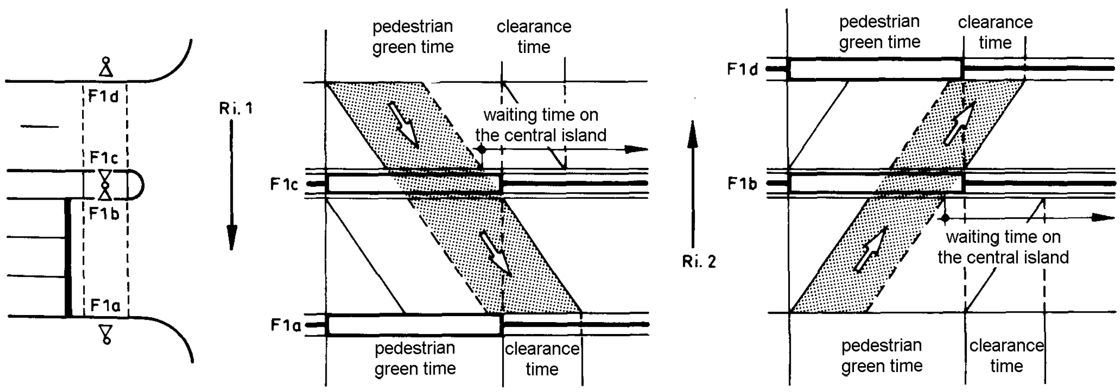


Figure 2.3: Simultaneous signalisation of successive crossings with one signal group

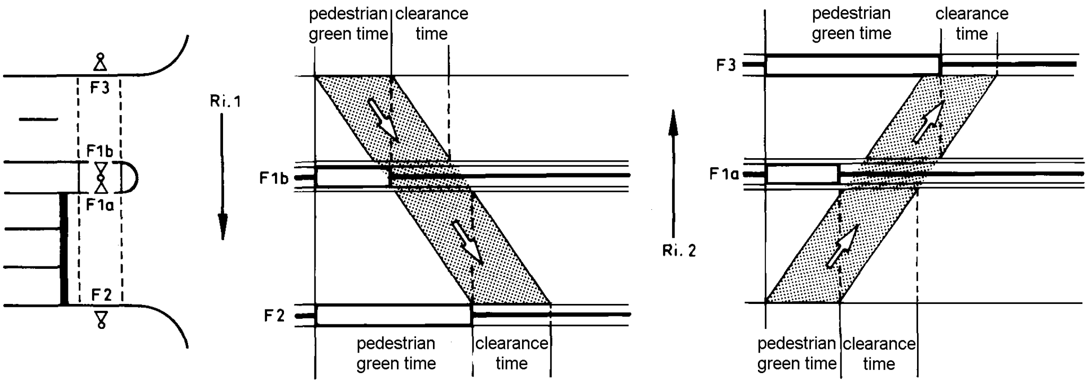


Figure 2.4: Progressive signalisation of successive crossings with three signal groups

If two signal heads are arranged within a short distance at separately signalised pedestrian crossings, pedestrians run the risk of misconception of the green signal of the second crossing and relating it to the first if the red signal on the central reservations or the separating strip has failed. This may lead to dangerous situations and requires the complete or partial de-activation of the signal system at the crossing concerned by signal safeguarding.

If the pedestrian crossings equally provide access to public transport stops, the green time crossing the carriageways has to be switched so that pedestrians waiting at the roadside can still reach an approaching public transport vehicle.

All-green for pedestrians can be applied in case of high pedestrian traffic volume and low motorised traffic volume. Pedestrians receive simultaneously green time at all crossings while all vehicle signals show RED. Such a pedestrian phase implying all-RED for vehicle traffic avoids turning vehicles which may endanger pedestrians.

2.3.1.6. Cycle Traffic

There are **three basic types of signalisation** for cycle traffic:

- joint signalisation with motorised traffic
- joint signalisation with pedestrian traffic
- separate signalisation

The same basic type of signalisation for cycle traffic should be used for the coequal approaches at intersections, and along the major direction connecting some intersections.

The separate signalisation for cycle traffic should only be applied after considering jointly signalisation with motorised traffic, and pedestrian traffic. Consequently, advantages for traffic safety, acceptance, and traffic flow quality have to be justified the additional cost and operation expenses.

The basic structure of signalisation should encourage the acceptance of cyclists, therefore:

- waiting time should be as short as possible,
- the cyclists should cross the separate carriageways without stopping,
- the green times should be determined so long that all cyclists can be released within individual cycle times
- the green times should not be significantly shorter than that of parallel motorised traffic.

Joint signalisation with motorised traffic can be applied:

- if the cyclists are led with motorised traffic on the approaches,
- by a separate cycle lane and joining the head-start areas with motorcycles at the intersection,
- by guiding cyclists on the bus lanes if there is no separate signals for buses available,

If cycle traffic is jointly signalised with motorised traffic, when calculating the intergreen times, it has to be taken into account that the cycle's clearance times may be longer than those of motorised traffic.

Joint signalisation with pedestrian traffic has to be applied:

- when cyclists and pedestrians share a path, when cycle traffic is permitted on the pedestrian path, and when a cycle path without the liability is used,
- when the cycle path is guided with an adjacent pedestrian crossing and no separate signalisation is used,

Joint signalisation of cyclists and pedestrians should be shown by the combined symbol of cyclist and pedestrian on the optical lens of the signal head.

Separate signalisation of cyclists indicated by a three-lens signal head can be applied to cycle lanes with a non-backward cycle crossing,

- if cycle traffic can receive a separate phase or the time lead in order to reach the conflicting areas before turning vehicles arrive (see **section 2.7.5**), to be led in the mixed traffic at the end of a cycle lane, or to enter the bottle-neck areas before

successive motorised vehicles (e.g. on the exit of intersection with a narrow width),
- if at large-scale intersections and the long clearance times for cyclists, the green time for cyclists should be terminated earlier than for vehicle traffic and
- if cycle traffic is led on the bus lanes with separate signals for buses.

In case of multi-lane guidance for turning traffic, parallel cyclist traffic has to be signalised separately.

The separate signalisation of cycle traffic is not considered when joining guidance with motorised traffic on the carriageway, and when using protective lanes.

2.3.2. Number of Phases

The signal phasing and the number of phases are determined by the decision on which movements are to be protected by signalisation. If signal programs of neighbouring intersections have been coordinated, additional boundary conditions are resulted from the time-distance planning of the traffic streams.

If all permitted traffic flows shall be protected by signalisation, they usually need at least three phases at T-junctions and four phases at crossing intersections.

Normally, the permitted traffic flows are not protected by signalisation of a two-phase control unless certain turning movements are prohibited. For traffic safety reasons, signal control with more than two phases may be considered.

Regarding the cycle time duration the total necessary intergreen time should be kept as short as possible and non-conflicting traffic streams requiring more or less equal green time should be combined in one phase.

2.3.3. Phase Sequence

Regarding the capacity, the best phase sequence usually results from the total necessary intergreen time and the relevant green times, which together lead to the shortest cycle time unless the phase sequence is completely or partly determined by one of the following boundary conditions:

- The phase sequence for complex intersections may be determined by the condition that certain directions have to be released one after the other, so that queuing vehicles do not cause impediments.
- To allow heavy pedestrian or cyclist traffic flows to cross successive crossings rapidly, the sequence of a certain pedestrian and cyclist green time can impact the overall phase sequence at the intersection.
- The coordination of the signal programs of neighbouring intersections or public transport management may entail green time offsets determining the phase sequence at individual intersections.
- In order to improve the traffic flow quality, certain road user groups or individual traffic streams can receive green several times within one cycle.

The suitable phases and phase sequences can be depicted in a phase sequence plan (see **Figure 2.5**). For traffic-actuated signal control, the phases are selected by interconnecting logical and time conditions in the control algorithm.

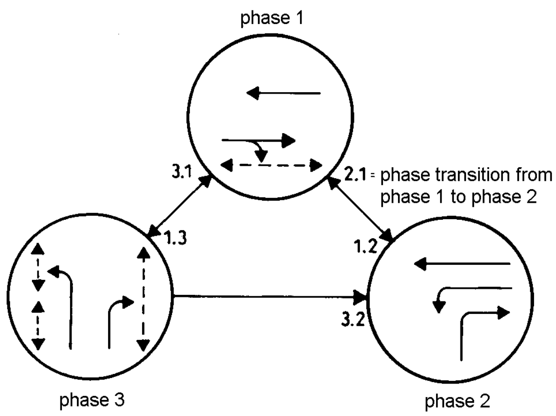


Figure 2.5: Example of a phase sequence plan

2.3.4. Phase Transitions

The change from one phase to the other is laid down in the phase transition (see **Figure 2.6**). The phase transition is the time between the end of green time of the signal group in the ending phase, whose green time ends first, and the beginning of green time of the signal group in the starting phase, whose green time begins last.

The phase transition includes at least the intergreen time necessary for changing the phases. It may also be reasonable to include boundary conditions for green and red time. (see **Section 2.7**).

Traffic-actuated interventions in phase transitions are only permitted if intergreen times and signal sequence are kept.

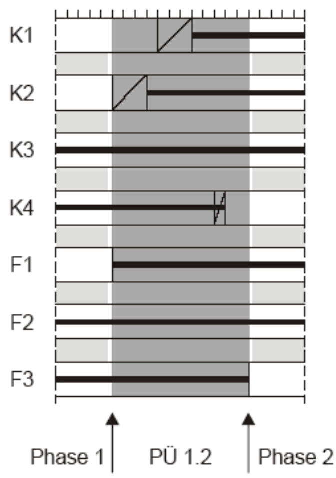


Figure 2.6: Example of a phase transition

2.4. Transition Times

For vehicle movements reasons, motorised traffic is indicated the change from green to red by the transition signal AMBER before RED. The transition time AMBER (t_G) hereby depends on the permissible speed on the approach.

- t_G = 3 s at V_{zul} = 50 km/h
- t_G = 4 s at V_{zul} = 60 km/h, and 70 km/h

Therefore, amber times may vary on the individual intersection approaches.

Separately signalised turning vehicles, travelling at the maximal speed of 50 km/h, even on approaches of V_{zul} = 70 km/h or 60 km/h, may be assigned an amber time of t_G = 3 s. At all the other signal groups with the signal sequence DARK – AMBER – RED – DARK, the amber time is to amount to t_G = 5 s.

If separate special signals for buses are applied, the transition time may be adapted to that of motorised vehicles.

A transition signal for public transport vehicles may be dropped:

- if vehicles always have to stop at the signal or
- if a signal changing from "FREE" to "RED" within the operating braking distance is not possible and
- if V_{max} = 20 km/h.

In some countries, the transition signal RED/AMBER before GREEN is used to prepares road users for the immediately following the green signal. This transition time duration is 1 s.

The transition from RED to GREEN for public transport vehicles is generally not indicated by a transition signal. At the public transport stops before traffic signal systems, however, an information signal may be useful (for example a door closing signal from 5s to 10s) to have public transport vehicles to be ready to start in time.

For separately signalised cyclists the uniform transition time should be 2 s for AMBER.

The signal sequence for pedestrians does not include any transition time.

2.5. Intergreen Times

The intergreen time is the interval between the end of the green time for one traffic stream and the beginning of the green time for the next, the conflicting traffic stream.

The shortest necessary intergreen time t_z is determined by the crossing time $t_ü$, the clearance time t_r and the entering time t_e:

$$t_z = t_ü + t_r - t_e$$

The intergreen times have to be rounded to full seconds.

The intergreen times have to be calculated for all combinations of conflicting traffic flows. Hereby, all road user groups (pedestrians, cyclists, public transport, and motorised vehicles) have to be considered as separate flows, even if jointly signalised. The relevant (maximum) intergreen time of the respective signal groups are compiled in an intergreen time matrix (see **Figure 2.7**).

In case a diagonal green is used for temporarily protected left-turning vehicles at the intersection, the intergreen time relating to this signal between left-turners and opposing traffic as well as to conflicting pedestrians or cyclists have to be identified in the intergreen matrix.

The assumptions on the intergreen time calculation given below are applicable to the standard case. Hereby, road users are expected to follow only those signals which actually address them. Local characteristics (e.g. speed limits, great longitudinal gradients on the intersection approach, particularly slow-moving vehicles) require different assumptions, which may lead to longer intergreen time.

		starting signal groups														
		K1*)	K2	K3	K4	K5	K6	K7	R1	F1	F2	F3	F4	F5	F6	F7
ending signal groups	K1*)			4			5	6		4					7	
	K2			5	8	5	5		4	2		8	8			
	K3	5	4			4			1		4	4				6
	K4		2				2							4		
	K5		3	5			4			6					3	
	K6	4	4		10	5			4			6	6			4
	K7	2														3
	R1		2	6			3			9					3	
	F1	9	7			6			4							
	F2			6												
	F3		4	6			4									
	F4		4				4									
	F5				4											
	F6	7				8			9							
	F7			5			7	6								

*) Signal group K1 includes signals K1a and K1b; the same applies to further signal groups.

Figure 2.7: Example of an intergreen time for the intersection

2.5.1. Determination of Clearing and Entering Distances

When determining the intergreen time, first of all the clearing and entering distances have to be identified. As reference lines for measuring their length, generally the centre lines of the lanes or footways allocated to the traffic streams involved have to be used (e.g. lane centre lines and crossing midways). For glancing intersections the relevant bordering lines instead of the centre lines have to be taken into account.

The clearing distance s_r is composed of the basic clearing distance s_o and a fictitious vehicle length l_{Fz}. The basic clearing distance for vehicles is the distance between the stop-line and the point of intersection with the entering route of the starting traffic stream (conflict point), for pedestrians and cyclists, if jointly signalised, the distance between the beginning of the crossing and the end of the conflict area.

The clearing of the conflict area is taken into account as much as it is necessary with regard to the safety issues and the responsibility of the entering traffic streams. It is assumed that long and big vehicles are recognised at complete length when clearing the intersection and their priority is respected when they are occupying the conflict area. The minimum necessary green times of the entering traffic flows have be considered in this context (see **Section 2.7.4**). Therefore, when calculating the intergreen times, the following fictitious vehicle lengths are used:

- bicycles: 0 m
- motorised vehicles (incl. truck trailers, buses): 6 m
- motorcycle: 2 m

If motorcycles and cars share a lane, however, the car length is always chosen when calculating the intergreen times.

The vehicles' entering distance s_e is the distance from the stop-line to the point of intersection with the clearing distance of the ending traffic stream or up to the beginning of the crossing. For pedestrians and jointly signalised cyclists, it is the distance between the beginning of the crossing and the beginning of the conflict area. If the conflict area is immediately adjacent to the starting point, the entering distance is zero.

If no distances are defined in the inner intersection area (e.g. for turning traffic flows), the intergreen time calculation has to be based on plausible distances with regard to driving geometry.

2.5.2. Crossing and Clearance Times

The crossing time $t_ü$ is the interval between the end of the green time and the beginning of the clearance time, determined for the intergreen time calculation.

The clearance time t_r is the time needed to cover the clearing distance s_r (see **Section 2.5.1**) at a clearance speed v_r:

$$t_r = \frac{s_r}{v_r}$$

The following equation applies to the crossing and clearing of straight-ahead vehicles (**Case 1**) or turning vehicles (**Case 2**):

$$t_ü + t_r \geq t_G + 1$$

It is to ensure that vehicles which cannot stop at the stop-line within the amber period of a signal change do not constitute a direct hazard for starting traffic streams, particularly pedestrians and cyclists, whose conflict zone is next to the stop line.

The intergreen time calculation has to distinguish five different cases of crossing and clearing:

Case 1: Straight-ahead moving vehicles are clearing

Disregarding the legal speed limit, the crossing time for straight-ahead moving vehicles is set at $t_ü = 3$ s. A clearance speed of $v_r = 8$ m/s has to be expected.

This leads to the following determinations:

Crossing time: $t_ü = 3$ s

Clearance speed: $v_r = 8$ m/s

Basic clearing distance: s_0 = distance [m] between stop-line of cars and conflict point, measured at the centre of the lane (see **Figure 2.8**)

Vehicle length: $l_{Fz} = 6$ m

Crossing and clearance time: $t_ü + t_r = 3 + \dfrac{s_0 + 6}{8}$

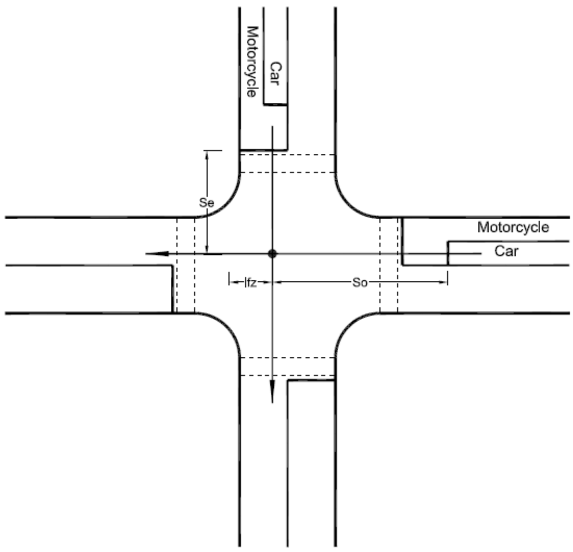


Figure 2.8: Example of the case "Straight-ahead moving vehicle is clearing / vehicle is entering"

Case 2: Turning vehicles are clearing

The crossing time for turning traffic flows is set at $t_ü = 2$ s. Then the assumed clearance speed is $v_r = 5$ m/s. At a radius of the inner lane edge of R < 10 m the clearance speed has to be reduced to $v_r = 4$ m/s.

The results are the following determinations:

Crossing time: $t_ü = 2$ s

Clearance speed: $v_r = 5$ m/s
($v_r = 4$ m/s at R < 10 m)

Basic clearing distance: s_0 = distance [m] between stop-line of cars and conflict point, measured at the centre of the lane (see **Figure 2.9**)

Vehicle length: $l_{Fz} = 6$ m

Crossing and clearance time: $t_{ü} + t_r = 2 + \frac{s_0 + 6}{5}$

$$(t_{ü} + t_r = 2 + \frac{s_0 + 6}{4} \text{ at } R < 10\text{m})$$

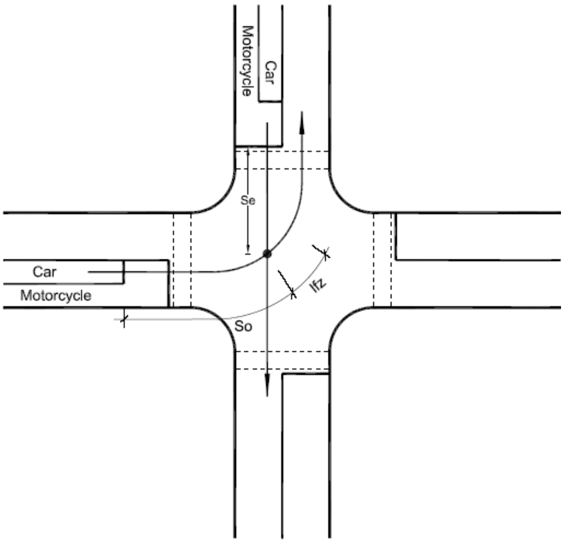


Figure 2.9: Example of the case "Turning vehicle is clearing / vehicle is entering"

Case 3: Public transport vehicles - stopping before the intersection

In case of the public transport vehicles stopping regularly in front of the intersection (for example at the bus-stop), in which at the end of the green time, the public transport vehicles start from the stop, then accelerate to the maximum operating speed permitted on the subsection.

For buses, the acceleration rate normally is a = 1.2 m/s^2. A variation from a = 1.0 to 1.5 m/s^2 may occur. The upper value is considered a passenger-dependent limit, since buses can achieve much higher acceleration rates.

Therefore, the clearing process is based on the following assumptions:

Crossing time: $t_{ü} = 0$

Start-up acceleration of buses:

a = 1.2 m/s^2 (with variation from 1.0 to 1.5 m/s^2)

Basic clearing distance: s_0 = distance [m] between stop-line of buses and conflict point, measured at the centre of the lane.

Vehicle length: l_{Fz} = 6 m for buses

Crossing and clearance time:

$$\text{for } (s_0 + l_{Fz}) \le \frac{(V_{max})^2}{2 \cdot 3.6^2 \cdot a}$$

$$t_{ü} + t_r = \sqrt{\frac{2\,(s_0 + l_{Fz})}{a}}$$

$$\text{for } (s_0 + l_{Fz}) > \frac{(V_{max})^2}{2 \cdot 3.6^2 \cdot a}$$

$$t_{ü} + t_r = \frac{V_{max}}{3.6 \cdot a} + \frac{s_0 + l_{Fz} - \frac{(V_{max})^2}{2 \cdot 3.6^2 \cdot a}}{V_{max}/3.6}$$

Case 4: Cyclists are clearing

The crossing time for cyclists is set at $t_{ü}$ = 1s, even if a transition signal is missing (joint signalisation with pedestrian traffic).

v_r = 4 m/s is the assumed clearance speed for cyclists. If cyclists have to follow very narrow bends before or after the crossing, it has to be reduced.

When calculating the intergreen time, the following determinations are relevant:

Crossing time: tü = 1 s

Clearance speed: v_r = 4 m/s (possibly lower)

Basic clearing distance: s_0 = distance [m] between stop-line of bicycles and conflict point, measured at the centre of the lane (see **Figure 2.10**).

Vehicle length: $l_{Fz} = 0$

Crossing and clearance time: $t_{ü} + t_r = 1 + \frac{s_0}{4}$

If the cyclists are signalised jointly with motorised traffic or pedestrian traffic, conflict situations like "cyclist is clearing"/"cyclist is entering" and "cyclist is clearing"/"pedestrian is entering" can generally be considered as permitted conflicting, i.e. they do not have to be taken into account in the intergreen time calculation. This also applies to tangential relations between cyclists and vehicles coming from the left.

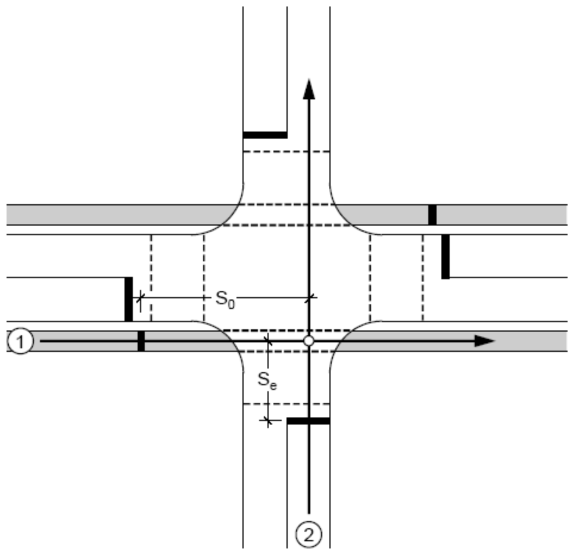


Figure 2.10: Example of the case "Cyclist is clearing/ vehicle is entering"

Case 5: Pedestrians are clearing

For pedestrians, $t_ü = 0$, as it is assumed that they do not step onto the road after their green time has ended.

The normal value of the clearance speed for pedestrians is $v_r = 1.2$ m/s. A variation from $v_r = 1.0$ m/s to $v_r = 1.5$ m/s can also be adapted.

Where crossings have been installed to protect people with impaired mobility, a lower value should be applied. At all other traffic signal systems, a reduction of the clearance speed is not required, because the blind and visually impaired people usually step on the crossing at the beginning of the green time and therefore the calculation of the intergreen time is not relevant.

The maximum value of $v_r = 1.5$ m/s should only be applied in exceptional cases.

Therefore, the following determinations are relevant for the calculation of the intergreen times:

Crossing time: $t_ü = 0$

Clearance speed: $v_r = 1.2$ m/s
(variation from 1.0 to 1.5 m/s)

Basic clearing distance: s_0 = distance [m] between stop-line and conflict point, measured at the centre of the lane (see **Figure 2.11**).

Crossing and clearance time: $t_ü + t_r = \dfrac{s_0}{v_r}$

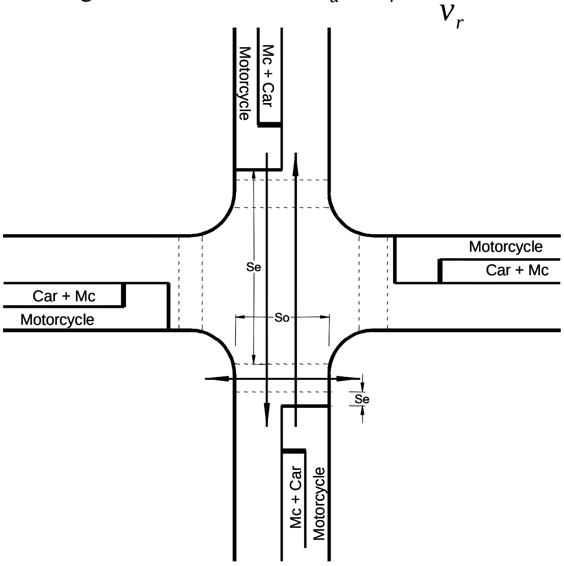


Figure 2.11: Example of the case "Pedestrian is clearing/vehicle is entering

2.5.3. Entering Times

The entering time t_e is the period of time needed to cover the entering distance s_e.

The first motorised vehicle is assumed to cross the stop-line at the beginning of the green time independently from permissible speed and direction at an entering speed of $V_e = 40$ km/h. The entering time then is calculated as follows:

$$t_e = \frac{3{,}6 \cdot s_e}{40}$$

An exceptional case for the entering time calculation is that when using a diagonal green for the lagging green time for left-turners. In this case, because the left-turners have already been in the intersection area before the lagging time starts, hence the entering distance $s_e = 0$ m.

If public transport vehicles regularly enter the intersection starting after a stop, standing acceleration is assumed at the stop-line when the green time begins. The entering time then amounts to:

$$t_e = \sqrt{\frac{2 \cdot s_e}{a}}$$

$$\text{for } t_e \le \frac{V_{max}}{3.6 \cdot a}$$

Jointly signalised with motorised vehicles, cyclists are not relevant for the entering process due to their low start-up acceleration and speed. If led on separate cycle paths or lanes and equally signalised separately, cyclists cross the stop-line at an assumed $v_e = 5$ m/s after the green time has begun.

If the conflict area between pedestrians and vehicles begins directly at the lane edge, the "entering process" does not have to be taken into account any more, so that $t_e = 0$. If the clearing traffic must not use the nearside lane, the assumed "entering speed" of pedestrians is $v_e = 1.5$ m/s.

2.5.4. Checking Intergreen Times

After having started a traffic signal system, the determined intergreen times have to be checked by repeated observations. A particular attention has to be paid to situations where left-turning vehicles are impeded by opposing traffic. The actual clearance and entering times of public transport muss be monitored carefully and compared with the arithmetical assumptions.

2.6. Cycle Time

The following methods determining the cycle time will be introduced, they can help select the value of cycle time if the cycle time is not given in cases of the networks or neighbouring intersections.

The necessary cycle time for a signal program generally results from the total relevant green times of individual phases and the total necessary intergreen times:

$$t_{U,erf} = \sum t_{F,maßg} + \sum t_{Z,erf}$$

t_U = cycle time [s]

t_F = green time [s]

t_Z = intergreen time [s]

Here, $\sum t_{F,maßg}$ includes green times of traffic flows that are released successively in the cycle time. The traffic flow receiving the green time $t_{F,maßg}$ can be formed from individual motorised traffic vehicles, from buses, from pedestrians, or from cyclists. In the calculation of the cycle time $t_{U,erf}$, while the last three road user groups mostly receive the green times not depending on the cycle time, motorised traffic can receive the green time either depending or not depending on the cycle time (e.g. the minimum green time $t_{F,min}$).

Of the necessary intergreen times between individual relevant traffic flows of individual phases, the maximum value is used.

If the green times depend on motorised traffic only, the minimum cycle time $t_{U,min}$ is calculated based on the model that: on the highest loading lane of a phase of the relevant signal groups, the number of motorised vehicles arriving $q_{FS,maßg}$ [veh/h] during the cycle time is equal to the number of motorised vehicles releasing q_S [veh/h] during the green time. This is simply presented by the following equation:

$$\frac{q_{FS,maßg}}{3600} \cdot t_{U,\min} = \frac{q_S}{3600} \cdot t_F$$

q_S = saturation flow rate

If during the green time, motorcycles are released priorly and cars are released successively, then:

$$\begin{cases} \dfrac{q^{mc}_{FS,maßg}}{3600} . t_{U,min} = \dfrac{q^{mc}_S}{3600} . \sum t^{mc}_{Fi} \\ \dfrac{q^{car}_{FS,maßg}}{3600} . t_{U,min} = \dfrac{q^{car}_S}{3600} . \sum t^{car}_{Fi} \end{cases}$$

If p is the number of phases, then the minimum cycle time is:

$$t_{U,min} = \frac{\sum_{i=1}^{p} t_{Z,erf,i}}{1 - \sum_{i=1}^{p} \frac{1}{f} . \left(\frac{q^{mc}_{FS,maßg,i}}{q^{mc}_{S,i}} + \frac{q^{car}_{FS,maßg,i}}{q^{car}_{S,i}} \right)}$$

Where f = adjustment factor for the saturation flow due to a part of the green time is used for mixed traffic.

f = 1 if there is no mixed traffic (MCs ahead, cars follow).

f = 0.9 ÷ 0.95 if a part of green time is used for mixed traffic.

f = 0.8 ÷ 0.9 in case of using mixed traffic on lanes.

$q^{mc}_{FS,maßg}$; $q^{car}_{FS,maßg}$ = traffic volume of MCs and cars.

q^{mc}_S ; q^{car}_S = saturation flow of MCs and cars, respectively (values of these saturation flows are given in **Annex 3** of these guidelines).

Especially, in fixed-time signal control, in order to take a random variation of the traffic flow into account, the calculated maximum saturation flow q_S has to be reduced to a permitted saturation flow $q_{S,zul}$ by the saturation degree g:

$$q_{S,zul} = g.q_S$$

For the saturation degree g, the value between 0.8 and 0.9 can be used. If $\sum t_{F,maßg}$ contains only the green times of motorised traffic depending on the cycle time, the necessary cycle time is given as follows:

$$t_{U,erf} = \frac{\sum_{i=1}^{p} t_{Z,erf,i}}{1 - \sum_{i=1}^{p} \frac{1}{f} . \left(\frac{q^{mc}_{FS,maßg,i}}{g_i . q^{mc}_{S,i}} + \frac{q^{car}_{FS,maßg,i}}{g_i . q^{car}_{S,i}} \right)}$$

If all g_i =1 is used in this formula, it becomes the given relationship of the minimum cycle time $t_{U,min}$.

In addition, if there are phases in which their green times do not depend on the cycle time (e.g. minimum green times $t_{F,min}$), the corresponding formula for the cycle time has to be worked out. It means that besides the total intergreen times $\sum t_{Z,erf}$, all relevant green times not depending on the cycle time are added together as shown in the numerator, while the green times depending on the cycle time are shown in the denominator:

$$t_{U,erf} = \frac{\sum_{i=1}^{p} t_{Z,erf,i} + \sum_{k=1}^{p_2} t_{F,min,k}}{1 - \sum_{j=1}^{p_1} \frac{1}{f} . \left(\frac{q^{mc}_{FS,maßg,j}}{g_j . q^{mc}_{S,j}} + \frac{q^{car}_{FS,maßg,j}}{g_i . q^{car}_{S,j}} \right)}$$

p = total number of phases

p_1 = number of phases depending on traffic volume

p_2 = number of phases not depending on traffic volume

And, $p = p_1 + p_2$

Another possible method for the calculation of the cycle time is minimising delay of motorised vehicles. The method bases on the assumption that vehicles arrive randomly (Poisson distribution), therefore, an adjustment factor for the random variation of arriving flows must not be taken into account. The optimal delay cycle time is determined as follows:

$$t_{U,opt} = \frac{1.5.\sum_{i=1}^{p} t_{Z,erf,i} + 5}{1 - \sum_{i=1}^{p} \frac{1}{f} . \left(\frac{q^{mc}_{FS,maßg,i}}{q^{mc}_{S,i}} + \frac{q^{car}_{FS,maßg,i}}{q^{car}_{S,i}} \right)}$$

Boundary conditions of pedestrians, cyclists or public transport vehicles for the cycle time have to be taken into account when co-ordinating signal programs of some intersections or arterials.

The following values of the cycle time are recommended:

Minimum 30 s

Maximum 120 (150) s

The cycle times of more than 120 s should be avoided if it is possible; the maximum cycle time is 150 s. If the cycle times of more than 120 s are required, to guarantee traffic flow quality, the operating times for these signal programs have to be delimited.

2.7. Green Times and Red Times

2.7.1. Calculation of Green Times

If motorcycles are released priorly and cars are released successively, the necessary green time $t_{F,erf}$ of motorised vehicles depending on the cycle time is determined according to the formula:

$$t_{F,erf} = \frac{1}{f} \cdot \left(\frac{q^{mc}_{FS,\text{maßg}}}{q^{mc}_{S,zul}} + \frac{q^{car}_{FS,\text{maßg}}}{q^{car}_{S,zul}} \right) . t_U$$

Where: - the necessary green time for motorcycles

$$t^{mc}_{F,erf} = \frac{1}{f} . \frac{q^{mc}_{FS,\text{maßg}}}{q^{mc}_{S,zul}} . t_U$$

- the necessary green time for cars

$$t^{car}_{F,erf} = \frac{1}{f} . \frac{q^{car}_{FS,\text{maßg}}}{q^{car}_{S,zul}} . t_U$$

In case an amount of time is still reserved within the cycle time, it can be divided for different aims, for example, an optimization of a co-ordination, or significant considerations of pedestrians.

If the reservation time is divided for all phases, the following formula is given:

$$t_{F,i} = \frac{t_U - \sum_{i=1}^{p} t_{Z,erf,i}}{\sum_{i=1}^{p} \frac{1}{f} \cdot \left(\frac{q^{car}_{FS,\text{maßg},i}}{q^{car}_{S,i}} + \frac{q^{mc}_{FS,\text{maßg},i}}{q^{mc}_{S,i}} \right)} \cdot \frac{1}{f_i} \cdot \left(\frac{q^{car}_{FS,\text{maßg},i}}{q^{car}_{S,i}} + \frac{q^{mc}_{FS,\text{maßg},i}}{q^{mc}_{S,i}} \right)$$

2.7.2. Return to the Same Phase

In case of traffic-dependent control strategies, successive requests arise and the control algorithm has already terminated the green time of the first request, the complete signal sequence has to be followed before GREEN appears again, even if returning to the same phase.

2.7.3. Maximum and Minimum Red Times

The definition of the maximum red time for a road user group or a traffic flow very much depends on the control strategy and the balancing of conflicting objectives. The influential factors are, for example:

- acceptance by pedestrians, cyclists, and motorcycles,
- available queuing space for motorised traffic,
- available waiting areas for pedestrians and cyclists or
- total travel time for public transport on a subsection.

Furthermore, it has to be considered that the selected red times enable the aspired traffic flow quality to correspond with the evaluation according to **Annex 2**.

The minimum red time is 1 s.

2.7.4. Minimum Green Times

The minimum green time must not be shorter than 10 s.

Additionally it has to be ensured that at a single crossing, pedestrians should be able to cover at least half of the crossing during the green time. This value increases to the entire crossing in case of acoustic devices for visually impaired people.

In case of pedestrians having to cross two successive crossings in the same phase, the green time should be so long that pedestrians can cross the longer one of two crossings, the central reservation or separating strips and a half of the second crossing. In case of a higher number of successive crossings, an overall design of signal programs that eases pedestrians has to be taken into account. In the most cases, however, a co-ordination based on manifold boundary conditions cannot be realised.

2.7.5. Time Lead at the Conflict Area

If a permitted turning traffic flow is released together with parallel priority pedestrians or cyclists, the green time beginnings have to be offset, allowing pedestrians or cyclists to step onto the crossing 1 s or 2 s before a turning vehicle arrives.

In analogy hereto, a time lead for public transport vehicles led in central or lateral positions should be given in cases these vehicles are not led by a separate phase, in order to emphasise their priority against turning vehicles.

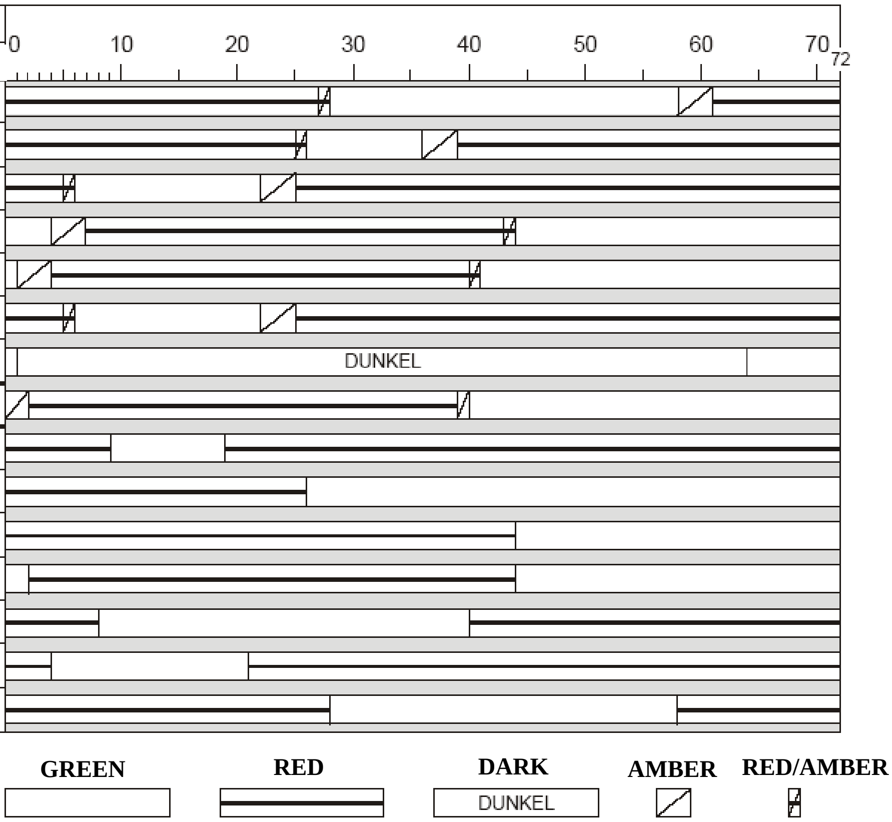
2.7.6. Delayed Green Time Beginning

If in permitted movements, left-turning vehicles have crossed the stop-line and are impeded by opposing traffic, they must be given the opportunity to clear the conflict area safely during the phase transition. If they may not be made out in time, the following conflicting traffic stream should be released about 2 s to 4 s later than determined by the intergreen time calculation. An analogy is given when returning to the same phase.

2.8. Signal Timing Plan

The results of the traffic engineering calculations for a fixed-time signal program are depicted by the signal timing planning (see **Figure 2.12**), the intergreen time matrix (see **Figure 2.7**) and the intersection layout. Traffic-actuated signal control requires additional descriptions of the control algorithm.

signal group	t_{Gr} [s] begin	end	period
K1*)	28	58	30
K2	26	36	10
K3	6	22	16
K4	44	4	32
K5	41	1	32
K6	6	22	16
K7	64	1	9
R1	40	72	32
F1	9	19	10
F2	26	72	46
F3	44	72	28
F4	44	2	30
F5	8	40	32
F6	4	21	17
F7	28	58	30



*) Signal group K1 includes signals K1a and K1b; the same applies to further signal groups.
If RED/AMBER is not used, it can be ignored.

Figure 2.12: Example of a signal timing plan with a fixed-time signal program

3. Inter-relations between Traffic Signal Control and Road Design

3.1. General Remarks

Traffic flow to be achieved by means of traffic signal control makes particular demands on the road design. In the same way, the intersection and subsection layout as well as environmental considerations substantially impact on the traffic signal control. Therefore, the road design and the traffic signal control have to be considered as an entity, developed step by step in mutual coordination. The requirements of different road user groups have to be balanced, utilising the various possibilities of the traffic signal control optimally, in particular as road space is not freely available.

When constructing new signalised intersections, standard forms of intersection layouts are selected in order to minimise impacts on the environment. They may be oriented mainly at traffic signal requirements. Impacting pedestrians and cyclists as well as intergreen times, the inner intersection area generally should be kept as small as possible.

When re-constructing signalised intersections by installing additional traffic signal systems, and by minimising impacts on the environment, local characteristics are often more important than traffic signal requirements, therefore standardised intersection layouts can only be achieved roughly and flexible design of elements can be applied.

3.2. Lanes

The number and the division of lanes as well as head-start areas for motorcycles at an intersection depends on the traffic volume and the desired traffic flow quality of all road user groups, on requirements of the traffic safety as well as of the space available.

Lanes can also be used dynamically. This usually requires a compulsive measure of signal technics and traffic-dependent control strategies to organize the successive times for multi-shift usages.

3.2.1. Head-start Lanes for Motorcycles

Head-start lanes for motorcycles are designed to give motorcycle riders an opportunity to get in front of cars on approach during the red time. Hereby, when the signal turns green, head-start motorcycles are priorly released, and cars follow.

In order to create the head-start lanes for motorcycles, a part of road space must be reserved for this purpose. The number of lanes for cars, therefore, may be reduced. Consequently, it is recommended to use shared lanes for go-through and right-turning cars if road space is limited.

However, mixed traffic must be used in case of one-lane approach because there is no more road space available for head-start lanes of motorcycles.

Depending on the proportion of motorcycles in the traffic flows, there are three possibilities to guide motorcycle riders to get in the head-start lanes:

- If the proportion of motorcycles is high (recommended more than 50%), motorcycles and cars should share lanes along roads, but on approaches, motorcycles are guided to get in the head-start lanes during red time (see **Figures 3.1** and **Figure 3.2**):

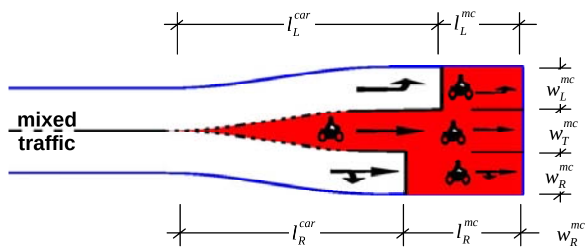


Figure 3.1: Head-start area with low traffic volume of left-turning motorcycles

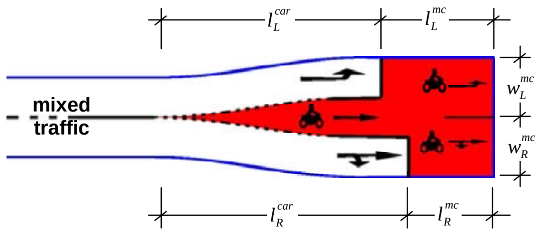


Figure 3.2: Head-start area with high traffic volume of left-turning motorcycles

- If the proportion of motorcycles is medium (recommended from 30% to 50%), it is reasonable to separate motorcycles from car traffic along the road. Hereby, motorcycles use their own lane (see **Figures 3.3, 3.4, and 3.5**):

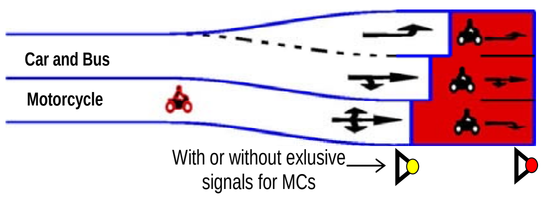


Figure 3.3: Head-start area with an exclusive lane for motorcycles on approach

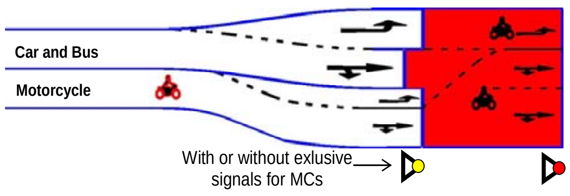


Figure 3.4: Head-start area with an exclusive left-turning lane for motorcycles on approach

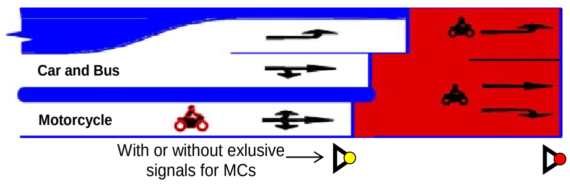


Figure 3.5: Head-start area with an exclusive lane for motorcycles by constructional measure

In case of not using exclusive signals for motorcycles, it has the disadvantage that motorcycle riders arriving during green have to pay attention on car traffic streams to join the green phase. This is acceptable, however, because motorcycle traffic volume is relatively low.

In case of using exclusive signals for motorcycles, the signal sequence RED-DARK-RED is used and coordinated with the major signals on the same approach (see **Figure 3.6**). In this case, traffic safety is increased. However, waiting time for motorcycle riders will be longer, and the capacity of motorcycles is reduced.

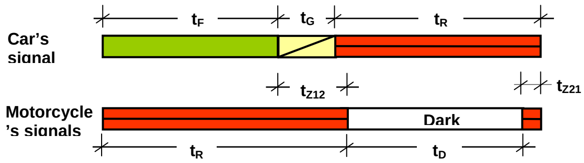


Figure 3.6: Signal coordination between exclusive signals for MCs and major signals on the same approach

- If the motorcycle proportion is low (recommended less than 30%), it is not necessary to separate motorcycles from car traffic because the impacts of motorcycles on overall traffic flow is not so critical.

Dimensions of the head-start lanes are determined by the following steps:

Step 1: Determining the red time of the signal group:

$$t_{Ri} = t_U - t_{Fi} - t_{Gi}$$

Step 2: Determining the number of motorcycles arriving during the red time:

$$q_{Ri}^{mc} = \frac{q^{mc}}{3600}.t_{Ri}$$

Step 3: Determining the length of the head-start lanes:

$$l_i^{mc} = \frac{q_{Ri}^{mc}}{w_i^{mc}}.2$$

Step 4: Determining the length of the car's lane backward to the head-start lanes.

This length is determined according to HBS-2001, see **Annex 3**.

Where: t_{Ri} = red time, t_U = cycle time, t_{Fi} = green time, t_{Gi} = amber time [s]

q_{Ri}^{mc} = number of motorcycles arriving during the red time

q^{mc} = motorcycle traffic volume [veh/h]

l_i^{mc} = length of the head-start lane [m]

w_i^{mc} = number of motorcycles that can stop in a row on the head-start lane (chosen as multiple of 1 m because one motorcycle is fit in 1 m wide of road space)

2 = length of a motorcycle [m].

3.2.2. Continuous Lanes

Generally the number of continuous lanes at the intersection is to be kept on downstream subsections. In built-up areas it may become necessary to increase the number of continuous lanes on the approaches in order to better match the capacity of the intersection with that of the subsections.

The minimum length l of the lanes to be continued unchanged in number on the intersection exit can roughly be determined as follows:

$$l\,[m] = 3 \bullet t_F\,[s]$$

Therefore, as the green time t_F, the necessary green time during peak-hours has to be determined. Pre-condition for a reduction in number of lanes is a sufficient capacity on the downstream subsections.

The distortion l_{zl} of the lanes has to be symmetrical and relatively long (the minimum l_{zl} = 30 m), providing best for smooth traffic flow by the merging (see **Figure 3.7**).

If a continuous lane must be converted into a turning lane, early and unequivocal marking and signing is required. Otherwise, an unexpected and a late lane changing may occur.

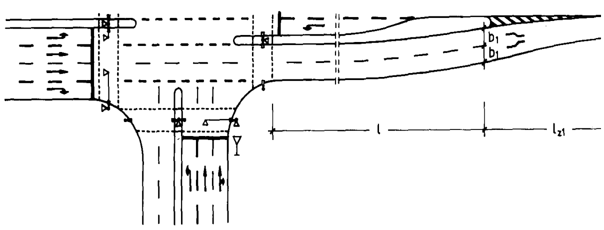


Figure 3.7: Reduction of the number of lanes on intersection exits

3.2.3. Left-turning Lanes

If a left-turning traffic stream is signalised separately, separate left-turning lanes have to be available.

The length of left-turning lanes should be chosen so that the adjacent lanes are not congested. This is validly given if the length of left-turning lanes exceeds the critical 95%-queue length on the left-turning lanes or on the neighbouring lanes. In critical cases, monitoring a queuing space is recommended. The queue length according to HBS is presented in **Annex 3** of these guidelines.

If there is no exclusive phase for left-turning movements, left-turning lanes or queuing space should only be dropped, if left-turners can leave the intersection unimpaired, or if all left-turners of one cycle time can queue up in the inner intersection area.

If neither a left-turning lane nor queuing space can be established on an intersection approach, left-turning, however, cannot be prohibited, traffic on the entire approach should be phased separately.

Under confined circumstances it may be better to establish left-turning lanes or queuing space of a shorter length than to drop them.

Left-turning lanes must not be separated from continuous lanes by delineation if less than 5.50 m of an approach width are available. In case of an approach width between 4.25 m and 5.50 m, however, it is better to establish one lane for cars and the remaining road space for motorcycles (see **Figure 3.8**).

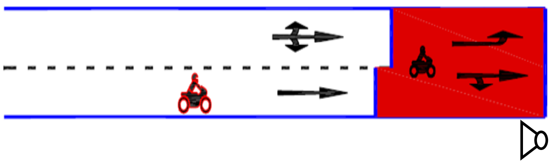


Figure 3.8: An exclusive lane for MCs on a narrow approach

If left-turning has to be prohibited, it is recommended to indicate alternative routes early.

3.2.4. Right-turning Lanes and Right-turning Carriageways

If right-turning movements are phased separately, right-turning lanes have to be available. On these lanes, it is not necessary to separate motorcycles from car traffic (see **Figure 3.9**). Their length is determined analogously to the length of left-turning lanes corresponding to a 95%-queue length that is determined in **Annex 3** of these guidelines. In this case, to determine the queue length of mixed traffic, motorcycles have to be converted into passenger car units by equivalent factors shown in **Annex 3**.

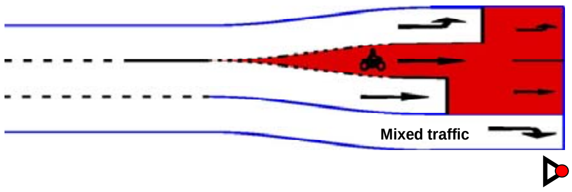


Figure 3.9: Exclusive right-turning lane

In built-up areas, right-turning carriageways next to a triangular island at signalised intersections can be used for non-signalised right-turners to increase capacity if pedestrians and cyclists concerned are not impaired. In contrary cases, the crossing at the right-turning carriageway is signalised, a total crossing time of pedestrians may become long, and therefore a danger of disregarding the signals may arise. In case of small triangular islands, locations of signal heads for pedestrians have to be arranged unequivocally.

In case of a high proportion of motorcycles, the following layout approach may be considered (see **Figure 3.10**):

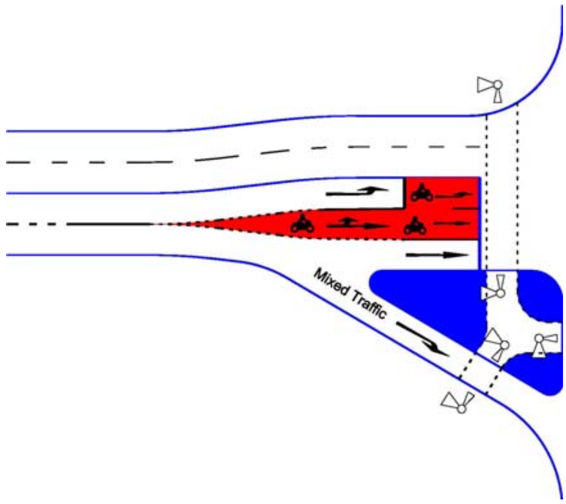


Figure 3.10: Right-turning carriageways with triangular islands (1)

In case of a low proportion of motorcycles, the following layout approach may be considered (**see Figure 3.11**):

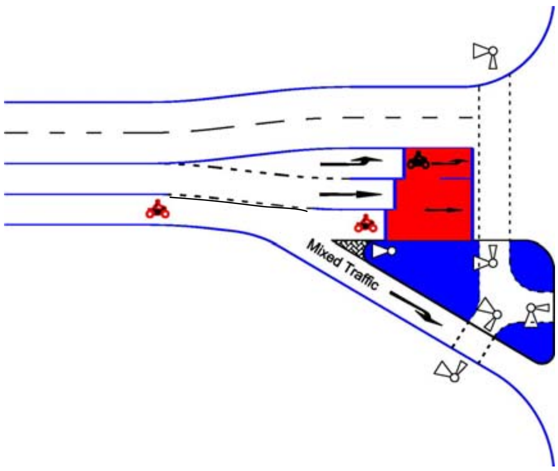


Figure 3.11: Right-turning carriageways with triangular island (2)

At non-signalised right-turning carriageways the priority rules to non-backward cyclist lanes and to motorcycle lanes at the beginning and end of the right-turning carriageway as well as to a pedestrian crossing in the middle of the right-turning carriageway have to be indicated clearly. In exceptional cases pelican crossings do not have to be established, but then no pedestrian crossing is to be marked either.

If there is a two-direction cycle path or a small triangular island alternatively in the middle of the right-turning carriageway, a priority traffic sign for the cycle crossing next to the pedestrian crossing has to be considered.

3.2.5. Separate Lanes for Public Transport

If there is no separate lane for buses, buses have to share lanes and join signalisation with car traffic.

If road space is sufficient to establish an exclusive lane for buses, especially on the arterials or ring roads, the following layouts should be considered.

In the first case, buses share a lane with motorcycles along the road due to a high proportion of motorcycles, but at intersections, this lane is reserved only for buses by marking as illustrated in **Figure 3.12**.

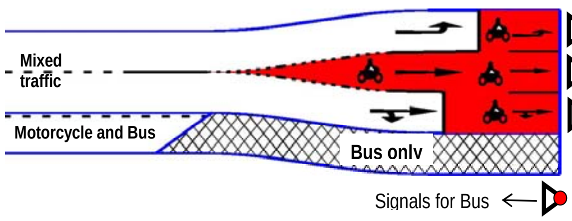


Figure 3.12: Partial lane for buses

In the second case of lower proportion of motorcycles, a separate lane for buses is established along the road and also at intersections as illustrated in **Figure 3.13**.

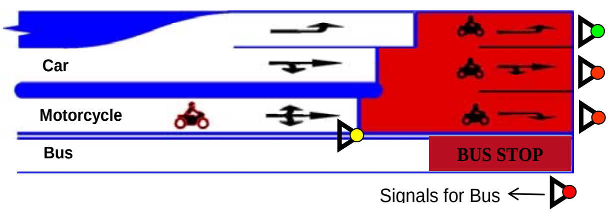


Figure 3.13: Exclusive lane for buses

In both cases, it is necessary to use exclusive signals for buses.

3.2.6. U-turn Lanes

On major roads with central reservations, or central bus lane, u-turn lanes on a road section should be given, because complete u-turns at intersections lead to a great loss of safety, capacity and traffic flow quality. Besides, u-turning lanes can provide alternative routes for left-turners having missed turning, or possibilities for crossing vehicles.

If u-turn lanes cannot be established, the signal program has to be selected so that u-turning vehicles and other traffic movements (also pedestrians and cyclists) do not mutually impede.

U-turn lanes are situated properly if gaps in opposing traffic streams due to traffic signal control at neighbouring intersections can be used for u-turn manoeuvres.

Traffic signal control for u-turning traffic is necessary, if

- opposing traffic for free u-turns, for example turning vehicles, is too heavy,
- bus lanes on the central reservation must be crossed.
- on the queue lane before the u-turn or on the discontinuous central reservation, there is not enough queuing space available for u-turning traffic or
- the view onto opposing traffic is not as unobstructed as should be.

3.3. Guidance for Cycle Traffic

There are some possibilities to guide cycle traffic on the carriageway or on the roadside. They have significant influences on the signalisation of cycle traffic and on the guidance for turning cyclists at signalised intersection.

The direct guidance has to be considered if the cyclists ride on downstream subsections on the carriageway. Then, they can be signalised jointly with motorised traffic and also can turn left directly.

If cycle traffic is led on downstream subsections by cycle paths or cycle lanes, cyclists can join in head-start lanes with motorcycles to have an opportunity to turn left directly.

The indirect guidance for left-turning cyclists (see **Figure 3.14**) has to be considered if cycle paths or cycle lanes are available on the downstream subsections.

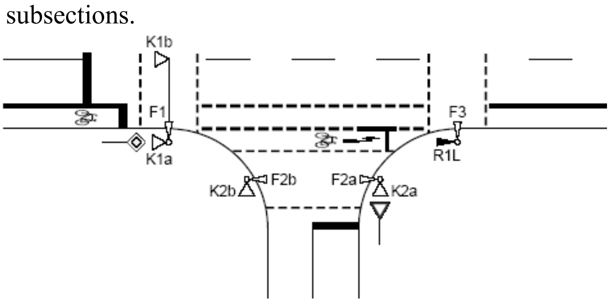


Figure 3.14: An example of indirect left-turning cyclists

In many cases, a separate queuing space for left-turning cyclists has to be marked on the left-hand or right-hand side of the cycle crossing. The separate routing of cyclists should be signed.

Cyclists having crossed the adjacent road and waiting on the queuing space to turn left should be signalised separately or jointly with the pedestrian signals of the desired direction.

However, joint signalisation requires that:

- the pedestrian crossing is not placed back far,
- green time at the pedestrian crossing must ensure a sufficient time lead to the permitted turning vehicles in the same phase,
- both pedestrian crossings are released simultaneously if there is a central reservation.

3.4. Central Reservations and Separating Strips

Central reservations and separating strips generally serve traffic flow route, pedestrian and cyclist protection, location of traffic installations (occupancy, signs, traffic signals and direction signs) as well as grass planting.

Central reservations and separating strips at signalised approaches are priorly designed as crossing aids if the traffic safety of pedestrians and cyclists at the deactivated traffic signal systems is required or if the traffic safety is determined by the signal program, especially in case of protective guidance for pedestrians.

Depending on their width, central reservations and separating strips have to be assessed in different ways with regard to pedestrian signalisation:

- For separating strips or central reservations more than 4 m wide, a separate signalisation of both crossings is generally not considered to be critical, because pedestrians perceive the successive crossings independently from each other.
- For separating strips or central reservation less than 4 m wide, at separately signalised crossings, there is the risk that pedestrians may overlook the signals, cannot assign them correctly or violate them. To make sure that pedestrians do not misconceive the signals addressing them, the signals of successive crossings have to be arranged in an aligned row.
- The width of separating strips or central reservations should not be less than 2.50 m wide in order to provide sufficient queuing space for cycle traffic.

3.5. Crossings

Pedestrian and cycle crossings should be established as near the edge of the parallel road as possible due to having a better view on the intersection.

If go-through vehicles and right-turning vehicles share a lane, the parallel pedestrian and cycle crossings can be placed backward 5 m to 6 m from the edge of carriageway in order to create a queuing space for a passenger car. The priority of cyclists and pedestrians to the turning-vehicles at the conflicting area must be recognized unequivocally. On roads with more important cycle traffic connections, the cycle crossing should not be placed backward if strong deviations of cycle traffic systems are required. A cycle path can also be changed to a cycle lane on an approach or in the inner intersection area.

The normal width of a pedestrian crossing is 4.00 m, the minimum width is 3.00 m. The cycle crossing should be as wide as the cycle traffic layout. At the pedestrian width of more than 8.00 m, the second signal head should be installed on each direction.

Near pedestrian crossings, sufficient waiting areas have to be provided at the roadside or on traffic islands, so that pedestrians arriving on red can be taken in (line-up density approx. 2 persons/m2, 1 cyclist/1.5m2).

3.6. Stops

Some issues concerning operation, location and diversity of passengers have to be balanced by arrangements and forms of the stop.

The location of stops in the intersection area has to be determined in close connection with the signal control. Locating stops before the intersection bears the advantage that lost times due to signalisation can be used for passenger boarding and alighting. However, it may also have a disadvantage that while passengers are alighting and boarding, crossing traffic is usually not released especially passengers in a hurry.

If straight-forward and right-turning buses have to be taken into account only, their stops can be situated immediately before the inner intersection area or on the non-heavy right-turning lanes. Hereby, the bus drivers can be indicated leading green by a bar signal or a permissive signal, allowing them to enter the inner intersection area before private traffic after passengers have alighted and boarded.

Locating stops after the intersection has the advantage that the green time at the downstream traffic signals can be requested early and switched reliably for public transport, because there is sufficient preparation time and the varying duration of the stops must not be taken into account.

Another advantage is that after the cancellation of public transport vehicles, the crossing traffic can be released immediately.

3.7. Facilities

3.7.1. Stop-lines

The stop-line for motorists should be marked at a distance of 3.00 m, at least 2.50 m, from the signal head, whereby the minimum distance to be kept from the edge of pedestrian crossing is 1.0 m (see **Figure 3.15 and 3.16**).

If not joining the head-start lanes with motorcycles, the stop-line of cycle lane is marked equally with that of motorcycles. In addition, if there is an exclusive bus lane on the roadside, the stop-line of the parallel cycle lane should be marked approx 1.00 m ahead of the bus stop-line, hereby, the bus driver can have a good view on the waiting cyclists.

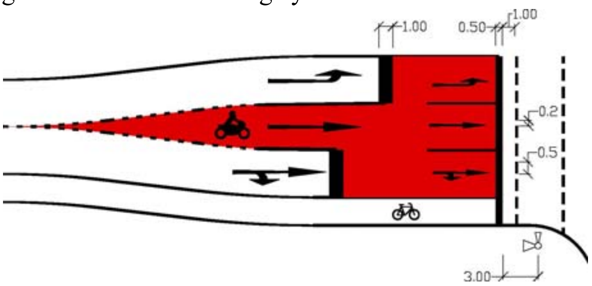


Figure 3.15: Stop-lines (1)

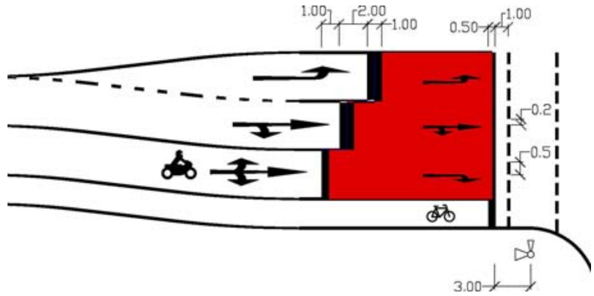


Figure 3.16: Stop-lines (2)

If the right-turning vehicles and the adjacent opposing traffic have to share an area on the approach or in the inner intersection, especially in case of acute-angled T-junctions, a displacement of the stop-line is required.

On multi-lane approaches of opposing traffic, staggered stop-lines may be recommended (see **Figure 3.17**).

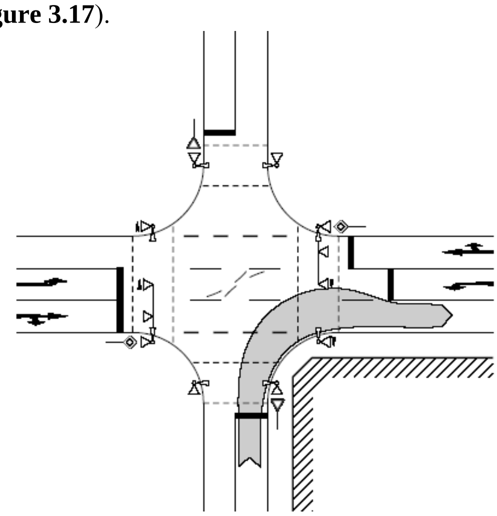


Figure 3.17: Marking and signing of an intersection with staggered stop-lines

3.7.2. Directional Line and Head-start Area Marking

The clarity and the unequivocal guidance for individual traffic flows at signalised intersections can be ensured by help of directional line marking.

In order to lead left-turners in the inner intersection area, either solid lines or 1.00 m long broken lines can be used when intersecting with the markings of crossing lanes.

In case of scarcely dimensioned queuing space in inner intersection areas, it is generally sufficient to lead left-turners past each other by outer delineation. It is important to mark the point up to which left-turners can proceed without invading the parallel straight-ahead lane of the opposing direction. The left-hand delineation to the opposing straight-ahead lane is better than the generally possible marking of a stop-line.

The head-start area for motorcycles is marked by dark-red colour. Hereby, the car drivers can recognize early enough and do not invade motorcycles' area when stopping. Motorcycle riders can also recognize easily their waiting area.

3.7.3. Signing

When signing signalised intersections, the basic principles listed below have to be followed:

- Signalised intersections are always equipped with priority and non-priority (give-way) signs to regulate traffic flows when the traffic signal system has been switched off or fails.
- If the traffic streams of an intersection approach are not allowed to proceed in all directions, the permitted directions have to be indicated by corresponding signs. Mandatory right- or left-turning sign should only be used together with directional signals.
- Outside built-up areas, traffic signal systems can be indicated in advance by a traffic signal sign. Within built-up areas this sign is to be set up only if motorists may not be able to see the signs early enough or temporarily, or if a new traffic signal system is being set up.
- If the road is right-bending and the intersection and the signal head arranged on the right-hand side cannot be seen from where the traffic signal sign has been arranged, it is recommended to complement this sign by an amber flashing light.
- If along roads, speed of more than 70 km/h is permitted, speed has to be limited at sufficient distance to 70 km/h before traffic signal systems by a speed limit sign. In case of constructional limitations (e.g. missing left-turning lane or particularly narrow carriageway) or for safety reasons it is reasonable to set the permissible speed at 60 km/h. On high-speed roads speed limits have to be graded.

4. Control Strategies

4.1. Overview on the Control Strategies

A control strategy describes a process of a signal program, e.g. the type, the size and the interactions of variable control parameters and signal program elements.

The control strategies for traffic signal systems differ in types as well as in the control of realised traffic streams and in the influence able or changeable degree of the signal program elements. The defined objectives determine which control strategy has to be chosen.

Table 4.1 shows the control strategies and their possible combinations according to the criterion "traffic-dependent variable signal program elements".

Generally, two control levels, macroscopic level and microscopic level, are distinguished.

The macroscopic control strategies respond to macroscopic parameters (for example, the average queue length, the average traffic density, the boundary values of emission) and mainly serve the consideration of long-term changes of traffic loads in the network, in parts of the network or at isolated intersections. The signal programs selected from a given set of signal programs depending on time (A1 in **Table 4.1**) or traffic (A2) or alternative traffic-dependent signal program formations (A3) are switched for longer periods of time. Time-dependent and traffic-dependent selection criteria can also be combined with each other.

However, in MDCs, only time-dependent signal program selection (A1) is able to be implemented because it requires much effort or is impossible to collect traffic data online in MDCs by conventional technologies.

Table 4.1: Overview on Control Strategies

<table>
<tr><th rowspan="2"></th><th colspan="2">control strategy</th><th rowspan="2">number</th><th colspan="2">activation</th><th colspan="5">traffic-dependent variable elements of the signal programs</th></tr>
<tr><th>general term</th><th>main feature of signal program modification</th><th>time-dependent</th><th>traffic-dependent</th><th>cycle time</th><th>phase sequence</th><th>number of phases</th><th>green time</th><th>time offset</th></tr>
<tr><td rowspan="3">A:macroscopic control level</td><td rowspan="2">signal program selection</td><td>time-dependent signal program selection</td><td>A1</td><td>X</td><td></td><td colspan="5" rowspan="3">in combination with the variable elements of the signal programs from a control strategy of the group B</td></tr>
<tr><td>traffic-dependent signal program selection</td><td>A2</td><td></td><td>X</td></tr>
<tr><td>signal program formation</td><td>forming signal programs by traffic-dependent selection</td><td>A3</td><td></td><td>X</td></tr>
<tr><td rowspan="6">B: microscopic control level</td><td>fixed-time signal program</td><td></td><td>B1</td><td colspan="2" rowspan="6">activation according to control strategies of group A</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="4">signal program adaptation</td><td>green time adjustment</td><td>B2</td><td></td><td></td><td></td><td>X</td><td></td></tr>
<tr><td>phase swapping</td><td>B3</td><td></td><td>X</td><td></td><td></td><td></td></tr>
<tr><td>demand phase</td><td>B4</td><td></td><td></td><td>X</td><td>X</td><td></td></tr>
<tr><td>time-offset adjustment</td><td>B5</td><td></td><td></td><td></td><td></td><td>X</td></tr>
<tr><td>signal program formation</td><td>free modification possible</td><td>B6</td><td>X</td><td>X</td><td>X</td><td>X</td><td>X</td></tr>
</table>

Remark: control strategies usually not applicable in MDCs

In general, the strategies of the microscopic control level are activated from the macroscopic control level. Unless fixed-time signal programs are used, changes in the respective traffic situation at the intersection are taken into account without delay.

The microscopic control strategies consider short-term periods, for example, changes of traffic situations occurring within seconds or within a cycle time. They can be broken down into three different sub-categories, depending on which individual elements of the signal program is variable:

- fixed-time signal programs (B1),
- strategies of signal program adaptation (B2, B3, B4 and B5)
- strategies of signal program formation (B6)

All three sub-categories presuppose off-line calculated signal programs or at least part of signal program.

Fixed-time signal programs do not provide a modification of the signal program elements.

In control strategies, signal program adaptation can modify individual elements within a cycle time of a signal program depending on traffic situations.

By the green time adjustment strategy, the green time regarding duration or position in the signal program can be adjusted to the corresponding traffic situations. Hereby, if the fixed cycle time is given and the offset is determined by the relative beginning of the green times within the cycle time, only periods of green time adjustment are allowed to vary (B2).

Phase swapping means that the phase sequence is modified, while all other elements remain unaltered (B3).

If requested, a demand phase can be included into the given phase sequence at one or several positions of the signal program by a temporary cutting of green periods assigned to other phases (B4).

With the time offset adjustment strategy, the relative beginning of the green times within the cycle time can be varied (B5).

The strategies B2 to B5 are often combined with each other.

With the signal program formation, the variable elements of a signal program can be formed in a traffic-actuated way (B6).

At different times of day, different macroscopic and microscopic control strategies can be used.

With the application of the signal group oriented control, the term "phase" in **Table 4.1** can be replaced by the term "signal group", because in the difference between signal group oriented control and phase oriented control, the individual signal group is the element to be referred.

The implementation of different control strategies can result from either rule-based or model-based (see **section 4.5**).

4.2. Control Parameters

4.2.1. Combination of Parameters

In order to be able to implement the control of a traffic signal system according to the defined objectives, directly or indirectly measurable target values have to be defined. The directly measureable traffic parameters that belong to the period of time are: green time request, time headway as well as degree of occupancy. The indirect parameters, for example, average waiting time, and queue length can be calculated by a certain model. Additionally, if possible, parameters evaluating impacts on the environment are included, these may be partially derived from the above parameters or must be collected separately.

Control strategies base on individual parameters or on a combination of different parameters.

4.2.2. Collecting and Processing Parameters

4.2.2.1. Green Time Request

The presence of motorcycles in the head-start lanes is detected by the detectors lying in front of the stop-line (usually 1 m to 1.5 m from the beginning of the detectors) so that the first request vehicle can stand on them, the shape of required detectors can be easily recognized by marking. If necessary, a traffic sign "please advance to the stop-line" can be applied. The vehicles can also be detected by infrared or video detectors.

Cyclists are detected by small inductive loops with directional recognition by angular arrangements. In case of modern bicycles with a low proportion of metal, cyclists can be detected by push-button or other detectors.

Pedestrians and jointly signalised cyclists usually request their green times by a push-button or a touch-button.

In order to give priority to public transport vehicles or emergency vehicles at signalised intersections, these types of vehicles must be separately and individually detected by a request or a cancellation. Depending on the local situations, it can also be necessary to collect vehicle types as well as route and short information.

Normally, in the traffic-dependent control strategies, the following requests have to be carried out: pre-requests to prepare for green times and main-requests to switch accurately green times, cancellations of requests are carried out after the vehicles have passed the stop-line.

The stops have to be significantly taken into account by pre- and main-requests or supplemental requests. Additionally, the requests from actions of the drivers (for example the readiness for departure) or from the vehicle side (door-closing contact) can also be taken into account.

4.2.2.2. Time Headways

When measuring time headway, a detector on the intersection approach measures the net intervals between succeeding vehicles of one traffic stream. The green time is extended until the measured time headway is at least as long as a given time headway value ZL or until the longest fixed green period or the latest point of extension has been reached in the cycle. Those time headways, only which after the expiry of the minimum green time or after the earliest point of green time termination (T_1) are at least as long as ZL are used for control (see **Figure 4.1**). The time headways leading to green time abortion may have already begun during the minimum green time or before T_1.

Values between 2 s and 5 s can be set as time headway ZL for the green time abortion. For highly loaded intersections these values should lie between 2 s and 3 s. Time headways above 3 s should only be chosen in exceptional cases (e.g. unfavourable intersection geometry, slopes, and high proportion of heavy traffic).

The distance l_D of the detectors from the stop-line depends on the selected time headway value ZL, the amber time period and the cruise speed V (see **Figure 4.2**).

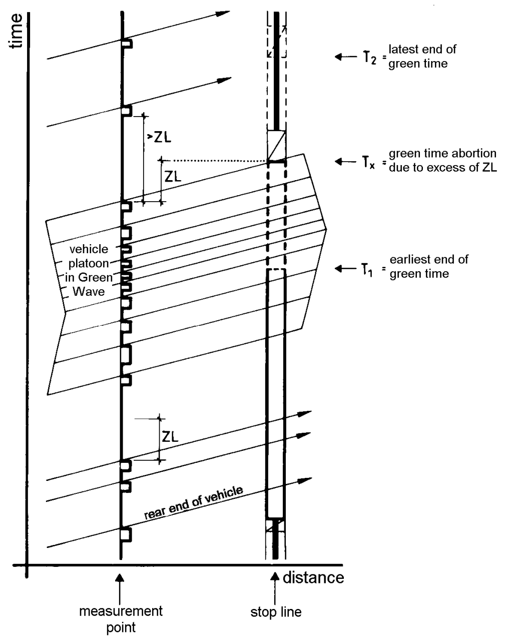


Figure 4.1: Measurement of green time adjustment by means of time headway in a green wave (principle)

Depending on the speeds driven on the respective lanes of the approach, the distances between the detectors and the stop-lines in **Table 4.2** result for time headways of 2 s to 3 s.

In order to ensure motorcycles to be detected, it is recommended to turn the detectors an angle of 30 to 45 degrees to increase the inductance of motorcycles in the magnetic areas.

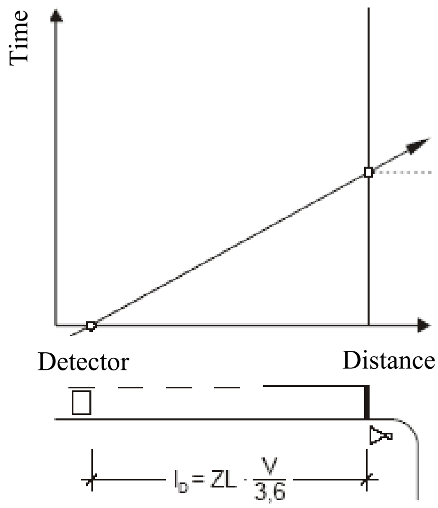


Figure 4.2: Location of the detector for time headway measurement

Table 4.2: Detector distances l_D for a measurement of time headway

V	detector distances at	
	ZL = 2 s	ZL = 3 s
30 km/h	15 m	25 m
40 km/h	20 m	35 m
50 km/h	30 m	40 m
60 km/h	35 m	50 m
70 km/h	40 m	60 m

If the distance l_D between the detectors and the stop-line is fixed, it has to be taken into account when dimensioning the minimum green time t_{min} (see **Figure 4.3**) that all vehicles queuing between detective areas and the stop-line (mean queuing length of a vehicle l_{Fz} and motorcycles in head-start areas) can clear within this period of time (mean time required t_B).

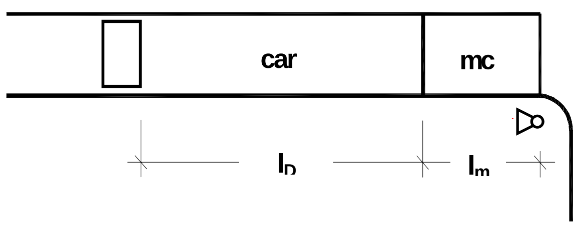


Figure 4.3: Determination of the minimum green time as fixed value

$$t_{min} \geq \frac{l_D}{l_{Fz}}.t_B + t_{mc}$$

Where t_{mc} = necessary green time for motorcycles.

Time headways can also be detected by long loops. However, this kind of long loops is unsuitable for the case of motorcycle traffic because motorcycles standing inside the long loop are sometimes not detected due to being small in size and having a low proportion of metal.

If the detectors do not only serve time headway measurement but also green time request, it is necessary to arrange another detector each directly before the stop-line. This ensures that vehicles can request green time if they were not able to pass the previous green time.

When applying time headway measurement in Green Waves, it has to be taken care that the first vehicles of a platoon arrive at the measurement point before the earliest possibility of aborting the green period.

4.2.2.3. Degree of Occupancy Measurement

This method assesses traffic flow by determining the degree of occupancy, taking into account traffic volume, speed and vehicle length. Under certain conditions, reaction may be slightly more inert than in case of time headway control. Particularly the longer time headways occurring at the start-up of heavy vehicles do not lead to an early green time abortion.

The detective areas are arranged in the same way as for time headway. Their length in the direction of traffic should be approx. 2 m to 5 m.

However, this method cannot be applied to MDCs because it relates to traffic counting. Therefore, the degree of occupancy does not reflect traffic volume in case of motorcycle traffic.

4.2.2.4. Congestion and Queue Length Measurement

Detectors can help controlling critical congestion areas on intersection approaches:

- at the beginning of a turning lane, if there is the risk that congestion caused by turning vehicles tails back onto the parallel straight-ahead lane; here the detector is arranged at the beginning of the turning lane;
- on the straight-ahead lanes at the beginning of a turning lane or an exclusive public transport lane if there is the risk that the turning lane or the special-purpose lane cannot be reached because of the queuing of the straight-ahead traffic; then the detector is arranged on the straight-ahead lane;
- at the beginning of a motorway exit or a similar road;
- at the intersection exit in case of several closely spaced intersections.
- detectors can also help detect and divert the queuing of permitted turning traffic streams at an intersection. In many cases further-reaching design and constructional measures, e.g. establishing left-turning lanes or exclusive lanes for public transport, as well as the widening of narrow intersections can be avoided.

Congestion has been detected if a vehicle's period of occupancy on the detector exceeds a given threshold value. This value must not be too short so that slow-moving long vehicles are not closed by congestion. The threshold values between 5 s and 15 s are recommended.

The detectors for congestion detection are approx. 0.5 m narrower than the lane, and from 1.5m to 2 m wide. It is recommended to turn the detectors an angular of 30 to 45 degrees in order to ensure that they are really occupied in case of motorcycle traffic.

If the congestion detector is arranged on the approaches, it has to be located outside the area of usual congestion due to red. Otherwise congestion would permanently be detected and the green time correspondingly extended. The reaction times in the traffic-dependent strategies have to be taken into account. Until the measures taken to reduce congestion are effective, congestion may even increase.

If it is necessary to determine various queue lengths, so several subsections monitored by congestion detectors have to be distinguished in order to be able to react differently. Besides, the measures evaluating queue lengths have to be taken into account. These measures base on the raw data of conventional time headway from the detectors and the green time of traffic signal systems.

If control reacts to the parameter "congestion", different measures can be taken:

- extension of the green time on the approach concerned, leading to an earlier green time abortion of the previous phase or a shorter green time of the following phase (n). The green time in case of congestion detection should be sufficient to allow all vehicles queuing between stop-line and detector to clear.
- throttling the inflow into the congested area by cutting the corresponding green times at upstream intersections.

4.3. Details on the Control Strategies

4.3.1. Selection of Signal Programs

4.3.1.1. Boundary Conditions

At intersections with sufficient capacity reserves, it may be appropriate to use only one single signal program for all times of the day if a flexible control is realised by the microscopic control level. In general, however, different signal programs adapted to the current traffic situations are applied. They are activated by a time-dependent selection of signal programs.

In case of the rule-based implementation of control, it should be possible to interconnect neighbouring areas by the control algorithm in order to be able to react to crossed area events.

Having defined the area in which a signal program has to be selected, comprehensive traffic analyses have to be conducted, whereby at least the characteristics of the traffic volume and traffic structures have to be investigated and evaluated. Then the signal programs to be applied to the different traffic situations have to be worked out.

The individual signal programs have to cover the varying load levels during a day (peak hours, periods of normal and low traffic load), a week (Weekdays, Sundays, Saturdays, holidays), a year ("normal" times, holidays, main shopping times) and eventually exceptional traffic loads (e.g. pleasure trips, event-related traffic).

A change from one program to another requires a given and defined procedure or the possibility to choose one of several available procedures (see **section 4.5.4**).

4.3.1.2. Time-dependent Selection of Signal Programs

Time-dependent selection of signal program means that an appropriate signal program is selected from a series of given signal programs based on calendar day and time of day. This may be sufficient if high-load periods can be forecasted and stable, i.e. if they are recurring in the course of a day or a week.

The determination and evaluation of the parameters, the calculation of the control values and the signal programs as well as the definition of the switching times for the traffic signal systems take place off-line.

4.3.2. Fixed-time Signal Programs

Depending on local and traffic characteristics, fixed-time signal programs are sufficient for the requirements. Based on traffic volume of motorised traffic and other road user groups concerned, the major importance is attributed to the setting of the cycle time. Because there is no any change of the signal program elements, fixed-time signal programs should be applied especially when the load status keeps unchanged for a long time.

4.3.3. Adaptive Signal Programs

4.3.3.1. Green Time Adjustment

The green time is adjusted to the current demand of the inflowing vehicles after the selected minimum green period has expired or after having reached an earliest point in the cycle. There are several methods of adjusting green times. They mainly differ in the criteria by which a running green time is changed in favour of another traffic stream. Hereby, it can be carried out by help of the following measurements:

- time headway,
- queue length or
- models of calculated parameters as well as waiting times or stops.

4.3.3.2. Phase Swapping

Phase swapping means modifying a given phase sequence on request, while the number of phases is kept. This can be useful, for example, in connection with speeding-up measures for public transport, if the predictive arrival time point does not fall in the time slot of a possible green time adjustment.

4.3.3.3. Demand Phase

By a request, a phase is fitted into a given phase sequence in order to allow temporarily arising traffic streams (e.g. turning traffic, public transport, cyclists and pedestrians) to enter the intersection.

To keep the waiting times for the requesting road users as short as possible, the phases should not become effective at a fixed point in the cycle but until the latest possible point provided by coordination can be activated. If possible, the phase can be admitted at several points in the signal program.

If public transport requests a demand phase, the request signal has to be sent out as early as possible before the stop-line is reached. Since depending on speed and local conditions the request point may be located up to 500 m before the stop-line, additional inquiry criteria are necessary if intersections or stops are closely spaced.

The combination of green time adjustment with the demand phase for public transport allows more or less undisturbed traffic flow, even if separate public transport lanes are only available on some subsections.

4.3.3.4. Time-offset Adjustment

Time-offset adjustment means that the beginning of all green times within the cycle time is variable to a defined value. This is particularly important if the green times of individual phases at the neighbouring signalised intersections are to be co-ordinated optimally.

The time-offset adjustment is particularly suitable for the consideration of traffic load change.

4.4. Coordination

4.4.1. Goals and Objectives

For the coordination, the green times of the successive signalised cross-sections are co-ordinated together with suitable time-offsets. Consequently, the higher number of road users can pass a series of traffic signal systems without stopping.

Co-ordinations are relevant to all road user groups e.g. motorised vehicles, public transport vehicles, cyclists and pedestrians at individual or close neighbouring traffic signal systems. On arterials or in the road network, the coordination is particularly important for motorised traffic and public transport. It is only significant for the cyclists in case of higher speeds.

The coordination mainly serves to decrease the travel times of motorised vehicles in the road network, and to reduce fuel consumption and emissions. For that purpose, but equally for an improved traffic safety, it is aspired to keep the speed variation of individual vehicles and the number of stops of all vehicles as low as possible. The objective in the network is an overall optimisation.

In addition to the traffic and environmentally relevant advantages already mentioned, the implementation in form of Green Wave supports the urban traffic management objectives of bundling traffic flows on major roads and wide-area relief of minor roads.

The planning of a coordination has to take into account the demands of private traffic, public transport, pedestrians and cyclists, possibly those of emergency vehicles from fire brigades, police and ambulances, too. If the different road user groups are to be considered appropriately, temporally and locally differentiated compromises have to be found which do not discriminate any of these groups. It is advantageous to reach the planning objectives. Depending on the target requirements, different co-ordinations can be developed for road users, for example, regarding the possible waiting times of pedestrians, the number of stops of motorised traffic or the optimisation of public transport.

4.4.2. Basic Principles

The coordination can be illustrated in a time-distance diagram. It depicts the vehicular movement in form of so-called green bands (time-distance bands). Their width reflects traffic volume led in the coordination. This width can be varied along the roads of coordination depending on the traffic volume. Road users in form of leading green and lagging green of the continuous green band have to be clearly marked in the time-distance diagram. Regarding traffic volume in this case, motorcycles can be converted into passenger car units, or contrarily.

To develop a co-ordination, the direction and volume of traffic flows have to be determined. Significant changes of traffic situations during the time of day or in special events require different designs.

4.4.3. Coordination at Intersections

In large and big round-about intersections with signalisation at some cross-sections, an inner coordination is particularly important in order to keep traffic streams unstopped. Therefore, the compulsive conditions for the selection of signal phasing and phase sequences can be given.

The boundary conditions for a coordination of public transport at intersections are the same as those of motorised traffic. Additionally, time for stopping has to be taken into account.

A coordination of pedestrians at intersections is significant with:

- successive crossings on roads with central reservations or separating strips,
- sequent crossings at all approaches of the intersection.

The different possibilities for the coordination are explained in section 2.3 (signal program structure).

Signals for cyclists should be coordinated in comparison with pedestrians in which cyclists have faster speeds, need larger queuing space on the central reservations or separating strips and need special waiting areas in case of successive crossings.

The following design principles have to be taken into account when coordinating:

- The time-offset of alternative sequent signals for cyclists has to be determined so that the green time ends at a slow enough speed.
- The beginning of green time for cyclists should be coordinated together so that each successive crossing receives the beginning of the green time at fast speeds.
- These boundary conditions should be taken into account by a joint signalisation with pedestrians if beside the pedestrian crossings, the cyclist crossings are also marked.

4.4.4. Coordination on Arterials

4.4.4.1. Constructional Pre-conditions

When designing a Green Wave for motorised traffic, the following boundary conditions have to be taken into account, which may have considerable impacts on the quality of the coordination:

- More than one continuous lane, or the layout of cycle traffic systems affect positively the quality of the coordination, if possible the cyclists are allowed to overtake on the carriageways.

- A stopping prohibition can avoid a negative influence on the quality of traffic flows due to stopping and parking vehicles.
- Turning vehicles should be assigned separate turning lanes in the intersection area so that go-through traffic is not impeded and rear-end collisions are prevented.
- Pedestrian crossings cross green-wave roads are not permitted.
- Green Wave for motorised traffic is effective for a traffic signal spacing of up to 750 m, under particularly favourable conditions up to 1000 m. A wider spacing makes vehicle platoons broken up and traffic signal coordination normally does not make sense any more.

4.4.4.2. Traffic Engineering Boundary Conditions

The cycle time for a Green Wave coordination must be equal at all intersections. It is indicated as the system cycle time. Firstly, individual cycle times of all intersections on the arterial have to be determined, of those the longest cycle time is selected to design the Green Wave.

With this selection of the cycle time, firstly, it guarantees that all traffic streams at individual intersections can be controlled without restrictive boundary conditions. Short-term deviations from the system cycle time, which may result from green time adjustments and green time requests have to compensate each other.

Short cycles within a system cycle time can be used for traffic flow control in case of

- low-loaded road sections to be connected to major roads,
- individual, scarcely dimensioned queuing space,
- pedestrian signal systems or
- intersections of light traffic crossing.

The total cycle time of the short cycles has to equal the system cycle time.

The **degree of saturation** must be lower than 0.85, hereby a good quality of coordination can be achieved. The aim is to ensure that there is sufficient capacity for motorised traffic at all intersections with Green Wave by the use of constructional and operating measures.

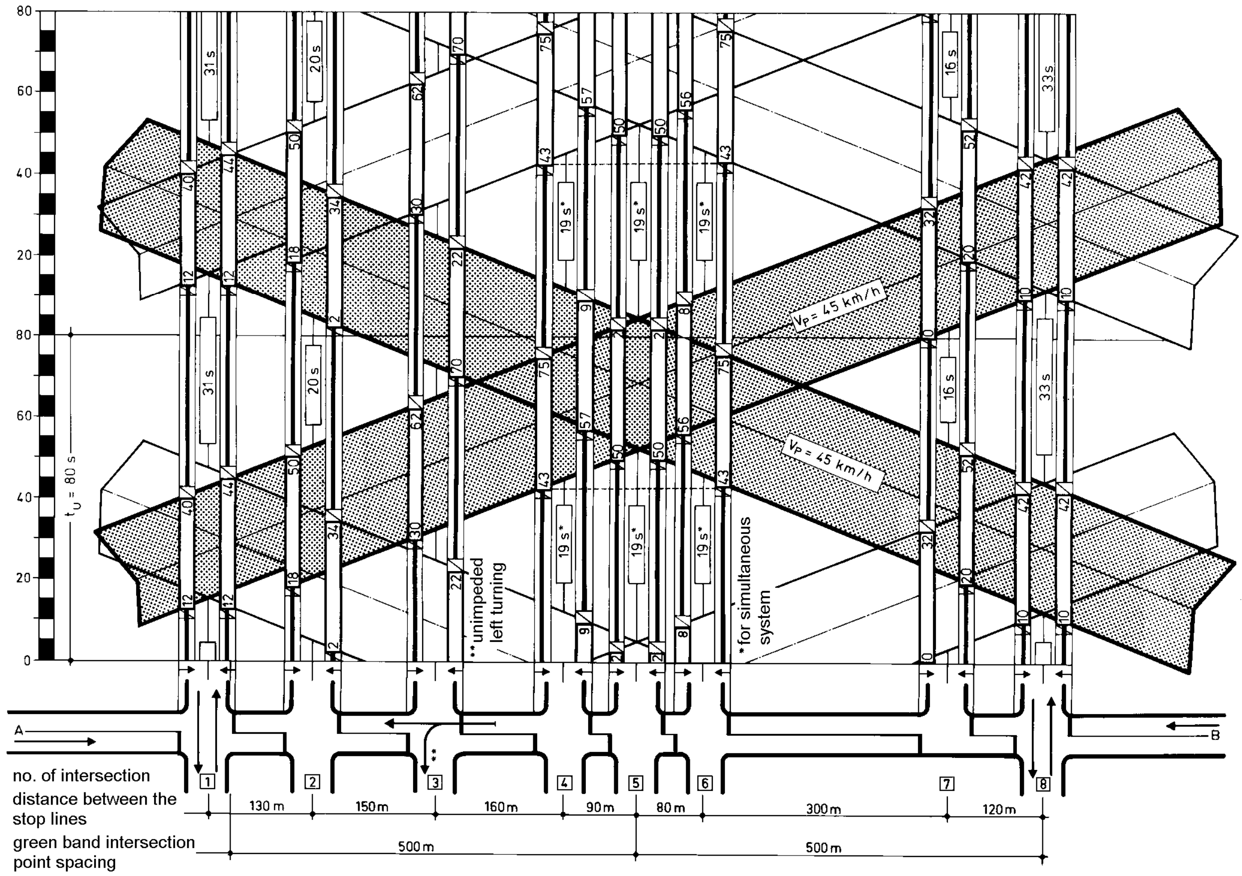


Figure 4.4: Example of a progressive system with continuous green time guidance

The **progression speed** V_P is plotted as the gradient of the central line of the green band in the time-distance diagram. Basically, it should not be higher than the permissible speed on the arterial. Normally, the progression speed from 90% to 100% of the permissible speed is recommended.

Speed-reducing features (e.g. high proportion of heavy traffic, high gradients, narrow bends, bad road surface conditions) have to be taken into account when designing.

The design progression speed of each direction $V_{P,Ri1}$ and $V_{P,Ri2}$ (unit in [km/h]) with the calculated green times for coordinated traffic streams and the green bands of go-through traffic streams are generally illustrated in the time-distance diagram (see **Figure 4.4**).

The green band point (TP) of the Green Wave on two-direction roads is a characteristic point in the time-distance diagram, where the central lines of two opposing green bands are intersecting. The distance between neighbouring points is called green band point spacing l_{TP} [m].

If an intersection is located exactly in the position of the green band point, and with a given cycle time, the period available for the green times of crossing directions and the necessary intergreen times reaches its maximum. The green bands overlap completely.

The minimum is reached if the green bands do not overlap any more. Then, the intersection is located in the green band point spacing.

The cycle time t_U, the progression speeds V_P in direction and counter-direction as well as the green band point spacing are interrelated as follows:

$$t_U[s] = \frac{3.6 \cdot l_{TP}}{V_{p,Ri1}} + \frac{3.6 \cdot l_{TP}}{V_{p,Ri2}} \; [s]$$

Speed signals indicate the speed for the drivers on the arterial of Green Waves at which theoretically the downstream signalised intersection can be passed through without stopping. By decelerating the first vehicles and accelerating the last vehicles of the same platoon, a better cohesion of the vehicle platoon until the next downstream traffic signals and also a better usage of capacity can be achieved.

Because the speed indication refers to the green time signal at the successive traffic signal systems, reliable speed indications are only possible by using fixed-time control.

4.4.4.3. Considerations of Public Transport

The trips of individual public transport vehicles in the system differ very much from those of private traffic platoons. Due to scheduled stops and their limited start-up acceleration and braking deceleration out of consideration for standing passengers, public transport vehicles reach considerably lower travel times on green-wave sections than private traffic.

Varying dwell times at stops lead to varying arrival times of public transport vehicles at the traffic signals.

In order to guarantee a good quality of traffic flow for public transport, their specific characteristics have to be taken into account when designing Green Waves.

Traffic signals can improve the Green Waves for public transport vehicles as follows:

- A door-closing signal is an advantage if the traffic signal system is located directly after the stop. It requests the driver to close the door after passengers have alighted and boarded, mostly shown and ended within 5 s to reach the green time. Subsequently, the traffic signal system can be passed without further deceleration.
- If a signalised intersection is located up to 100 m after the stop, an additional local signal is recommended at the stop. Therefore, in progression, the successive signals at the intersection are switched. Thus, the procedure of deceleration and stop between the stop and the intersection is avoided.
- If a signalised intersection is located more than 100 m after the stop, the forthcoming signals can be used at a measurement distance before the traffic signal system. They display the switched green time signal in progression if the successive traffic signal system can be passed without stopping and not braking the permissible speed. There is no signal term (signal head “Dark”) if a stopover is probably needed.

4.4.4.4. Considerations of Cycle Traffic

If there are major progressive routes of cycle traffic in the network, cycle traffic should be considered to be coordinated, in which the time-distance diagram of cycle traffic are also illustrated in the time-distance diagram of motorised vehicles. In order to be more clearly, it is also necessary to make a separate time-distance diagram for cyclists. It should be considered if cycle traffic can be led in the secondary wave within the green wave of motorised traffic.

A problematic aspect when considering cycle traffic is that when designing the band width at the travelling speed of (about 10 km/h to 25 km/h), it leads to a strong expansion of vehicle platoons from intersection to intersection. When designing, a progression speed between 16 km/h and 20 km/h can be applied.

4.4.5. Coordination in Road Networks

A co-ordination in the road network can be achieved if the arterials cross each other with traffic signal systems. It has the same regulations as the coordination on the arterials, especially the general system cycle time.

There have been problems of competitive requirements of different road user groups and traffic streams, already. The problems are even more complex in the network because more goal conflicts are existing:

- more arterials with green waves crossing each other, the central intersections usually have high traffic volume.
- cluster of traffic streams with high or similar traffic volume as well as unstable traffic situations due to overloaded intersections.
- increasing possible conflicting objectives between road user groups in the road networks.

The larger road networks should be divided into smaller sub-networks. Therefore, the sub-networks can be considered to be represented as transitional points.

4.5. Design a Control Project

4.5.1. Rule-based Implementation of Control Strategies

For the rule-based implementation of control strategies, a diagram process mostly in seconds will be thoroughly run. This bases on conditions and actions, and is led based on the current control situations and parameters to decide the signal program form (see **Figure 4.5**). Swell and comparative values such as time headway value, and travelling time e.g. speed, and also specifications such as permitted green time areas or the green time beginning delay will be defined by parameterizations.

In practice, the following points have proven to be positive for a structural, comprehensible and simply variable control:

- clear outline of the algorithm in simple and straightforward functions,
- guidance in form of comment inside the algorithm,
- application of simple parameters, for example, one- or two-dimensional indicated parameters (parameter rank and parameter field), which allows that parameters can be adjusted without changing algorithm e.g. changing the source code.

The combination of decision and action elements can be presented differently.

For a description of the algorithm, it is recommended to use standardised forms of the program process and structural chart, or decision tables. An example is presented in **Figure 4.6**. For rule-based control, diagram processes and structural charts are usually in the document forms. For a certain case of application, e.g. the traffic-dependent signal program selection in a sub-network, the decision tables as document forms is established. They describe which actions are run under which conditions. The three elements of a decision table are the conditions, the regulations, and the actions. The regulation formulates which situation the conditions must have; hereby the respective action is led. All conditions, regulations and actions are, therefore, arranged according to the priorities.

In order to ensure that processes and control structures must not be iteratively formulated, the special functions and sub-programs should be used, which can be applied as modules at different intersections with the same traffic engineering mode of operation. The characteristics of these modules are controlled by parameters.

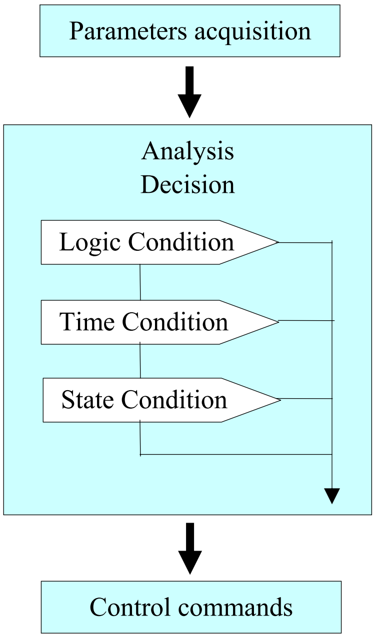


Figure 4.5: Rule-based implementation of a control strategy

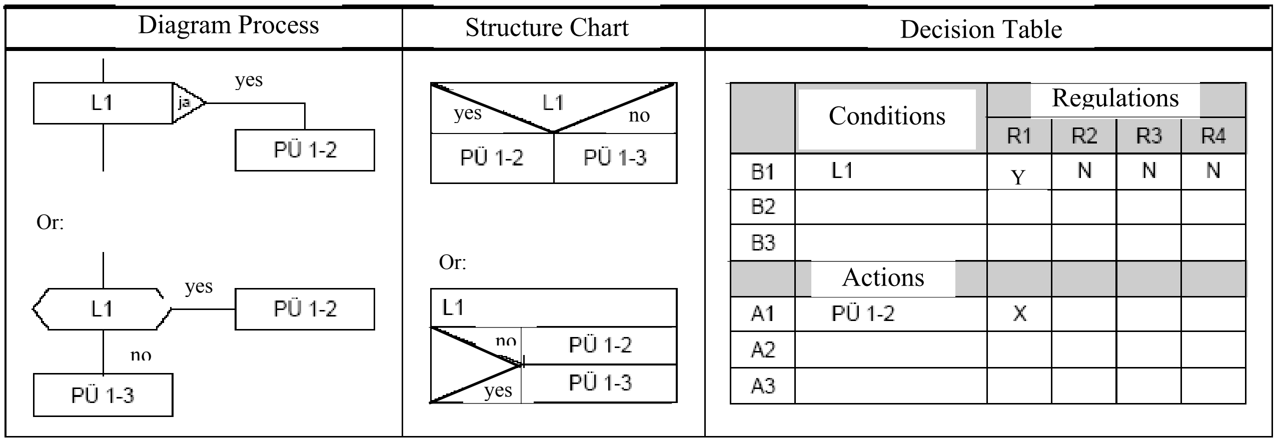


Figure 4.6: A form describing an algorithm query

4.5.2. Standard Rule-based Implementation of Control Strategies

With the standard rule-based implementation of control strategies, traffic-dependent strategies can be formed by a provision of characteristic parameters and control parameters.

Characteristic parameters belong to:

- required parameters
- measurement parameters
- congestion detection
- public transport parameters

Control parameters belong to:

- minimum and maximum green time and red time
- boundary parameters
- phase sequence parameters and
- priority parameters (signal groups, streams, phases)

The parameters will be collected according to the parameter groups; therefore, various traffic situations can be arranged by different parameter groups.

4.5.3. Switching Procedures

4.5.3.1. General Remarks

The change from one signal program to the other is called the switching process. It has to take place at a point in time at which from the traffic engineering point of view the transition makes sense. It should be realised with as few traffic disturbances as possible and be achieved by reasonable control technology expenses.

If at complicated intersections the phase transitions between the same phases of the different signal programs must not be altered, appropriate technical and programming measures have to ensure that the switching begins or terminates within such a phase transition.

When planning switching processes for complex networks, it has to be taken care that due to the available technical devices, considerable secondary conditions may become effective.

4.5.3.2. Switching Point

The switching procedure, at least with individual controls requires the lowest planning expense. The switching points UZP can be set in the coordinated direction either during the green times or outside. They must not be set during a transition period.

With **direct switching**, if there is a request to change the signal program, the currently running signal program is followed until switching point UZP_1 is reached. Having reached UZP_1, the currently running signal program is de-activated and the new program, starting at switching point UZP_2, is activated (see **Figure 4.7**).

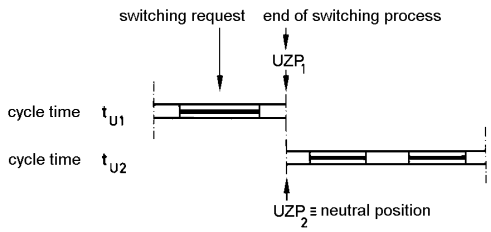


Figure 4.7: Principle of direct switching

Switching including idle period results from different cycle times, respectively one signal program has different UZP and an idle period arises between UZP_1 and UZP_2. (see **Figure 4.8**).

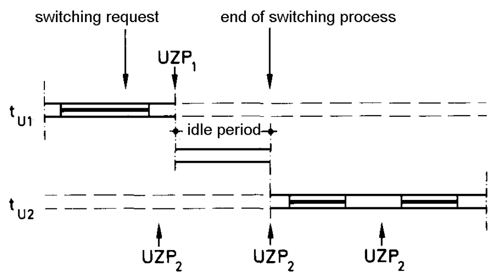


Figure 4.8: Principle of switching including idle period

The maximum idle period is possibly equal to the cycle time. Therefore, idle periods are only reasonable, if the stopped directions can cope with and if a possibly arising queue can be reduced within a short period of time.

The switching process including idle periods offers itself as a simple solution for coordinated control since it does not require too much planning effort. Their duration, however, is limited because of the disturbing impacts on traffic flow.

When switching a different signal program on the basis of traffic-responsive decisions, it has to be taken into account that traffic flow disturbances caused by idle periods again entail further decisions on traffic-responsive switching and therefore may lead to unstable control.

There are various possibilities to reduce the duration of occurring idle periods or to avoid them completely:

- Definition of maximum duration for idle periods. The synchronisation of the signal program to be switched then takes several steps and a number of cycles.
- Selection of the shortest idle period within a given number of cycles. Hereby, it is advantageous to use cycle times whose common multiple is as low as possible.
- Definition of several switching points in each signal program. The idle periods to be expected are determined in pairs. At the shortest idle period the switching process takes place.

4.5.3.3. Switching by Compression and Extension Method

When alternating signal programs according to the compression and extension method, it has to be checked in the alternated signal program whether there is a time point in the signal stage that corresponds with the current signal stage. In this case, the new time point is installed immediately and the new signal program is set up. The synchronisation with the referential time registers can be re-established by a compression and extension method. The aim is to re-establish the synchronisation of the system as quickly as possible without influencing the traffic engineering process.

A compression is only possible with the pre-conditions that the signalisation does not change meanwhile and the minimum time is not shortened.

If the signal program has to be extended, the synchronisation is re-established within a cycle time, too. The waiting time can be spread variably over the signal phases.

The identical signal stage cannot be found until the running program reaches it or the switching point. Therefore, in each case, the switching point of the new signal program can be changed.

4.5.3.4. Switching without any defined switching point

If switching is requested, the currently running signal program is de-activated and its signal stage is compared with that of the requested signal program. If they do not match, the signal groups concerned are converted to the signalisation stage of the requested signal program, taking into account minimum green times $t_{F,min}$, intergreen time t_z, available time-offset conditions and possibly maximum green times and red times in a synchronisation. If all signal groups have reached their targeted stage, the new signal program is activated (see **Figure 4.9**).

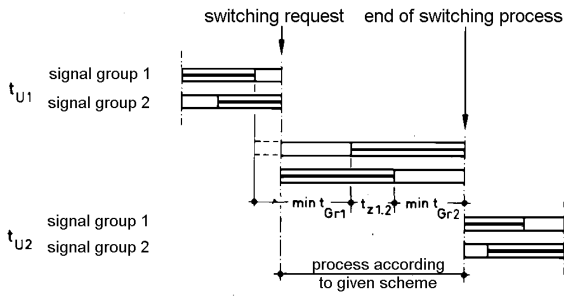


Figure 4.9: Switching principle without any defined switching point

The currently running signal program does not have to be followed until the next switching point defined. The transition from the current signalisation stage to the requested signal program takes place under the given boundary conditions within the shortest possible time.

4.5.4. Testing the Control

By testing the control algorithm, errors can be found. During the procedure, it is checked whether all traffic engineering determinations have been fulfilled. It can take a lot of time to find out such errors on the site after inauguration.

Normally, to implement the test, the test procedure has to be prepared. As many as test cases per a signal program have to be achieved so that every branching points must be tested at least once during the procedure. Complex processes can lead to a high number of test cases required.

The testing of all branching points is eased by automatic tests with a multiplicity of occupancies in the entrance. Automatic tests offer more safety because the unforeseen occurrences of the control can be found.

The test cases should not only be defined according to the planned flow chart, but according to the traffic planning objectives of control. The size of the running tests depends on the measure and should be determined from the operators.

5. Technical Design

5.1. Control Device

The control device is the central component of traffic signal control. Important functions are:

- to activate the connected signal heads,
- to monitor the connected components including radar devices,
- to avoid the dangerous situations of signalisation,
- to control the process of the relevant intersections,
- to collect traffic data by the connected devices,
- to prepare and process data for the devices to run the control logic,
- to prepare and process data for transferring to a higher-level traffic calculation devices.

The control devices can be operated in different forms:

- in a single control without connecting devices for the traffic calculation and without co-ordinating other traffic signal systems,
- in a single control with connecting devices for the traffic calculation, but without co-ordinating other traffic signal systems,
- in coordination with other traffic signal systems, without connecting devices for the traffic calculation and
- in coordination with other traffic signal systems and with connecting devices for the traffic calculation.

5.2. Signal Lamp

5.2.1. Lighting Regulation

The important signal element at a traffic signal system is the optical light signal including signal heads for motorised vehicles, pedestrians, cyclists, and buses as well as auxiliary signals (amber flashing light), and speed signals. Furthermore, there are acoustic and tactile signal heads, which serve the orientation and the green time control for visually impaired people.

5.2.2. Visibilities of the Signals

The visibility of the signals mainly depends on

- the position of the signal heads,
- the luminous intensity and its distribution,
- the size of the luminous section and
- the contrast between luminous section and environment.

Under normal environmental conditions a signal should be visible from a distance of 35 m (80 m) at a permissible speed of 50 km/h (70 km/h). The visibility field to the signals must be kept free, without any obstacle.

Less conspicuous signals can be improved by a higher luminous intensity, a better contrast (design and dimensions of contrasting visors **see 5.2.14**) and by larger diameters of the optical units. The signal heads have to be aligned so that road users can conveniently see the relevant signals from any position when approaching a signalised traffic area, particularly if incoming traffic is split up and sorted onto several lanes and if the waiting position is immediately before the stop-line.

For closely spaced traffic signal systems or signalised intersections situated at the beginning, it has to be taken care that signals are not mistaken or overlooked in the dark. Signal heads with optical units of varying diameters on one road section or changing positions of signal heads for the same traffic stream on successive intersection approaches can only be accepted, if misunderstandings with regard to signal assignment are ruled out and clear visibility is ensured.

It has to be paid attention to the fact that nearby signals are generally more conspicuous than downstream, more distant signals.

5.2.3. Phantom Light

Phantom light impairs signal visibility. It may occur if bright external light enters the signal head and is reflected. Depending on time of day and season phantom light by insolation particularly affects signal heads which are aligned facing East, South or West.

The design of signal heads as well as the size and form of the screening visors (see **section 5.2.13**) influence the luminous intensity of phantom light. It can be further reduced by special lenses or insets.

5.2.4. Size of the Optical Units

In normal cases, signal heads with optical units of 200 mm in diameter are used.

Not depending on the style of the lamps, signal heads with optical units of 300 mm in diameter are recommended for

- non-built-up roads, at least in the main direction,
- large-scale intersections in built-up areas to increase the conspicuousness of the signals, if required by the local circumstances,
- roads with a permissible speed of 70 km/h,
- signal heads for temporarily protected left-turners at intersections (green arrow, amber flashing light) and

- all other cases where the conspicuousness and visibility of the signals cannot be ensured by other measures.

Signal heads with optical units of 100 mm in diameter can also be applied to the signalisation of cyclists.

5.2.5. Operating Voltage

LED lamp is often applied in traffic signal systems. In these cases, the operating voltage is 230 V or 40 V. A standard interface is existed for a variant of 40 V. Besides reducing the electric consumption, LED technic also gives a longer lifetime, a higher safety of phantom, and possibly a longer period of time between maintenances. However, the period of time between maintenances, which is still necessary for regular cleaning of the optics should not be chosen too long.

The classic signal lamps with the operating voltage of 230 V and 10.5 V are still existed. However, they are mostly not applied to the new traffic signal systems.

5.2.6. Vehicle Signal Heads

Signal heads for vehicle signals normally have three optical units showing the colours red, amber and green. The red optical unit is at the top, the amber one in the middle, and the green one at the bottom. In certain cases, two-unit or even one-unit signal heads can also be used.

It has to be indicated by the same direction arrows on the optical units of the signal heads if vehicle signals are assigned to certain directions only. This holds for arrow combinations, too (see **Figure 5.1** and **Figure 5.2**). Luminous arrows on dark background show a better outline but a lower luminous intensity than coloured optical units displaying black direction arrows. By this principle, black direction arrows on amber and red optical units have to be used, and arrows on green units always have to be green luminous arrows on dark background.

In case not all lanes of an intersection approach are released simultaneously, direction arrows on the optical units can be dropped if the lanes leading into different directions are separated by constructional measures so that road users can make out clearly which direction the individual signal heads are assigned to.

If on an intersection approach with turning lanes which are not separated by constructional measures a turning traffic stream is signalised separately by direction arrows, it is generally sufficient to show direction arrows only for the turning traffic stream.

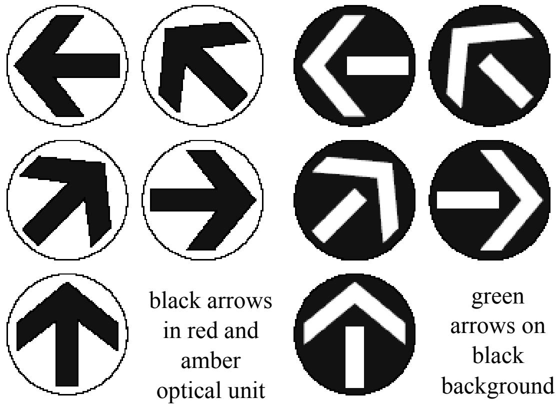


Figure 5.1: Direction arrows

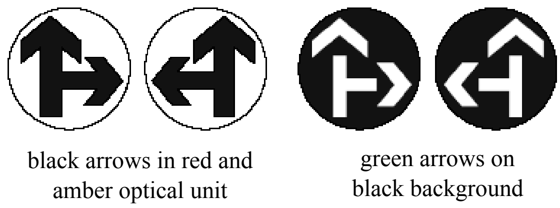


Figure 5.2: Combination arrows

5.2.7. Pedestrian Signal Heads

Signal heads for pedestrian signals have two or three optical units (i.e. with two red units). The green optical unit is at the bottom. The red one has to show the symbol of a standing person, the green one the symbol of a walking person (see **Figure 5.3**).

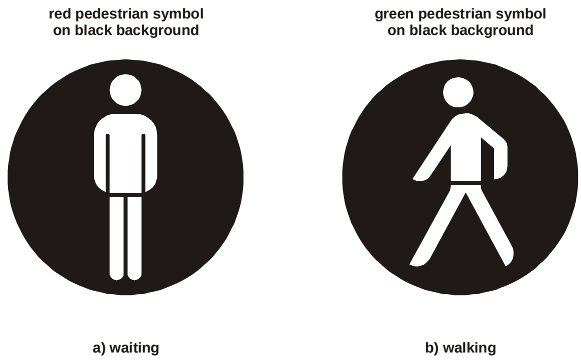


Figure 5.3: Symbol for pedestrian signal

5.2.8. Cycle Signal Heads

For the separate signalisation of cycle traffic, signal heads for cycle traffic have to be safely arranged in front of the conflict area (three optical units). On each standard-sized unit the symbol of a bicycle (luminous on dark background) must be displayed (see **Figure 5.4a**). The red optical unit is at the top, the amber one in the middle and the green one at the bottom. If the signals are assigned to certain directions, all three units also have to display the corresponding luminous arrows together with the bicycle symbol (see **Figure 5.4b**).

If smaller signal heads (e.g. the optical units being less than 110 mm in diameter) are used, the bicycle symbol can be shown as a white luminous symbol on dark background or a scaled-down traffic sign for cyclists can be mounted above. The coloured optical units then do not show any symbols, but possibly direction arrows.

For joint signalisation with pedestrian traffic, cyclists have to observe pedestrian signals. Hereby, the optical units show either pedestrian symbols only or combined symbols for pedestrian and cyclists if a cycle path is also led on the pedestrian crossing. Signal heads showing combined symbols for pedestrians and cyclists are positioned after the conflict area (two-unit signal heads).

a) Standard design

b) With direction arrow (with other directional arrows,

Red symbols on black background

Green symbols on black background

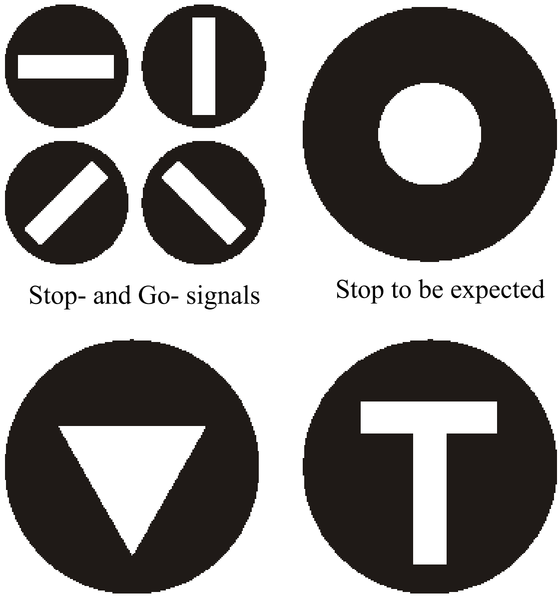
Figure 5.4: Symbol for cycle signals and for combined pedestrian- and cycle signals

5.2.9. Signal Heads for Public Transport Vehicles

Signal heads for buses have to be arranged clearly and visibly upstream of the conflict area to be protected, generally on the right-hand side. They have to be designed according to **Figure 5.5**

For reasons of clarity and signal safeguarding and due to varying intergreen times to be taken into account, generally a separate signal head has to be used for each direction.

At traffic signal systems, in order to be able to accelerate public transport vehicles, an additional signal should be used to provide the drivers a better perception. This signal informs the driver if his/her vehicle was activated in the respective system. The script and its operating instructions as well as other information can be applied according to the transportation company.



Stop- and Go- signals

Stop to be expected

Permissive signal

Door closing

Figure 5.5: Signal heads with white luminous fields on black background

5.2.10. Auxiliary Signal Heads

To warn of hazards, one-unit signal heads with amber flashing light (with or without symbol) can be used. These signal heads have to be located at or before the respective conflict areas (for example, pedestrian crossing, crossings across public transport lanes). As pedestrian signals, the auxiliary signal head has to be arranged behind the conflict areas. However, the auxiliary signals should be used economically and only if there is no other suitable means of warning. Too frequent use wears out the warning effect of the amber flashing light.

On the auxiliary signal heads, only black symbols on amber optical units are allowed, the symbols for “pedestrian”, “cyclist”, “bus” and “rider” (see **Figure 5.6**).

The signal heads have to be positioned so that they warn conspicuously of the hazardous spot.

The (diagonal) auxiliary signal (see **Figure 5.7**) meant to warn permitted left-turners of starting opposing traffic streams in the intersection areas can be designed with or without a black arrow on the amber background. It can be combined with luminous green arrows (diagonal green) and arranged above the green arrow in a two-lens signal head.

Figure 5.6: Symbols for auxiliary signal heads

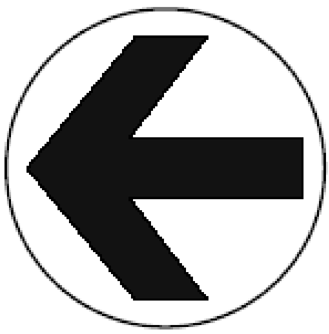


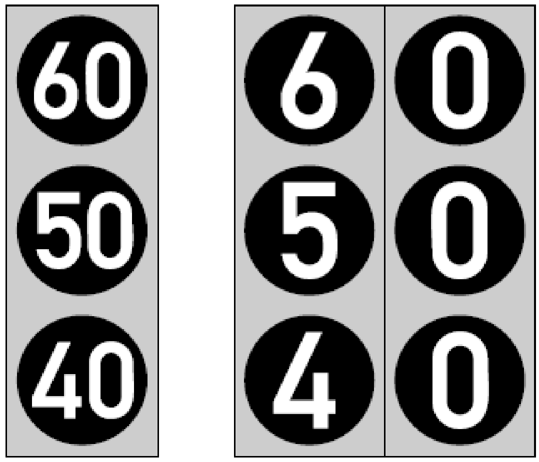
Figure 5.7: Arrow symbol for an auxiliary signal heads with amber flashing

5.2.11. Speed Signal Heads

Speed signal heads are usually positioned on the right-hand side of the carriageway, at the beginning of the subsection for which the speed recommendation is given.

Recommended travel speeds can be indicated by one or multi-unit signal heads showing white luminous figures or by grid signals (see **Figure 5.8**).

The same signal heads can also be used for public transport vehicles (though perhaps in slightly modified design). In order to avoid confusion with speed signal heads for motorised traffic, it is recommended that those for public transport vehicles indicate only one tenth of the respective value (for example, “3” for a recommended travel speed of 30 km/h).



White figures on black background

Figure 5.8: Speed signals (lamp design)

5.2.12. Standardised Design of Symbol in Luminous Fields

In order to ensure a uniform design of symbols, the manufacturers have to be able to rely on standard pictorial material for reproduction.

5.2.13. Hood and Vision Shields at Signal Heads

Hoods are equipped at the optical units of the signal heads as illustrated in **Figure 5.9** to shield against entering external light. Inside they have got a dark, dulled surface to avoid reflecting light.

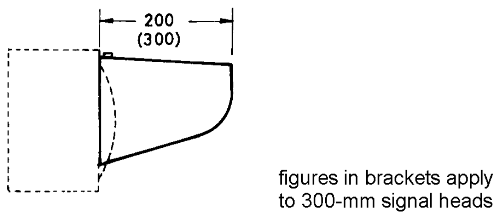


Figure 5.9: Hoods

The installation of **vision shields** is recommended, if road users may be irritated or induced to inappropriate reactions by signals shown to other road users, and if the local circumstances do not allow to change the location of the signal heads,

5.2.14. Backing Boards at Signal Heads

It is recommended to install backing boards to improve the contrast if the conspicuousness of the signals compared to the environment has to be enhanced - particularly against bright background. Hereby, the inner field of the backing boards has to be black. A white border with a black edge is to enhance the perception of the signals. **Figure 5.10** illustrates design and dimensions of backing boards.

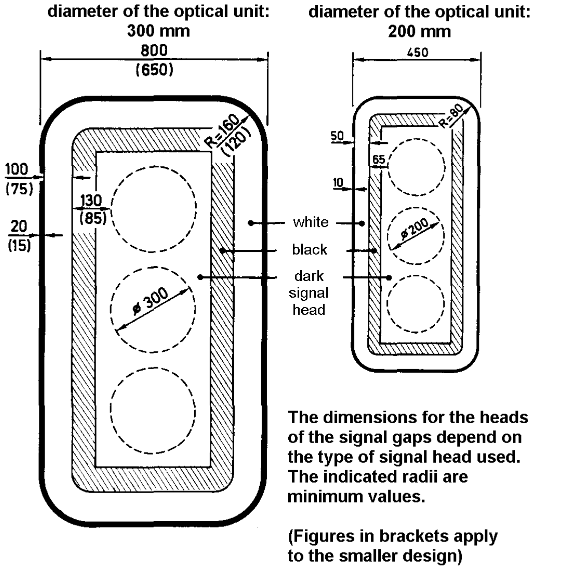


Figure 5.10: Backing boards at the signal head

5.3. Acquisition Equipments

Detectors are applied in order to realise the traffic adaptive control. Hereby, detectors are applied with different physical functions and constructional technic requirements.

- presence and request,
- request on different directions,
- green time measurement,
- queuing length measurement and
- speed calculation.

5.4. Number and Arrangement of Signal Heads

5.4.1. Signal Facilities at intersections

Normally, two signal heads for motorised vehicles have to be given on each approach of the intersection. For more than two-lane approach, further signal heads may become necessary.

The major signal is usually arranged on the right side. Repeated signals can be arranged on the left side and/or above the carriageway.

The signal heads for pedestrian traffic have to be positioned behind the conflict area to be safeguarded. The signal masts have to be set up in an aligned row, generally on the extended central axis of the pedestrian crossing. For narrow crossings it is recommended to position the signal heads on the side, whereby pedestrians feel more comfortable if the signal heads are positioned on that side of the crossing where the vehicle stop-line is marked, so that the drivers are stopped further away. If pedestrian and cycle crossings are jointly signalised, the signal heads should be positioned exactly between the two crossings.

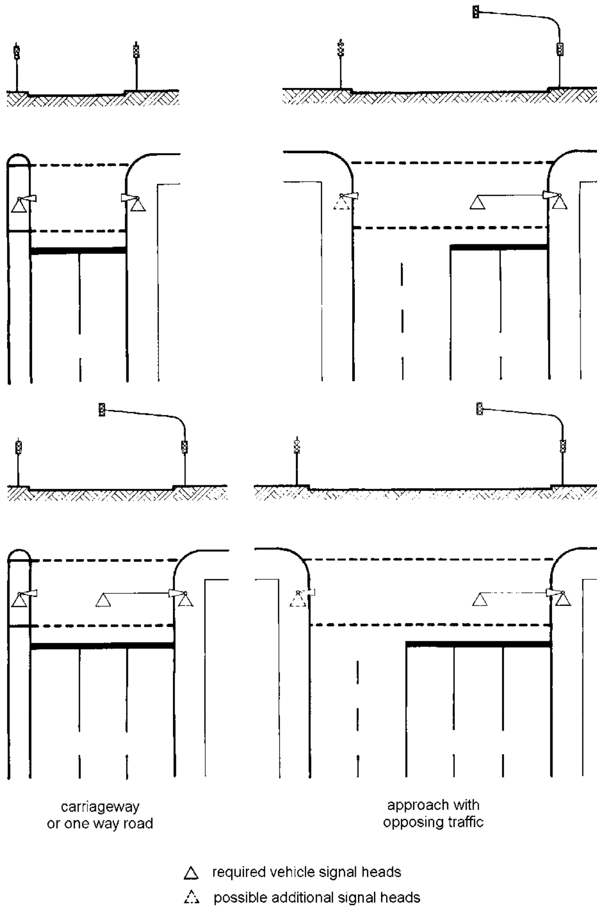


Figure 5.11: Arrangement of vehicle signal heads on one- and two-direction carriageway

If not all lanes on an intersection approach are released at the same time, at least one signal head (showing direction arrows) is necessary for each separately signalised turning direction, and at least two signal heads for straight-ahead traffic (usually no direction arrows).

Signal heads for direction signals are positioned on that side of the carriageway to which traffic turns. Multi-lane turning requires a second direction signal above the carriageway.

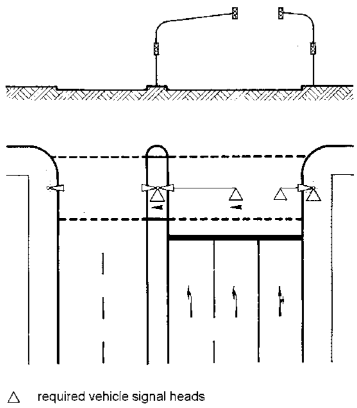


Figure 5.12: Arrangement of vehicle signal heads for two left-turning lanes and separating strip

On intersection approaches with opposing traffic the signal head displaying the direction arrows for left-turners is mounted above the carriageway. If possible, it should be separate from that for the straight-ahead traffic stream. If both signal heads have to be positioned close to each other on the same mast, the direction signals for left-turning traffic have to show luminous arrows.

Sight conditions permitting, the direction signal should be repeated by an additional signal head on the left-hand side of the carriageway.

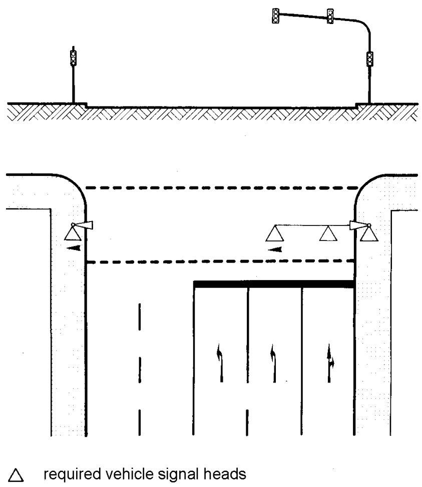


Figure 5.13: Arrangement of vehicle signal heads for two left-turning lanes and no separating trip

In case of indicated lagging and leading green for left-turners, the additional signal head has to be positioned so that the road users can see it only after having entered the inner intersection area. When observing it, left-turners have to be able to pay attention to opposing traffic at the same time. Therefore, it is recommended to mount the signal head on that signal mast which is located diagonally across on the right-hand side of the opposing traffic lane (see **Figure 5.14**).

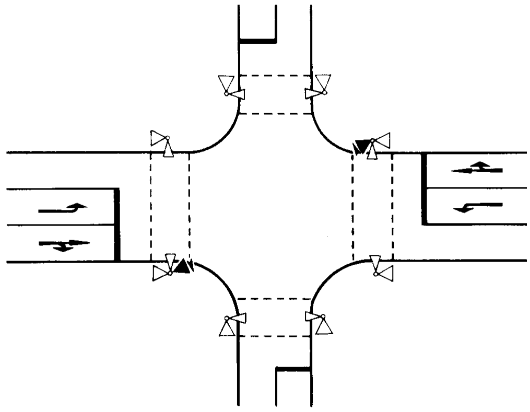


Figure 5.14: Example of the arrangement of direction signal head for left-turners at the intersection

One or two-unit direction signal heads for temporarily protected right-turning movements are arranged on the right-hand side of the three-unit signal head for the main phase (see **Figure 5.15**).

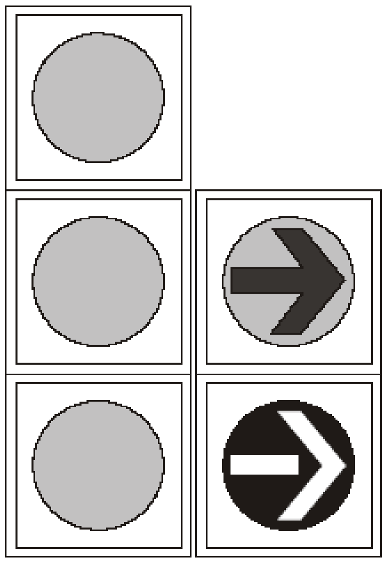


Figure 5.15: Arrangement of a two-unit direction signal head for right-turners

In case of expanded intersections with extra wide central reservation, additional signal heads can be positioned on the central reservation in order to make traffic flow easier and to protect it in the intersection area (see **Figure 5.16**).

This often becomes necessary if left-turners have to be stopped in the intersection area before the straight-ahead opposing traffic. Here, signal heads with three optical units have to be used, which have to be turned towards the carriageway axis in order to avoid unintentional remote effects of the additional signals.

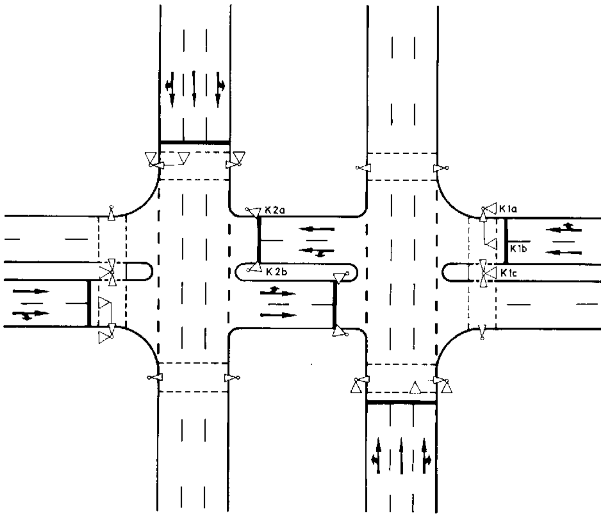


Figure 5.16: Example of the arrangement of additional signal heads at an expanded intersection

In **Figure 5.17**, an exclusive right turning lane is designed along with a go-through lane. In this form of signalisation, a signal head has to be mounted above the go-through lane.

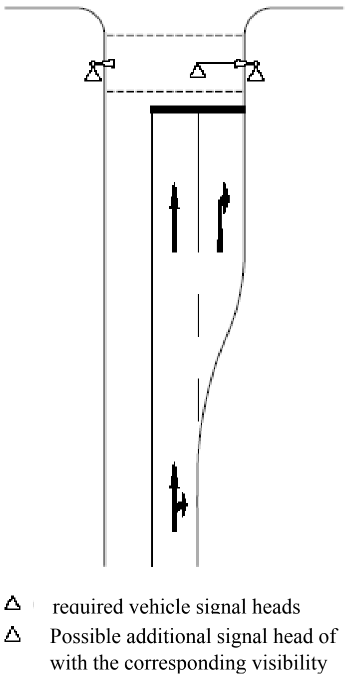


Figure 5.17: Arrangement of the signal head in case of an exclusive right-turning lane

5.5. Constructional Instructions

Signal heads have to be mounted so that they can be assigned unambiguously to the signalised traffic streams and that road users are not confused.

Signal heads are mounted on masts or cantilevers, in certain cases also on signal gantries or cables spanned across the carriageway. Since signal gantries impact the town-scape very much, they have to be designed with particular care. A construction once chosen should be kept for a longer road section.

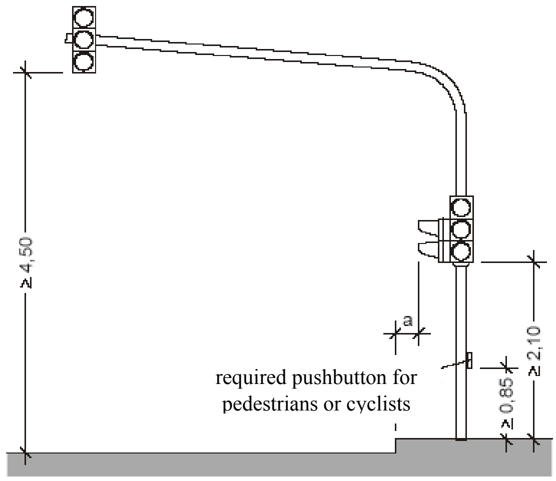
When mounting the signal heads, it has to be taken care that they neither impede nor endanger road users. Therefore, minimum heights above the carriageway and minimum distances from the roadside have to be maintained. The lower edge of the signal heads is to be fixed at least 2.10 m above the sidewalk, 2.20 m above the cycle path and 4.50 m above the carriageway.

Touch buttons for requests by pedestrians and cyclists should be mounted at a height of 105 cm.

The lateral distance (a) from the edge of carriageway of urban roads to the signal head depends on the permissible speed V (see **Figure 5.18**).

On urban roads with a permissible speed of 50 km/h and in constrained circumstances the lateral distance (a) may be reduced by up to 0.20 m. It is also valid for the signal heads on central reservations.

Outside built-up areas, the distance (a) from the edge of carriageway to the outer edge of signal heads is usually 1.50 m.



V [km/h]	a [m]
≤ 70	≥ 0,75
≤ 50	≥ 0,50

Figure 5.18: Minimum heights and minimum distances from signal heads

Annex 1: Details on the Traffic Load

Pre-condition for planning traffic signal systems at intersections, and for calculating the number of lanes is traffic volume.

For the development of the existing intersections, motorised traffic volume of the typical traffic situations must be available. Therefore, traffic streams at the intersection have to be collected for the relevant periods of time (sub-divided by the interval of 15 minutes as well as of an hour). In addition, it is necessary to collect traffic volume of the main directions in a longer period of time (for example 16 hours). Then, signal programs have to be provided for the characteristic periods, normally for:

- peak hours in the morning
- normal hours (between the peak hour in the morning and in the afternoon)
- peak hours in the afternoon
- weak hours (low traffic volume)

From the traffic data collected, it is necessary to consider whether other signal programs are needed for a certain period. Furthermore, the signal programs for traffic in the weekend and special events have to be provided.

If traffic volume does not change much during the peak hour, an extrapolation of the traffic volume is not required. Only in case of traffic volume changing much during the peak hour, the investigations based on the interval of 15 minutes are conducted. The calculation method is carried out so that a consecutive 15-minute interval can be derived.

For the calculation of queue length and the number of lanes, relevant traffic volumes in the peak hour have to be shown.

In the traffic collection, vehicle types have to be distinguished. Therefore, the proportion of heavy vehicles to car traffic volume, and motorcycle traffic volume has to be given. Cyclists have to be collected separately. Depending on the intersection layouts proposed, it might be not necessary to convert other types of vehicles into passenger car units when calculating signal program elements.

Traffic volume of public transport can be received directly by counting, or by the time schedule of public transport. Furthermore, time for stopping at the bus-stop should be taken into account if the bus-stop is located on the approach.

High traffic volume of pedestrian in the relevant directions has to be collected, otherwise the average value of two directions is generally enough.

Annex 2: Traffic Flow Quality

2.1. Criteria for Traffic Flow Quality

On the approach or at the crossing, when the green time terminate, the individual road user has to wait at traffic signals. The waiting time is the most important criterion to estimate the traffic flow quality. Depending on point of times for arriving at and releasing traffic signals, the waiting time of individual road users is different, it means that the waiting time is randomly variable. In practice, therefore, the average waiting time is considered.

Besides the waiting time, other criteria are also considered to estimate traffic flow quality, for example, the number of vehicles in queue length, the number of stops or through drives, degree of saturation, the number of cycle times overloaded.

The application of individual criterion can be:

- calculated analytically (realistic computed models must be available) and/or
- able to be investigated by simple measurement technics

2.2. Level of Traffic Flow Quality

According to FGSV (2001), traffic flow quality are classified into six levels from A to F depending on the boundary of average waiting time for individual transport modes as shown in the following table:

The individual quality level has following meanings:

Level A: The high number of the road users can pass the intersection unimpeded. The waiting times are very short.

Level B: All the road users arriving during the previous red time can pass the intersection during the next green time. The waiting times are short.

Level C: Almost all road users arriving during the previous red time can pass the intersection during the next green time. The waiting times are perceptible. The vehicles arriving in the middle can only decrease congestion at the end of the green time.

Level D: In motorised traffic, the rear-congestion is always available. The waiting times for all road users are significant. The traffic situation is still stable.

Level E: The road users are in substantial competition each other. In motorised traffic, congestion grows gradually. The waiting times are very long. The capacity will be achieved.

Level F: The traffic demand is higher than the capacity. The vehicles must drive up to get departure. Congestion grows continuously. The waiting times are extremely long. The intersection is overloaded.

For the motorised traffic, in case of not coordinated, traffic flow quality is estimated by the average waiting time. But in case of coordinated, the quality should be estimated by the percentage of through drives or by the number of stops, which correspond with the goals of coordination.

The estimation of the waiting time for public transport at signalised intersections is combined with the operation at the bus-stop.

For pedestrian traffic, if there is only one crossing, the maximum waiting time can be equal to the red time in fixed-time signal control. In case of more than one crossing, the signal timing of the crossings should be coordinated so that the approach can be passed by one walk. The average waiting time in individual quality levels in **Table 2.1** will be added by 5 s in case of more than one crossing on the approach.

Table 2.1: Boundary value of the quality levels for different road user groups and transport modes

Quality Level	Permitted Average Waiting Time w [s]				Percentage of through drives without stop (%)
	Public Transport	Cycle Traffic	Pedestrian Traffic (1)	Motorised Traffic (not coordinated approaches)	Motorised Traffic (coordinated approaches)
A	≤ 5	≤ 15	≤ 15	≤ 20	≥ 95
B	≤ 15	≤ 25	≤ 20	≤ 35	≥ 85
C	≤ 25	≤ 35	≤ 25	≤ 50	≥ 75
D	≤ 40	≤ 45	≤ 30	≤ 70	≥ 65
E	≤ 60	≤ 60	≤ 35	≤ 100	≥ 50*
F	> 60	> 60	> 35	> 100	< 50*

1) Adding 5 s in case of more crossings (FGSV, 2001)

* Ineffective coordination

Annex 3: Traffic Engineering Calculation

3.1. General Remarks

After designing dimensions for the head-start lanes of motorcycles as presented in section 3.2.1 of these guidelines, all the following contents are used to apply to car traffic that are backward to the head-start lanes. Therefore, it is noted that when calculating, all the parameters for cars are used even the green times for car traffic as calculated in section 2.7.1 of these guidelines (calculation of the green time).

The average waiting time for motorcycle riders is calculated similarly to the case of cycle traffic.

The capacity of motorcycle traffic is calculated similarly to car traffic. However, all relevant parameters of motorcycles must be used.

3.2. Saturation Flows

3.2.1. Saturation Flow of Car Traffic

The German Highway Capacity Manual 2001 provided the values of the saturation flows depending on the green time period (see **Table 3.1**):

Table 3.1: Saturation flow values of car traffic depending on the green time

Green time period t_F [s]	Saturation flow rate $q_{S,st}$ [veh/h]	Saturation headway [s/veh]
> 10	2000	1.8
10	2400	1.5
6	3000	1.2

(FGSV, 2001)

Then, these saturation flows are adjusted by the following formula:

$$q_S = f_1 . f_2 . q_{S,st}$$

Where:

f_1, f_2 = two highest factors among the five factors as presented in **Table 3.2**

$q_{S,st}$ = saturation flow rate as given in **Table 3.1**.

Table 3.2: Adjustment factors for the saturation flow

Parameters		**Adjustment factor**
Heavy vehicle proportion	SV < 2 %	$f_{SV} = 1$
	SV = 2 ÷ 15 %	$f_{SV} = 1\text{-}0.0083\ e^{0.21\ SV}$
	SV > 15 %	$f_{SV} = 1 / (1+0,015 . SV)$
Lane width	2.6 m	$f_b = 0.85$
	2.75 m	$f_b = 0.90$
	≥ 3.00 m	$f_b = 1.00$
Turning radii	R ≤ 10 m	$f_R = 0.85$
	≤ 10 m	$f_R = 0.90$
	> 10 m	$f_R = 1.00$
Approach gradient	+5 %	$f_S = 0.85$
	+3 %	$f_S = 0.90$
	0 %	$f_S = 1.00$
	-3 %	$f_S = 1.10$
	-5 %	$f_S = 1.15$
Pedestrian traffic	high	$f_F = 0.80$
	medium	$f_F = 0.90$
	low	$f_F = 1.00$

(FGSV, 2001)

3.2.2. Saturation Flow of Motorcycle Traffic

The saturation flow of motorcycle traffic is given in **Table 3.3**:

Table 3.3: Saturation flow of motorcycle traffic

Lane width [m]	Saturation flow rate [MCU/h]
2.75	9,500
3.00	10,000
3.50	11,000
4.50	13,000

(Nguyen Hien and Montgomery, 2007)

The equivalent factor converting other types of vehicles into motorcycles is given in **Table 3.4**:

Model 1: The traffic stream contains only motorcycles and passenger cars, in which passenger cars drive straight-on, only.

Model 2: Similar to model 1, but passenger cars can turn right or left.

Model 3: Traffic stream contains all types of vehicles, and all vehicles can go straight-on, turn right or left.

Model 4: Based on the model 2 with a longer period of green time counting.

Model 5: Based on the model 3 with a longer period of green time counting.

Table 3.4: Equivalent factor converting the other vehicles into motorcycle units

MCU value	Period-based			Cycle-based	
	Model 1	Model 2	Model 3	Model 4	Model 5
Straight-on car	3.67	3.04	3.19	4.58	3.81
Right turning car		4.61	4.56	4.69	5.70
Left turning car		4.63	4.88	5.55	5.81
Straight-on van			5.01		4.82
Right turning van			6.82		8.56
Left turning van			6.44		6.67
Straight-on bus			8.43		7.96
Right turning bus			8.73		9.07
Left turning bus			9.40		10.21

(Nguyen Hien and Montgomery, 2007)

3.3. Average Waiting Time

3.3.1. Waiting Time of Motorised Traffic

The waiting times presented in **Annex 2** are the most important criteria to estimate traffic flow quality at traffic signals. For motorised traffic, the waiting time covers the total lost time that vehicles have to undergo comparing with continuously driving. The waiting time includes two parts:

- Basic waiting time w_1 is the waiting time, which results from the red time of the approach by traffic signal systems without consideration of rear-congestion (queue length).
- Rear-congestion waiting time w_2 is the waiting time, which is caused by vehicles that could not be released during the green time (queue length at the end of green), and therefore impede the successive vehicles.

The average waiting time of a vehicle (w) in the fixed-time signal control is calculated by the following relationship:

$$w = w_1 + w_2 \qquad (6\text{-}19a)$$

$$w = \frac{t_U(1-f)^2}{2(1-q/q_S)} + \frac{3600.N_{GE}}{f.q_S} \qquad (6\text{-}19b)$$

Where:

w = waiting time of one vehicle [s]

t_U = cycle time [s]

f = proportion of the green time = t_F/t_U [s]

q = traffic volume on the relevant lane [veh/h]

q_S = saturation flow rate of the relevant lane [veh/h]

N_{GE} = average queue length at the end of the green time during the investigated time (normally, 1 hour) [veh].

The value of the second term in formula (6-19a) and (6-19b) depends considerably on queue length, which is recorded at the end of the green time.

Depending on the random characteristics of traffic flow, the average rear-congestion (queue length) may be ignored if the saturation degree $g \leq 0.65$, it means that $w_2 = 0$. With the saturation degree between $g >$ 0.65 and 0.90, the rear-congestion is even out by a constant value. From $g > 0.90$, rear-congestion grows depending on time, it means that rear-congestion must be accurately given on which period of time T is investigated or on how many cycle times $U = T/t_U$ (U = the number of cycle times) should be investigated. The formulas for calculating rear-congestion by a random traffic flow are given in **Table 3.5**. With these formulas, if the saturation degree is from $g > 0.90$, it is possible to determine the rear-congestion during the total period of time for investigation, or during any cycle time, too.

The waiting time in formula (6-19) cannot be applied to co-ordinated traffic signal systems while they are oversaturated. The calculation of waiting time for permitted left-turning movements and for right-turning movements that have to pay attention on pedestrian traffic will be described in **section 3.6**.

Table 3.5: Formulas for determining the queue length (rear-congestion)

Degree of saturation	Formula for determining the average rear-congestion (queue length) during the investigation time T, which covers U cycle times	Details
$g_1 \leq 0{,}65$	$N_{GE} = 0$	Rear-congestion does not depend on time, and constant.
$g_2 = 0{,}90$	$N_{GE} = \dfrac{1}{0{,}26 + m/150}$	
$g_3 = 1{,}00$	$N_{GE} = 0{,}3476\sqrt{n_C U}^{\,0{,}565}$	Rear-congestion depends on time, grows from cycle time to cycle time.
$g_4 = 1{,}20$	$N_{GE} = [n_C(g-1)U + 25 - 20g]/2$ $N_{GE} = 0{,}1.n_C.U + 0.5$	
$g_5 > 1{,}20$	$N_{GE} = n_C(g-1)U/2$	
	Formula for determining the average queue length (rear-congestion), which is available in cycle time U.	
$g_1 \leq 0{,}65$	$N_{GE} = 0$	Rear-congestion does not depend on time, and constant.
$g_2 = 0{,}90$	$N_{GE} = \dfrac{1}{0{,}26 + m/150}$	
$g_3 = 1{,}00$	$N_{GE,U} = 0{,}545\sqrt{n_C U}^{\,0{,}565}$	Rear-congestion depends on time, grows from cycle time to cycle time.
$g_4 = 1{,}20$	$N_{GE,U} = [n_C(g-1)U + 25 - 20g]$ $N_{GE,U} = 0{,}2.n_C.U + 1$	
$g_5 > 1{,}20$	$N_{GE,U} = n_C(g-1)U$	
Inter-value	$N_{GE,g} = N_{GE,gi} + \dfrac{N_{GE,gi+1} - N_{GE,gi}}{g_{i+1} - g_i}.(g - g_i)$	Inter-values determined by linear interpolation
Where: N_{GE} = the number of vehicles congested by the end of green time (average queue length) [veh] n_C = the maximum number of vehicles, which can pass during the green time of the cycle time = t_F . q_S / 3600 [veh] m = the average number of vehicles arriving = q. t_U / T [veh] U = the number of cycle times, which covers the investigation time T = period of time for investigation = U. t_U [s] t_U = cycle time [s] q = traffic volume [veh/T]		

(FGSV, 2001)

With the number of vehicles releasing n_C = 5, 10, 15, 20, and the degree of saturation g > 0.90, the queue length can be taken from **Figure 3-1** depending on the number of cycle times.

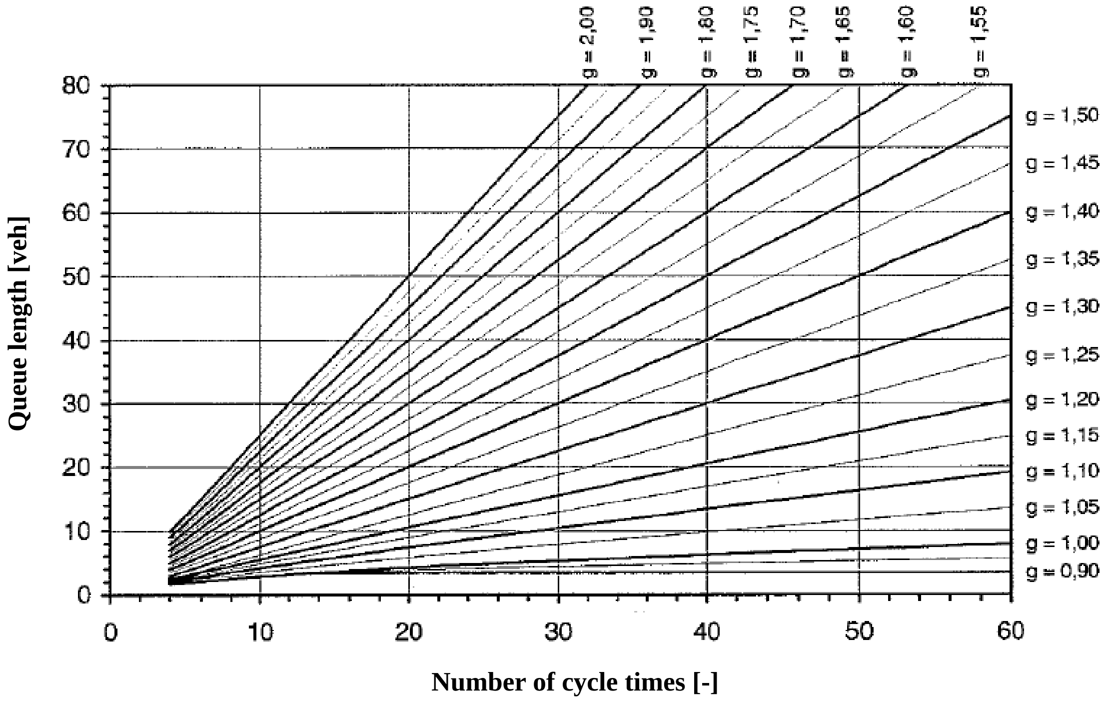


Figure 3-1a: Average queue length depending on the number of cycle times in case of n_C = 5 [veh] releasing

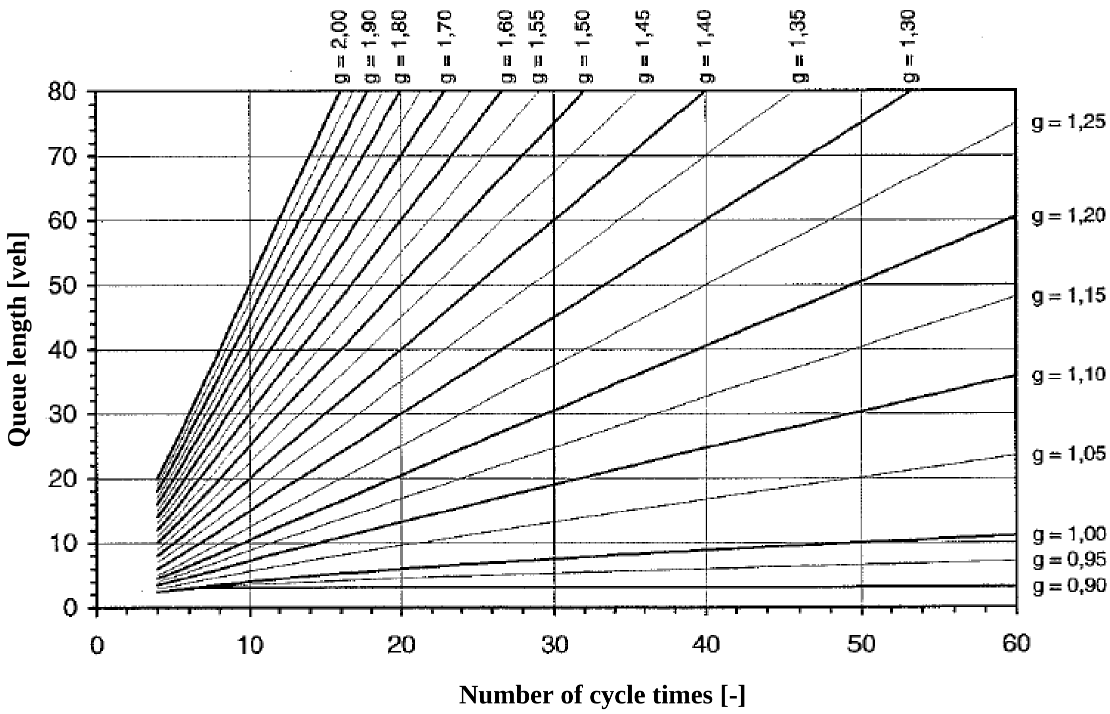


Figure 3-1b: Average queue length depending on the number of cycle times in case of n_C = 10 [veh] releasing

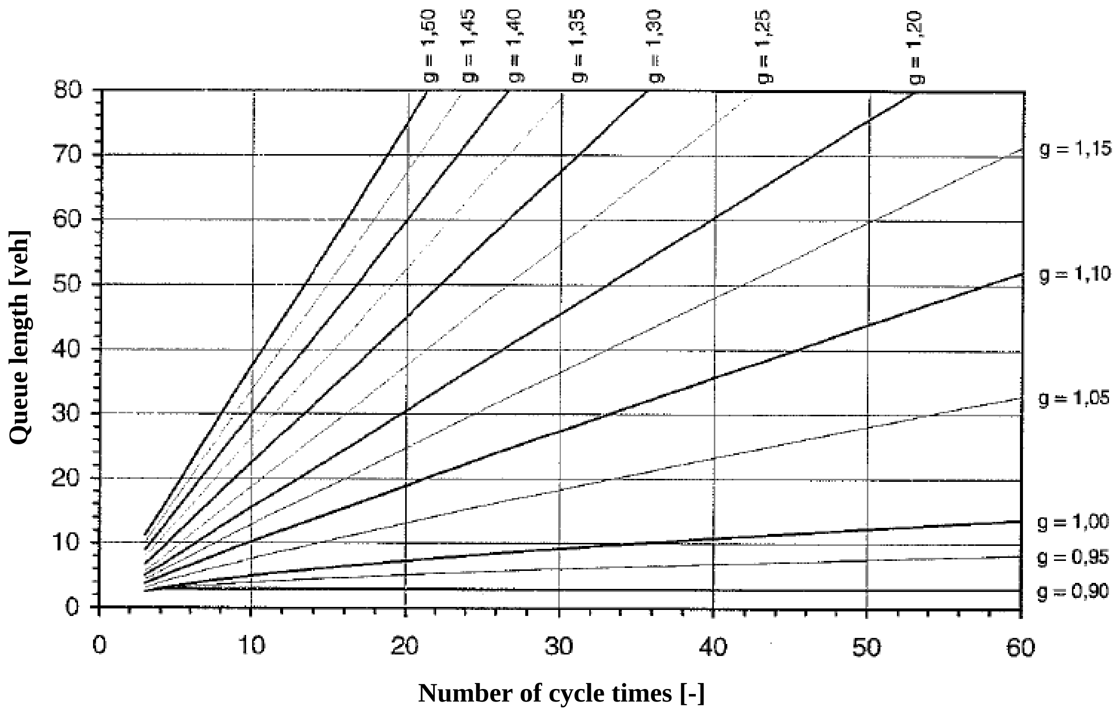


Figure 3-1c: Average queue length depending on the number of cycle times in case of n_C = 15 [veh] releasing

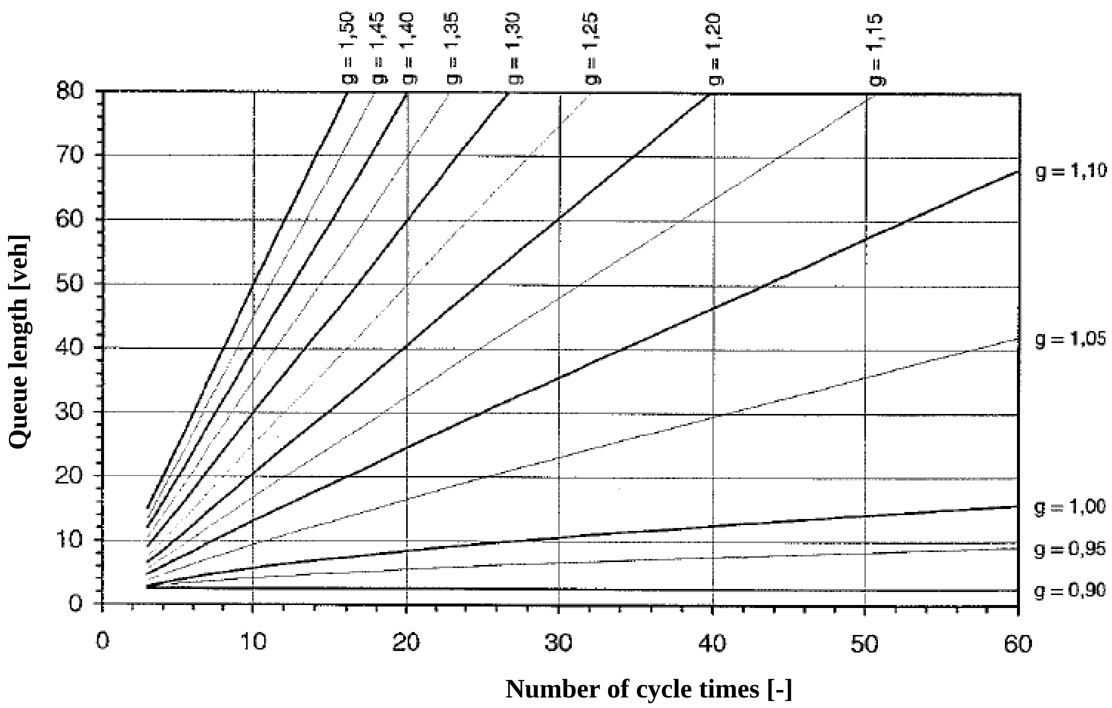


Figure 3-1d: Average queue length depending on the number of cycle times in case of n_C = 20 [veh] releasing

Application for calculating the waiting times is explained in the following examples:

Example 1:

Given:

Traffic volume on a lane q = 504 veh/h

Heavy vehicle proportion SV = 12 %

Adjustment factor $f_{SV} = 1 - 0{,}0083.e^{0{,}21.12} = 0{,}9$

Saturation flow rate q_S = 2000 * 0.9 = 1800 veh/h

Cycle time t_U = 90 s

Green time t_F = 28 s

Period of time for the investigation T = 1h

Calculation:

Time headway value t_B = 3600/1800 = 2 s/veh

The maximum number of vehicles releasing n_L = 28/2 = 14 veh

The green time proportion f = 28/90 = 0,31

Degree of saturation g = 54 * 90/(28*1800) = 0,9, it means that rear-congestion does not depend on periods of time, and is constant.

With the average number of vehicles arriving m = 504 * 90/3600 = 12,6 veh, from **Table 3.5**:

$$N_{GE} = 1/(0{,}26 + 12{,}6/150) = 2{,}91 \text{ veh}$$

Applying formula (6-19b)

$$w = \frac{90.(1-0{,}31)^2}{2.(1-504/1800)} + \frac{3600.2{,}91}{0{,}31.1800} = 48{,}5 \text{ s}$$

Example 2:

Traffic volume q = 560 veh/h, the other input data are the same as in Example 1.

Calculation:

Degree of saturation g = 560 * 90/(28 * 1800) = 1,0, it means that it reaches saturated. Period of time for investigation of 1 hour covers U = 3600/90 = 40 cycle times/h. For the 40th cycle time, rear-congestion having been formed after 39 cycle times is relevant. Simply, the average rear-congestion is calculated after 40 cycle times. The average rear-congestion within periods of investigation is considered by the formula in **Table 3.5** with g = 1 (the first half of **Table 3.5**):

$$N_{GE} = 0{,}3467.\sqrt{14}.40^{0{,}565} = 10{,}5 \text{ veh}$$

$$w = \frac{90.(1-0{,}31)^2}{2.(1-560/1800)} + \frac{3600.10{,}5}{0{,}31.1800} = 98{,}8 \text{ s}$$

With the formula of rear-congestion (the second half of **Table 3.5**), it is possible to determine rear-congestion at any cycle time according to the relationship: $N_{GE,U} = 0{,}545.\sqrt{n_L}.U^{0{,}565}$ for g= 1.

Therefore, rear-congestion can be calculated after:

the first cycle time: $N_{GE,1} = 0{,}545.\sqrt{14}$ = 2 veh

the 40th cycle time: $N_{GE,1} = 0{,}545.\sqrt{14}.40^{0{,}565}$ = 16,4 veh.

The average rear-congestion for the total periods of investigation is approximately calculated by the arithmetical average of two values (2,0 + 16,4)/2 = 9,2 veh. Hereby, the curve of the rear-congestion is convex, the value is a little decreased (9,2 veh comparing to 10,5 veh). However, it will be recorded accurately in individual cases concerning the waiting time (w = 90,5 s).

Example 3:

The average waiting time of vehicles shall be determined by the following traffic data:

Interval	1	2	3	4	other intervals
q [veh/15 min)	133	140	161	147	112

The other input data are the same as in example 1.

Calculation:

Firstly, degree of saturation must be calculated:

Interval	1	2	3	4	other intervals
Degree of saturation	0,95	1,0	1,15	1,05	0,80

The rear-congestion (queue length) after the 1st, 10th, 20th, 30th, 40th cycle time must be calculated. Then, the average waiting time can be determined similarly to the solutions of Example 2.

Rear-congestion (queue length) after the 1st cycle time

g = 0,90; m = 13,3 veh/cycle time; N_{GE} = 2,9 veh

g = 1,00 N_{GE} = 2,0 veh

The value of N_{GE} = 2.0 veh will be selected because the value of 2.9 veh with the degree of saturation g = 0.9 gives the higher number of cycle times.

Rear-congestion (queue length) after the 10th cycle time

g = 0,9 N_{GE} = 2,9 veh

g = 1,0 $N_{GE} = 0{,}545.\sqrt{14}.10^{0{,}565} = 7{,}5$ veh

Interpolation g = 0,95 N_{GE} = 5,2 veh

Rear-congestion (queue length) after the 20th cycle time

From the 10th to the 20th cycle time, degree of saturation prevails at g = 1, in which after the 10th cycle time, the rear-congestion of 5,2 veh is recorded.

U(x) – U(5,2) = 10

U(5,2) is determined from $5{,}2 = 0{,}545.\sqrt{14}.U^{0{,}565}$ and the result is: U(5,2) = 5,3. Therefore U(x) = 10 + 5,3 = 15,3.

The rear-congestion after the 20th cycle time is determined as follows:

$$N_{GE} = 0{,}545.\sqrt{14}.15{,}3^{0{,}565} = 9{,}5 \text{ veh}$$

Rear-congestion (queue length) after the 30th cycle time

g = 1,15 with N_{GE} = 9,5 veh after the 20th cycle time

U(x) – U(9,5) = 10

U(x) = 10 + U(9,5)

Now, the formulas for calculating rear-congestion must be applied in a suitable way, in which how many cycle times under traffic volume of the third interval are required in order to form the rear-congestion. Similarly, it is given in the 20th cycle time, at the end of the second interval. The question is that by which number of cycle times, its average rear-congestion of 9,5 veh is set up.

U(9,5) = ? for q = 161 veh/15 min (m = 16,1 veh/cycle time) and g = 1,15

In addition, the iteration of calculation is required, but it is not here described.

U(9,5) = 3,7 is derived. The example confirms the result as follows:

$$\left.\begin{array}{ll} g = 1{,}00 & N_{GE} = 4{,}3 \\ g = 1{,}20 & N_{GE} = 11{,}4 \end{array}\right\} \; g = 1{,}15 \;\; N_{GE} \approx 9{,}5 \text{ veh}$$

With U(x) = 13,7, gives:

$$\left.\begin{array}{ll} g = 1{,}00 & N_{GE} = 8{,}9 \\ g = 1{,}20 & N_{GE} = 39{,}4 \end{array}\right\} \; g = 1{,}15 \;\; N_{GE} = 31{,}8 \text{ veh}$$

Rear-congestion (queue length) after the 40th cycle time

g =1,05 with N_{GE} = 31,8 veh after the 30th cycle time

U(x) – U(31,8) = 10

U(31,8) = 30

U(x) = 40

$$\left.\begin{array}{ll} g = 1{,}00 & N_{GE} = 16{,}4 \\ g = 1{,}20 & N_{GE} = 113 \end{array}\right\} \; g = 1{,}05 \;\; N_{GE} = 40{,}6 \text{ veh}$$

After the 40th cycle time, a non-saturated section with g = 0,8 is installed. For g = 0,8, a rear-congestion of 1,8 veh is formed.

With the degree of saturation of less than 0,9, a constant and independent rear-congestion from the number of cycle times is set up. An available long rear-congestion under these cases can be removed when the following conditions are satisfied: $n_L > (m + 0{,}5 \,.\, N_{GE})$ with N_{GE} = const.

The available reserved capacity that can be used for the cancellation of congestion is determined by $n_L - (m + 0{,}5 \,.\, N_{GE})$.

The number of cycle times in which congestion is removed can be approximately determined by the following relationships:

$$U = \frac{N_{GE,before}}{n_L - (m + 0{,}5.N_{GE,after})}$$

For example, it is derived:

$$U = \frac{40{,}6}{14 - (11{,}2 + 0{,}5.1{,}8)} = 21{,}4 \qquad \text{cycle times}$$

Normal traffic flow rates will be achieved after 22 cycle times, it means that it is crowded for half an hour).

The results of calculation are presented in the following table:

Time interval	1	2	3	4	next	
Traffic volume [veh/15 min]	133	140	161	147	112 constant	
Degree of saturation	0,95	1,00	1,15	1,05	0,80	
Number of cycle times	1	10	20	30	40	62
Congestion N_{GE} [veh]	2,1	5,2	9,5	31,8	40,6	1,8
Basic waiting time	30,3	30,3	31,0	33,2	31,7	28,4
Rear-congestion waiting time	13,5	33,5	61,3	205,2	261,9	11,6
Average waiting time	43,8	63,8	92,3	238,4	293,6	40,0
Average waiting time in time section	53,8	78,1	165,4	266	166,7	40,0

3.3.2. Waiting Time of Buses

The waiting time of a public transport vehicle with a special or exclusive lane in case of fixed-time signal control and no bus-stop at the approach is determined as follows:

$$w = \frac{t_S}{t_U} \cdot \left(\frac{t_S}{2} + t_{az} \right) \qquad (6\text{-}20)$$

Where:

w = average waiting time of public transport vehicle

t_{az} = additional time for starting (appro. 9s by the speed of 50 km/h)

Because the green time for public transport is always sufficient, therefore the rear-congestion does not occur.

In case of no exclusive lane for public transport, the average waiting time is calculated by formula (6-19b).

3.3.3. Waiting Time of Pedestrians

The average waiting time for pedestrians, who cross one crossing or want to cross the carriageway, is determined as follows:

$$w = \frac{t_S^2}{2.t_U} \qquad (6\text{-}21)$$

Where:

w = average waiting time for pedestrians

t_S = red time

t_U = cycle time

and the maximum waiting time $w_{max} = t_S$.

The start-up lost time of pedestrians can be taken from 1 s to 2 s because pedestrians, in general, cannot go immediately when the green time begins. The average waiting time is always shorter than a half of the red time. The long red time, e.g. the long maximum waiting time, gives the relatively low average waiting time (for example, t_U = 90 s, $t_S = w_{max}$ = 80 s $\Rightarrow$ w $\approx$ 36 s).

In case of one walk to cross the approach, the determination of the waiting time is very simple according to formula (6-21). If pedestrians have to stop on the central reservation or on the islands, the waiting time depends on the green time offset of individual crossings. This waiting time is analysed in detail based on the signal program design.

3.3.4. Waiting Time of Cyclists and Motorcycle Riders

The average waiting time for cyclists at the crossing, who ride on one direction only, will be determined approximately by formula (6-22):

$$w = \frac{t_U.(1-f)^2}{2.(1-q/(s_b.3000))} \qquad (6\text{-}22)$$

Where:

w = average waiting time of one cyclist	[s]
f = green time proportion = t_F/t_U	[-]
q = traffic volume of cyclists	[cycles/h)
s_b = width of the cycle path ($s_b \geq 1$ m)	[m]

In practice, there is no rear-congestion for cycle traffic.

If the cycle traffic volume is not high, the average waiting time for cyclists can be simply determined by formula (6-21) disregarding the width of the cycle path.

In case of motorcycle traffic, it is assumed that all motorcycles arriving in every cycle time can access the head-start lanes during the red time and release during the green time. Hereby, there is no rear-congestion for motorcycle traffic. Therefore, the average waiting time of motorcycles can be calculated similarly to that of cyclists by formula (6-22).

3.4. Number of Stops

The number of vehicles that has to stop during the cycle time can be approximately determined as follows:

$$n_H = \begin{cases} n'_H & \text{for } n'_H < m \\ m & \text{for other cases} \end{cases} \qquad (6\text{-}23)$$

Where: $$n'_H = \frac{q.(t_U - t_F + N_{GE}.t_B)/3600}{1 - q/q_S}$$

m = average number of vehicles arriving during one cycle time = q. t_U / 3600 [veh]

n_H = number of stops in one cycle time [veh]

t_U = cycle time [s]

t_F = green time [s]

N_{GE} = number of vehicles congested by the end of green time [veh]

t_B = time headway [s/veh]

q = traffic volume on the relevant lane [veh/h]

q_S = saturation flow rate [veh/h]

The high degree of saturation leads to all vehicles stopping.

The total number of vehicles q_H that has to stop on one lane during one hour with fixed-time signal control is determined as follows:

$$q_H = n_H \,.\, U \qquad (6\text{-}24)$$

Where:

q_H = number of vehicles stopping per hour

n_H = number of vehicles stopping per cycle time

U = number of cycle times per hour

The proportion of stops h is:

$$h = 100.\ n_H / m = 100.\ q_H / q \qquad (6\text{-}25)$$

In case the average rear-congestion is significant, the arriving vehicles have to move up before passing the stop-line. The maximum number of vehicles moving up r is determined as follows:

$$r = \begin{cases} 0 \text{ for } r' \le 1 \\ r' \text{ for others} \end{cases} \qquad (6\text{-}26)$$

Where: $r' = \dfrac{N_{GE} + m}{n_C}$

N_{GE} = average rear-congestion [veh]

m = average number of vehicles arriving [veh]

n_C = the maximum number of vehicles passing the stop-line during the green time of the cycle time [veh].

The number of stops decides the influence on fuel consumption at traffic signals, which is additional fuel consumption due to stopping and accelerating from the stop-line comparing to through drives.

3.5. Degree of Saturation

The degree of saturation is the ratio of traffic flow arriving to capacity of the section (a lane or an approach) or of the entire intersection. It can be higher than 1.0 for an intersection (over saturation). The degree of saturation is calculated by the following relationship:

$$g = \frac{q.t_U}{q_S.t_F} = \frac{q}{f.q_S} \qquad (6\text{-}27)$$

Where:

g = degree of saturation	[-]
q = traffic volume	[veh/h]
q_S = saturation flow rate	[veh/h]
t_U = cycle time	[s]
t_F = green time	[s]
f = green time proportion = t_F / t_U	[-]

And it can be used to evaluate roughly traffic flow quality. Therefore, the following inter-relationships should be considered:

- Considerably different degrees of saturation for the relevant traffic streams of the phases imply that the green time division is improper and should be varied (checking and applying the formula:

$$t_{F1} : t_{F2} : t_{F3} : ... = \frac{q_{maßg1}}{q_{S1}} : \frac{q_{maßg2}}{q_{S2}} : \frac{q_{maßg3}}{q_{S3}} : ...)$$

Where:

t_{Fi} = green time of relevant lanes of phase i

$q_{maßgi}$ = relevant traffic volume of phase i

q_{Si} = saturation flow rate of relevant lanes in phase i

- Only when the degree of saturation of individual relevant streams of the phases is approximately equal to each other, the minimum waiting time will be achieved. The degree of saturation, which belongs to the minimum waiting time is:

$$g = \frac{2.B}{1+B} \qquad (6\text{-}28)$$

Where:

g = degree of saturation

B = sum of flow ratios of relevant traffic streams of the phases $= \sum_{i=1}^{p} b_{maßg,i} = \sum_{i=1}^{p} q_{maßg,i}/q_{Si}$

- Degree of saturation g > 1,0 (the approach is over saturated) means that motorised traffic has extremely long waiting time (quality level F). When all approaches are over saturated that usually occur in peak hours, the division of the green time must be considered so that degree of saturation of individual traffic streams of the relevant phases is approximately equal to each other.
- Generally, if degree of saturation was too small, the too long cycle time had been selected.

The average degree of saturation of an approach or an intersection is determined by an arithmetical average calculation:

$$\overline{g} = \frac{\sum_{i=1}^{k} g_i.q_i}{\sum_{i=1}^{k} q_i} \qquad (6\text{-}29a)$$

Where:

$\overline{g}$ = average degree of saturation [-]

k = number of lanes, which is relevant to the calculation [-]

g_i = degree of saturation for lane i [-]

q_i = traffic volume of lane i [veh/h]

$\sum_{i=1}^{k} q_i$ = the total traffic volume [veh/h]

As a significant criterion, it is recommended one more formula to calculate the average degree of saturation of an intersection according to the relevant phases:

$$\overline{g}_{maßg} = \frac{\sum_{i=1}^{p} g_{maßg,i}.q_{maßg,i}}{\sum_{1}^{p} q_{maßg,i}} \qquad (6\text{-}29b)$$

Where:

$\overline{g}_{maßg}$ = average degree of saturation of streams in relevant phases [-]

p = number of phases [-]

$g_{maßg,i}$ = degree of saturation of the stream of the relevant phase [-]

$q_{maßg,i}$ = traffic volume of the stream of the relevant phase [veh/h]

The average degree of saturation of the relevant phase-streams $\overline{g}_{maßg}$ is always higher than the average degree of saturation $\overline{g}$ under considerations of all lanes.

3.6. Development of Turning Traffic

If the turning drivers are led by an exclusive phase without conflicts, these streams are applied the same as go-through drivers concerning traffic engineering calculation. However, local conditions have to be taken into account (for example, turning radii) so that the saturation flow rates have to be applied in a suitable way (**Table 3.2**). For the following situations, the calculation concerning waiting time, queue length, capacity must be specially analysed.

1. Permitted right-turning drivers who have to ensure the priority to pedestrians
2. Permitted left-turning drivers who have to give priority to opposing traffic and pedestrian traffic.

3.6.1. Permitted Right-turning Traffic

The permitted right-turning drivers are phased with priority parallel pedestrians and have an obligation for waiting. In order to ensure priority to pedestrians, the time lead t_{vor} of 1 s to 2 s for pedestrians at the conflict areas have to be given, hereby pedestrians have already been on the crossing before vehicles arrive.

To calculate capacity, the green time for right-turners t_{Fu} without pedestrians is required, and it is determined as follows:

$$t_{Fu} = \max \begin{cases} t_F - t_{fuss} - N_A . t_B \\ 0 \end{cases} \qquad (6\text{-}30)$$

Where:

t_{Fu} = green time without pedestrians [s]

t_F = green time for right-turners [s]

t_{fuss} = occupancy time of pedestrians on the crossing, right-turners are not allowed to drive during this time [s]

N_A = number of stopping spaces between the stop-line and the crossing [veh]

t_B = time headway for one right turner [s/veh].

Occupancy of pedestrians on the crossing is usually given as soon as the green time begins. In case of low traffic volume of pedestrians, the occupancy time t_{fuss} of 4 s to 8 s can be taken. Depending on the number of pedestrians (P) per one cycle time, the occupancy time can be calculated according to formula (6-31):

$$t_{fuss} = \frac{P}{0{,}024.P + 0{,}48} - t_{vor} \qquad (6\text{-}31)$$

Where:

t_{fuss} = occupancy time of pedestrians [s]

P = number of pedestrians per cycle time [per]

t_{vor} = time lead for pedestrians from 1 s to 2 s [s]

The occupancy time t_{fuss} can also be determined directly from measurement.

Capacity of right-turners C_{RA} is determined as follows:

$$C_{RA} = \min \begin{cases} q_S . t_{Fu} / t_U + N_A . U \\ q_S . t_F / t_U \end{cases} \qquad (6\text{-}32)$$

Where:

C_{RA} = capacity of right-turners [veh/h]

t_U = cycle time [s]

t_F = green time for right-turners [s]

q_S = saturation flow rate [veh/h]

N_A = number of spaces for vehicles [veh]

U = number of cycle times per hour [-]

t_{Fu} = green time for right-turners without pedestrians [s]

For the calculation of the average waiting time for permitted right-turners, as the green time, green time without pedestrians plus the time for queue length of right-turners releasing (N_A . t_B) will be applied.

In case right turning during red is permitted (green arrow regulation), the additional capacity C_{RAROT} is added. Researches have already shown that under preconditions of an exclusive right-turning lane, in average, nearly half of all right-turners turn during red. With permitted right-turning movements during red, the "free" red time can be determined as the additional average green time depending on the ratio $a_{RAROT} = q_{RAROT} / q$ (6-33) as follows:

$$\Delta t_F = a_{RAROT} . t_U . q / q_S \qquad (6\text{-}34)$$

Where:

Δt_F = additional green time (free red time) [s]

a_{RAROT} = proportion of right-turners who turn during red [-]

t_U = cycle time [s]

q = traffic volume of the total right-turning streams [veh/h]

q_S = saturation flow rate of right-turning flow [veh/h]

q_{RAROT} = traffic volume of right-turning movements during red [veh/h]

This additional red time is usually added to the green time without pedestrians when calculating the average waiting time for the right-turners.

The proportion of right-tuners a_{RAROT} can be roughly determined between 20 and 70%, or better determined by a simple measurement (counting q and q_{RAROT}), too.

3.6.2. Permitted Left-turning Traffic

The capacity of permitted left-turning movements C_{LA} is composed of two parts:

- the left-turning vehicles that go across opposing traffic based on the sufficient time gaps (C_D)
- the left-turning vehicles that have already been in inner intersection areas and can pass the intersection during the phase transition (C_{PW})

Therefore, gives:

$$C_{LA} = C_D + C_{PW} \qquad (6\text{-}35)$$

Where:

C_{LA} = capacity of permitted left-turning vehicles [veh/h]

C_D = capacity of go-across vehicles [veh/h]

C_{PW} = capacity during the phase transition [veh/h]

The capacity C_D is determined as follows:

- in case of one-lane opposing traffic, C_D is approximately calculated by the following relationship:

$$C_D = \frac{3600.f - q.t_c}{t_f}.e^{-q.(t_0 - t_c)/(3600.f - q.t_c)} \qquad (6\text{-}36)$$

Where:

C_D = capacity of go-across vehicles [veh/h]

q = traffic volume of opposing traffic in the green time of left-turning movements [veh/h]

f = the green time proportion of left-turning traffic = t_F / t_U [-]

t_g = required time gap = 5,7 s [s]

t_g = successive time gap = 3,0 s [s]

t_0 = non-time gap = $t_g - t_f / 2$ = 4,2 s [s]

t_c = minimum time gap in opposing traffic = 1,8 s [s]

- in case of multi-lane opposing traffic:

$$C_D = \frac{3600.f}{t_f}.e^{-q.t_0/(3600.f)} \qquad (6\text{-}37)$$

Where:

q = total traffic volume of opposing traffic [veh/h]

The capacity of go-across vehicles depends on the opposing traffic volume and the green time proportion as presented in **Table 3.6**.

The capacity C_D is relatively decreased depending on average opposing traffic volume. Left-turning vehicles have to wait to get the sufficient time gap in opposing traffic.

Table 3.6: Capacity C_D of permitted left-turning movements driving across opposing traffic with different green time proportions f

Opposing traffic q [veh/h]	Green time proportion					
	0,15	0,20	0,25	0,30	0,35	0,40
One-lane						
100	60	115	170	230	290	350
150	25	65	120	175	230	285
200	5	30	75	125	175	230
250	-	10	40	80	130	180
300	-	-	15	45	90	135
350	-	-	5	25	55	95
400	-	-	-	10	30	65
Multi-lane						
200	40	75	120	165	215	270
250	25	55	95	135	185	230
300	15	40	75	110	155	200
350	10	30	60	90	130	175
400	10	25	45	75	110	150
450	5	15	35	65	95	130
500	5	15	30	50	80	110
550	-	10	25	40	65	95
600	-	5	20	35	55	80

Time gap boundary $t_g = 5{,}7$ s

Non-time gap $t_0 = 4{,}2$ s

Successive time gap $t_f = 3.0$ s

Minimum time gap in opposing traffic $t_c = 1{,}8$ s

Capacity is rounded by five vehicles.

The capacity C_{PW} is determined as follows:

$$C_{PW} = N_A \, . \, U \qquad (6\text{-}38)$$

Where:

C_{PW} = capacity during phase transition [veh/h]

N_A = number of stopping spaces in inner intersection areas [veh]

U = number of cycle times per hour = 3600 / t_U [-]

t_U = cycle time [s]

The waiting time of left-turning drivers can be approximately determined from formula (6-35) of capacity calculation by the following relationship:

$$C_{LA} = f.q_S = q_S \, . \, t_F / t_U \qquad (6\text{-}39)$$

Where:

C_{LA} = capacity of permitted left-turning movements [veh/h]

f = green time proportion = t_F / t_U [-]

q_S = assumed saturation flow rate of left-turning flow [veh/h]

t_F = green time [s]

t_U = cycle time [s]

From the above formula, in average, the dummy green time per cycle time that allows left-turning drivers to pass unimpeded, is determined as follows:

$$t_F = C_{LA} \, . \, t_U / q_S \qquad (6\text{-}40)$$

Then, the waiting time according to formula (6-19) will be applied. The rear-congestion (queue length) is determined by the formulas in **Table 3.5**.

The go-across left-turning drivers are usually applied only in case of small intersections and low left-turning traffic volume (app. < 180 veh/h).

3.7. Special Situations

3.7.1. Mixed Lanes

If more than one traffic stream share a lane, the saturation flow rate of mixed-flow q_{SM} is determined as follows:

$$q_{SM} = \frac{1}{\sum_{i=1}^{k} a_i / q_{Si}} \qquad (6\text{-}41)$$

Where:

q_{SM} = saturation flow rate of mixed-flow [veh/h]

a_i = traffic volume proportion of stream i on the mixed-lane [-]

q_{Si} = saturation flow rate of stream i [veh/h]

k = number of streams on the mixed-lane [-]

The capacity of the mixed-lane is, therefore, calculated as follows:

$$C_M = f. \, q_{SM} \qquad (6\text{-}42)$$

Where:

C_M = capacity of mixed-lane

q_{SM} = saturation flow rate of mixed-lane

f = green time proportion of mixed-lane

The degree of saturation is determined:

$$g_M = \frac{q_M}{C_M} = \frac{\sum_{i=1}^{k} q_i}{f.q_{SM}} \qquad (6\text{-}43)$$

Where:

g_M = saturation degree of the mixed-lane [-]

q_M = total traffic volume on the mixed-lane [veh/h]

q_i = traffic volume of stream i [veh/h]

k = number of streams, which share the mixed-lane

3.7.2. Short Lanes at Intersections

Because of the limitation of road space at intersection, a defined traffic direction often uses only a short lane. This short lane cannot achieve its full capacity if the discharged flow is higher than the available stopping spaces N_K on the short lane (t_F / t_B > N_K). Its capacity, therefore, depends much on how convenient the drivers can change from the share lane into individual lanes. The situations of short lanes concerning their use for individual traffic streams are described in **Figure 3.2**.

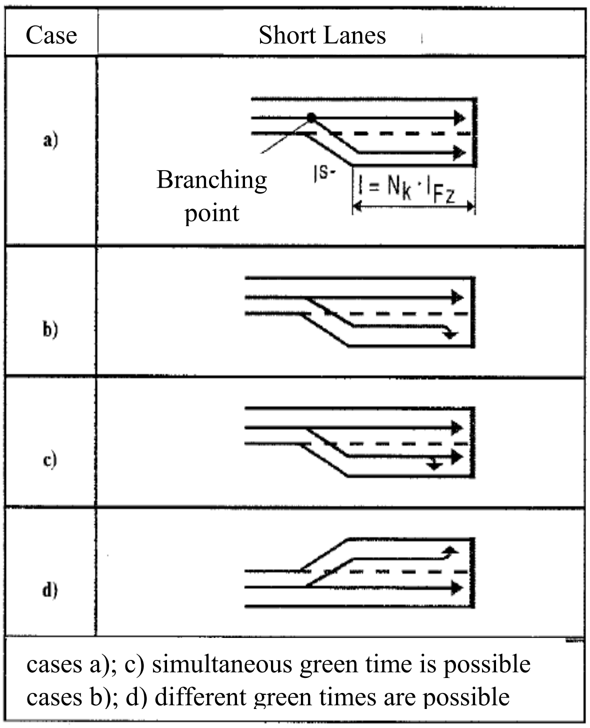


Figure 3.2: Principle descriptions for applying short lanes

For the calculations, the following symbols are used:

N_K = number of stopping spaces on the short lane length $l = l / l_{Fz}$ [-]

l_{Fz} = length of a vehicle = 6 m [m]

q_1 and q_2 = traffic volume of lane 1 and short lane 2 [veh/h]

C_1 and C_2 = capacity of lane 1 and short lane 2 [veh/h]

q_{S1} and q_{S2} = saturation flow rate of lane 1 and short lane 2 [veh/h]

U = number of cycle time per hour [-]

t_U = cycle time [s]

t_F = green time [s]

f = green time proportion = t_F / t_U [-]

Approximately, capacity of the go-through lane is calculated by f. q_{S1}, and capacity of the short lane is calculated by N_K. U. Hereby, the total capacity of the approach C is calculated as follows:

- in case of simultaneous green time for both lane:

$$C = C_1 + C_2 = f \,.\, q_{S1} + N_K \,.\, U \qquad (6\text{-}44)$$

Formula (6-44) above is valid in case of $q_1 > q_2$. In contrary case $q_2 > q_1$, formula (6-44) is calculated with f . q_{S2}. For the permitted left-turning traffic (go-across), it must be still checked with formula (6-35) whether the capacity N_K . U can be used.

- in cases both lanes do not receive the green time simultaneously

$$C = C_1 + C_2 = 2 \,.\, N_K \,.\, U \qquad (6\text{-}45)$$

For checking in practice, and/or safety of the calculated results, it is recommended that on such an approach of full green time, number of vehicles q_1 and q_2 discharging from both lanes can be counted.

3.7.3. Traffic Division on Lanes at Approaches

Within the framework of signal program calculation, traffic volume of individual lanes must be available to determine traffic volume of relevant phases. At the best, when collecting traffic data at existing intersections, this value is counted directly for every direction of individual lanes. Only when these traffic stream values are counted for whole lanes, then the traffic division on lanes is approximately carried out.

The vehicles on an approach, which are operated in the phase will be divided into individual lanes so that they have to wait approximately for the same period of time (the same waiting time).

This means that the flow ratios on individual lanes are set approximately as equal as each other:

$$\frac{q_1}{q_{S1}} : \frac{q_2}{q_{S2}} : \ldots = \text{const} \qquad (6\text{-}46)$$

Where:

q_i = traffic volume of lane i [veh/h]

q_{Si} = saturation flow of lane i [veh/h]

Therefore, it allows to determine traffic volume on individual lanes from the given traffic volume values.

Figure 3.3 describes a situation in which the traffic division on both lanes can be determined by formula (6-47) and (6-48).

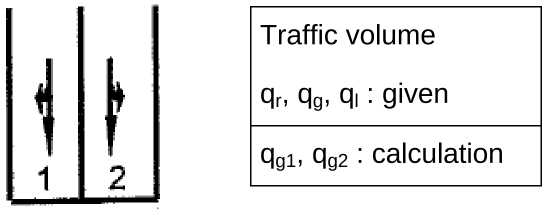


Figure 3.3: Layout of situation for dividing traffic on lanes

From the data and figure above, gives: $q_g = q_{g1} + q_{g2}$; $q_1 = q_r + q_{g1}$; $q_2 = q_l + q_{g2}$.

Because $q_1 / q_{S1} = q_2 / q_{S2}$, therefore:

$$\frac{q_r + q_{g1}}{q_{S1}} = \frac{q_l + q_g - q_{g1}}{q_{S2}} \qquad (6\text{-}47a)$$

From formula (6-47a), gives:

$$q_{g1} = \frac{(q_g + q_l).q_{S1} - q_r.q_{S2}}{q_{S1} + q_{S2}} \qquad (6\text{-}47b)$$

and

$$q_{g2} = q_g - q_{g1} \qquad (6\text{-}48)$$

At first, formula (6-47b) and (6-48) are calculated with $q_{S1} = (q_{Sg} + q_{Sr})/2$ and $q_{S2} = (q_{Sg} + q_{Sl})/2$. Then, saturation flow rate of the mixed lane will be determined according to formula (6-41) as follows:

$$q_{SM} = \frac{1}{\sum_i a_i / q_{Si}}$$

Where:

q_{SM} = saturation flow rate of the mixed lane [veh/h]

a_i = traffic proportion of stream i [-]

q_{Si} = saturation flow of stream i [veh/h]

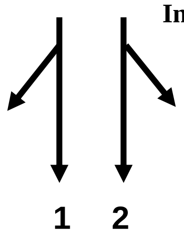
Then, the results of q_{SM} will be applied to formula (6-47b) and (6-48) to calculate the new value of q_{g1} and q_{g2}. If the new value of q_{g1} and q_{g2} are not approximately equal to the old values, these new values can be taken for the second iteration. Doing iterations until the correct results are derived.

Here is an example for calculation:

Determining the traffic division on individual lanes:

1. **Example of calculation 1**

Give: only the traffic volumes as well as respective saturation flows



Input data:

q_g = 800 veh/h; q_{Sg} = 2000 veh/h

q_r = 120 veh/h; q_{sr} = 1500 veh/h

q_l = 120 veh/h;q_{Sl} = 1700 veh/h

Calculation

q_{S1} = (2000+1500)/2 = 1750 veh/h

q_{S2} = (2000+1700)/2 = 1850 veh/h

$$q_{g1} = \frac{q_{S1}.(q_g + q_l) - q_{S2}.q_r}{q_{S1} + q_{S2}} = \frac{1750.920 - 1850.120}{1750 + 1850} = 386$$

veh/h

q_{g1} = 386 veh/h

q_{g2} = 414 veh/h

New calculation of q_{S1} and q_{S2}

q_{S1} = 1/(0,763/2000+0,237/1500) = 1854 ≈ 1855 veh/h

q_{s2} = 1925 veh/h

$$q_{g1} = \frac{1855.920 - 1925.120}{3780} = 390 \quad \text{veh/h}$$

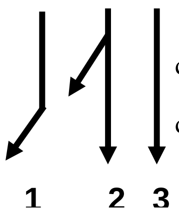
q_{g2} = 410 veh/h

q_1 = 510 veh/h

q_2 = 530 vhe/h

Dividing equally traffic volume, traffic volume of q_1 = q_2 = 520 veh/h is derived. The next examples show that the calculation methodology can be useful for complex ratios.

2. **Example of calculation 2**



Input data:

q_g = 800 veh/h; q_{S2} = 1800 veh/h

q_r = 500 veh/h; q_{sr} = 1600 veh/h

; q_{S3} = 2000 veh/h

Because only go-straight vehicles are used on the third lane, hereby saturation flow rate is set at q_{S3} = 2000 veh/h.

Calculation formulas:

$$q_{g2} = \frac{q_g.(q_{S1} + q_{S2}) - q_r.q_{S3}}{q_{S1} + q_{S2} + q_{S3}}$$;$q_{g1} = q_g - q_{g2}$

$q_{r1} = q_{g3} . q_{S1} / q_{S3}$; $q_{r2} = q_r - q_{r1}$

Calculation

q_{S2} = (1600+1800)/2 = 1700 veh/h

$$q_{g2} = \frac{800.(1600 + 1700) - 500.2000}{5300} = 309$$

veh/h; q_{g3} = 491 veh/h

q_{r1} = 491.1600/2000 = 393 veh/h ; q_{r2} = 107 veh/h

New saturation flow rate q_{S2} = 1745 veh/h

q_{g2} = 314 veh/h ; q_{g3} = 486 veh/h

q_{r1} = 389 veh/h; q_{r2} = 111 veh/h

q_1 = 389 veh/h; q_2 = 425 veh/h; q_3 = 486 veh/h

Check again: q_1 / q_{S1} = 389 / 1600 = 0,243

q_2 / q_{S2} = 425 / 1745 = 0,244

q_3 / q_{S3} = 486 / 2000 = 0,243

Remarks: The mixed lane is not used in case of low traffic volume on right-turning movements because the waiting time rates on the right-turning lane are then too long.

3.8. Dimension of Queue Length

Dimension of queue length on the turning or go-through lanes is called as the maximal queue length that is shortly performed after the end of the red time. Because the number of vehicles in the queue length is a random variable and varies from cycle time to cycle time, a certain guarantee against congestion must be provided.

Dimension of the queue length on the approaches is implemented for the relevant peak hours. However, the examination of queue length in normal hours should also be implemented. Concerning cases of the queue length, it has to be distinguished that:

Turning vehicles are not allowed to queue up on the neighbouring lanes. Otherwise, they will block the access of these lanes.

Turning lanes are blocked by the vehicles on the neighbouring lanes, because the left-turners cannot entrance to their left-turning lanes any more.

In normal hours of traffic, a smooth traffic process should be given for case a) and b). On the contrary, in peak hours, it may be sufficient if the turners have enough spaces for stopping (only case a).

In regular case, a statistical guarantee of 90% against congestion is applied (90%-queue length at the end of the red time). A higher guarantee has a corresponding longer queue length, but in practice, it is not realizable.

An overloaded lane on the approach shows a permanently growing queue length during the overload status. Such cases of traffic volume often occur in practice during peak hours, a dimension of queue length, therefore, is impossible. Thus, dimensioning the maximum queue length should be reckoned with a degree of saturation of g = 0.95. Actually, it will achieve in individual cases of investigation. As the dimension of queue length, the determination of the average rear-congestion shall be guided with a corresponsively decreased traffic volume.

Because the lane in construction is often insufficient for the queue length, the lower guarantee against congestion must be applied. In **Table 3.7**, the number of cycle times congested in a peak hour is given depending on different guarantees, in which the number of long cycle time is generally showed.

The number of vehicles congested at the end of red time depending on the statistical guarantee S can be simply determined by the following relationship:

$$N_{RE.S} = \left(e^{0,022.(S-50)} - 1\right)\sqrt{m_R + N_{GE}} + (m_R + N_{GE})$$

for S = 50…95% (6-49)

Where:

$N_{RE.S}$ = number of vehicles congested by the end of the red time [veh]

S = guarantee against congestion [%]

m_R = average arrivals by the red time = $q.t_S$ / 3600 [veh]

q = traffic volume on the relevant lane [veh]

t_S = the red time [s]

N_{GE} = number of vehicles congested by the end of the green time (determined according to **Table 3.5**) [veh].

Formula (6-49) in **Figure 3.4** is used to dimension the queue length in practice

Table 3.7: Number of cycle times not congested (a) and congested (b) during an hour with a certain guarantee against congestion.

Cycle time [s]	72	80	90	100	120
Number of cycle times per hour	50	45	40	36	30
Guarantee S against congestion	a) number of cycle times not congested per hour*				
95%	47,5	42,8	38,0	34,2	28,5
90%	45,0	40,5	36,0	32,4	27,0
85%	42,5	38,3	34,0	30,6	25,5
80%	40,0	36,0	32,0	28,8	24,0
	b) number of cycle time congested per hour*				
95%	2,5	2,2	2,0	1,8	1,5
90%	5,0	4,5	4,0	3,6	3,0
85%	7,5	6,7	6,0	5,4	4,5
80%	10,0	9,0	8,0	7,2	6,0

* Value not rounded

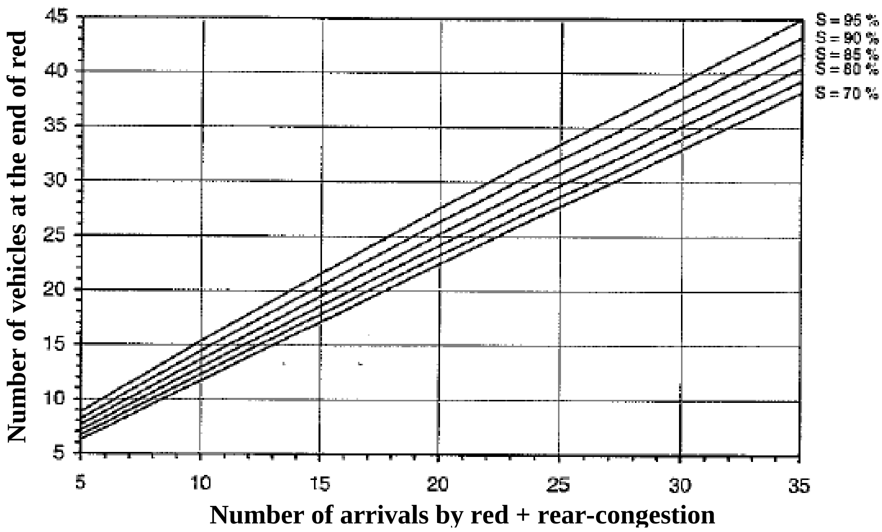


Figure 3.4: number of vehicles congested at the end of red time depending on the number of arrivals

A double-lane on the approach for a certain direction must be worked out if the lane of the free section of this direction is arranged (**Figure 3.5d**), the length of the additional lane shall be designed by the maximal number of vehicles discharged $n_C = t_F / t_B$ (formula (6-2), too). Therefore, it is ensured that the full capacity of the additional lanes can be also used (capacity of the short lane in section 5).

The basic principles for dimensioning the queue length are summarized in **Figure 3.5**.

The necessary length of the lane can be calculated as follows:

$$L = N.\ l_{Fz} \qquad (6\text{-}50)$$

Where:

L = the necessary length of lane [m]

l_{Fz} = the length of vehicle (6m) [m]

N = number of vehicles congested at the end of the red time according to formula (6-49) or the maximal number of vehicles releasing n_C according to formula (6-2) [veh]

The queue length dimension is determined on the basis of traffic volume per hour [veh/h]. The length of a vehicle of 6 m is applied:

1. In peak hours, the turners should not queue up on the neighbouring lanes (basic lanes) (ensure case b or eliminate case a). A guarantee against congestion of 90% should be provided (S = 90%), in which a maximal degree of saturation of g = 0,95 for turning traffic shall be taken.
2. In normal hours of traffic the arrivals on the turning lanes should be guaranteed (case c), it means that the turning lanes should not be blocked by the vehicles of the basic lanes. The guarantee against congestion of S = 95% should be applied.
3. With more turning lanes, the same traffic load on individual lanes is assumed if there is no other indication available.
4. In cases of limitation of spaces for queue length, a lower guarantee against congestion must be accepted. Verification for the influences on traffic process shall be carried out (indication of sufficient guarantee against congestion).
5. With a double-lane (case d), the length of the additional lane is designed based on the maximal number of vehicles that can discharge during the green time.

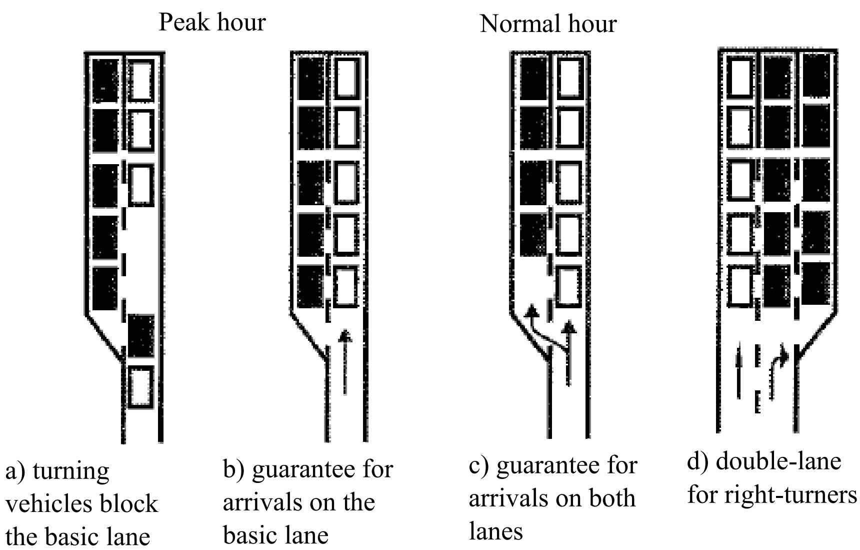


Figure 3.5: Instruction for the queue length dimension.

3.9. Capacity of Intersection

The capacity of a signalized intersection C_K is equal to the sum of capacity of individual lanes C_i:

$$C_K = \sum_{i=1}^{n} C_i \qquad (6\text{-}51)$$

Where:

C_K = total capacity of the signalised intersection [veh/h]

n = number of lanes [-]

C_i = capacity of lane i [veh/h]

The capacity of a lane C_i, whose traffic is released unimpeded in a phase, can be calculated according to formula (6-8).

$$C_i = f_i \cdot q_{Si}$$

Where :

C_i = capacity of lane i [veh]

f_i = green time proportion [-]

q_{Si} = saturation flow rate [veh/h]

Furthermore, the following cases should be considered when calculating capacity:

- Left turners across opposing traffic (applying formula (6-35)). If the green time is fully occupied by go-through traffic, the capacity of left-turning traffic across opposing traffic C_D is zero. There is only capacity of left-turning traffic C_{PW} during the phase transition.
- Permitted right-turners given simultaneously green time with pedestrian traffic (applying formula (6-32))
- Right-turners during red: beside normal right-turning capacity according to formula (6-32), the additional capacity C_{RAROT} should be taken into account.
- Shorting turning lane (applying formula (6-44) or (6-45))
- Mixed lane (applying formula (6-42))

Capacity at bottleneck that can be varied during the times of day is mostly given by a certain relationship on individual lanes. Therefore, it is sufficient to consider only the subset of lane capacities.

3.10. Dimension of Signalised Intersection

To develop a proposal for constructing an signalised intersection, the required number of lanes must be determined based on the traffic volume so that a good quality of traffic flows is guaranteed. Therefore, the sum of available traffic volume $q_{K\ vorh}$ for individual phases is not exceeded the permitted critical traffic volume $q_{K\ zul}$:

$$q_{K\ vorh} \leq q_{K\ zul} \qquad (6\text{-}52)$$

Where:

$q_{K\ vorh}$ = the sum of relevant traffic volume available [veh/h]

$q_{K\ zul}$ = permitted critical traffic volume [veh/h]

The permitted critical traffic volume can be determined by the average saturation rate $\bar{q}_S$ and the average permitted degree of saturation $\bar{q}_{zul}$ as follows:

$$q_{Kzul} = \bar{g}_{zul} . \bar{q}_S .(1 - T_Z / t_U) \qquad (6\text{-}53)$$

Where:

$q_{K\ zul}$ = permitted critical traffic volume [veh/h]

$\bar{q}_S$ = average saturation flow rate [veh/h]

$\bar{q}_{zul}$ = average permitted degree of saturation [-]

t_U = cycle time [s]

T_Z = sum of intergreen times [s]

Therefore, it can be calculated with a degree of saturation $\bar{q}_{zul}$ = 0,90 (exceptional case 0,95). The permitted critical traffic volume $q_{K\ zul}$ lies in a range of from 1100 to 1300 vehicles.

The maximal critical traffic volume $q_{K\ max}$ is determined with $\bar{g}$ = 1 as follows:

$$q_{K\max} = \bar{q}_S .(1 - T_Z / t_U) \qquad (6\text{-}54)$$

Where: $q_{K\ max}$ = maximal traffic volume [veh/h]

In practice, dimensioning an signalised intersection has to be proceeded so that a proposal for constructing should be developed and verified under which ratio between $q_{K\ vorh}$ and $q_{K\ max}$ and q_{Kzul}. If $q_{K\ vorh} > q_{K\ zul}$, the number of lanes on the approach must be increased. In contrary case, the number of lanes is reduced. **Figure 3.6** describes how to dimension a signalised intersection.

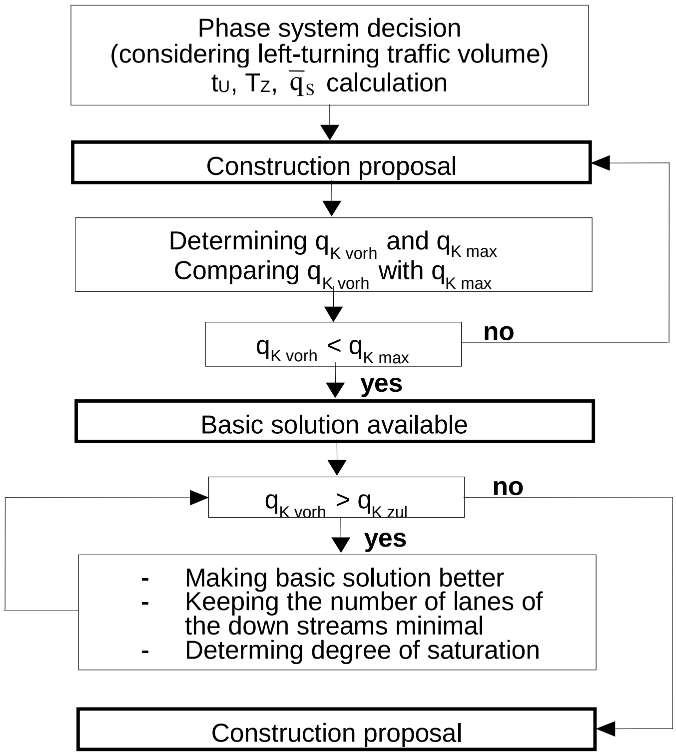


Figure 3.6: Development of proposal for constructing a signalised intersection

Annex B: Results of Empirical Studies in Hanoi

1. General remarks

This investigation is implemented in order to check the results, which were theoretically proposed in this study. The following table summarizes the parameters that need to be investigated.

Table 1: Parameters to be investigated

No.	Parameter	Value in theory	Investigation	
			Yes	No
1	**Saturation flow** - Counting traffic volume on approaches. - Checking the MC homogeneous saturation flow - Determining the adjustment factor f under the mixed traffic condition - Determining the adjustment factor f under the proposal intersection layouts	$q_S = f \cdot \frac{q^{car}+q^{mc}}{\frac{q^{mc}}{q_S^{mc}}+\frac{q^{car}}{q_S^{car}}}$ $q_S^{mc} = 11.000$ MCU/h/3.5 m $q_S^{car} =$ according to HBS	 X	X X X
2	**Cycle time** - Calculating the cycle time according to the peak and off-peak hours.	$C_o = \frac{1.5\sum_i t_Z + 5}{1-\sum \frac{1}{f}\cdot\left(\frac{q^{mc}_{FS,\text{maßg}}}{q^{mc}_S}+\frac{q^{car}_{FS,\text{maßg}}}{q^{car}_S}\right)}$ $t_{U,erf} = \frac{\sum_i t_{Z,erf}}{1-\sum_i \frac{1}{f}\cdot\left(\frac{q^{mc}_{FS,\text{maßg}}}{q^{mc}_{zul}}+\frac{q^{car}_{FS,\text{maßg}}}{q^{car}_{zul}}\right)}$		X
3	**Green time**	$t_{F,\text{maßg},i} = \frac{1}{f_i}\cdot\left(\frac{q^{mc}_{FS,\text{maßg},i}}{q^{mc}_{zul,i}}+\frac{q^{car}_{FS,\text{maßg},i}}{q^{car}_{zul,i}}\right).t_{U,erf}$		X
4	**Transition time** - Surveying the amber time on approach	3 s for V_{zul} = 50 km/h 4 s for V_{zul} = 60 km/h, and 70 km/h	X	 X
5	**Intergreen time** - Surveying the crossing time $t_{ü}$ - Surveying the clearing speeds v_r - Surveying the entering speeds v_e	3 s for go-through vehicles 2 s for turning vehicles 8 m/s for go-through vehicles 5 m/s for turning vehicles 40 km/h (not necessary)	X X X X	 X
6	**Reaction time t_{Re}**	1s (confirmed by the Vietnamese standard for road design)		X

However, to verify all the results of this study, the new layout of the intersection has to be designed and implemented according to chapter 4 of this study. This requires a legal permission and takes a long period of time. Therefore, in the scale of this study, only some of those results marked in Table 1 are investigated under the available mixed traffic condition in Hanoi.

2. Selecting the location of intersection

The Daewoo intersection in Hanoi is selected for the investigation. This intersection is considered to be the most modern signalised intersection in Hanoi, which is being controlled by fixed-time signal programs with three phases under the mixed traffic condition. The location of this intersection is indicated in the following general map:

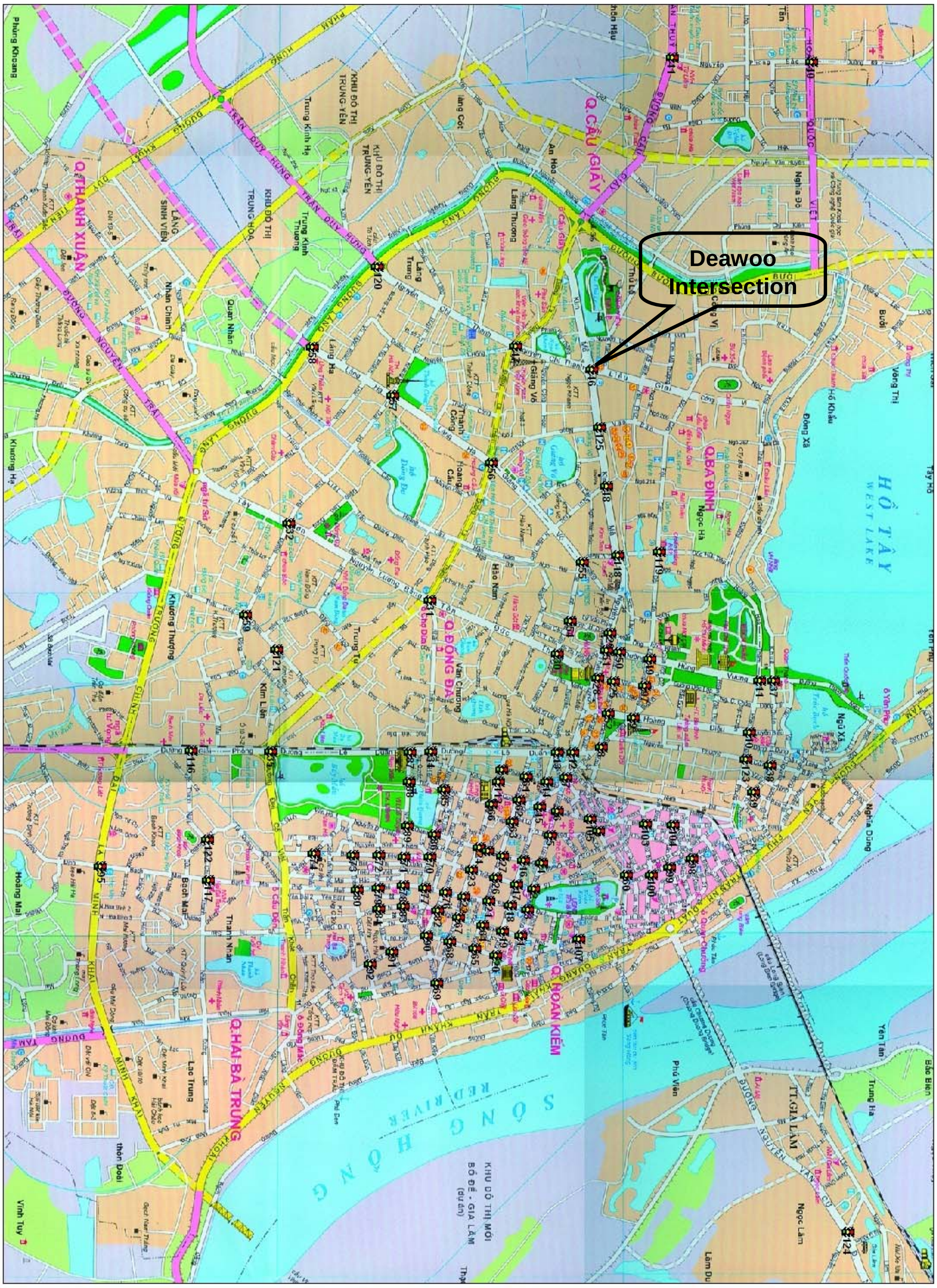


Figure 1: The location of the investigated intersection

3. Existing geometries and signal phasing of the intersection

3.1. Existing geometries

The existing geometries of the intersection are surveyed by the electric theodolite. Then, from the data collected by the theodolite, the geometries of the intersection are drawn as follows:

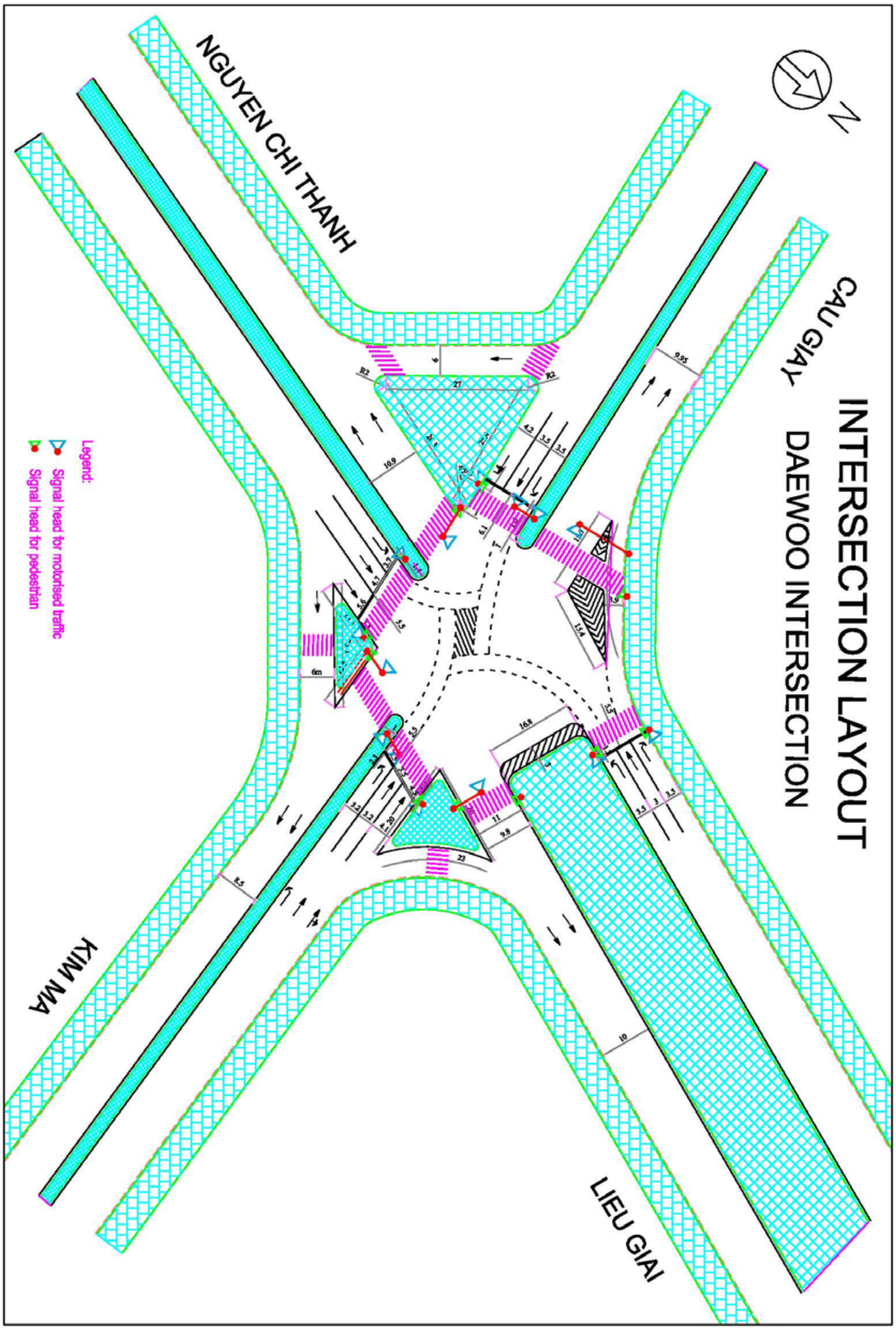


Figure 2: The existing intersection layout

3.2. Signal phasing

This intersection is being controlled with fixed-time signal programs in which the cycle time t_U is 133 s in peak hours, and 98 in off-peak hours.

With t_U = 133 s, the signal timings at Cau Giay approach are set as: t_R = 77 s; t_F = 53 s; t_G = 3s.

With t_U = 98 s, the signal timings at Cau Giay approach are set as: t_R = 64 s; t_F = 31 s; t_G = 3s.

Figure 3 shows the signal phasing at this intersection as follows:

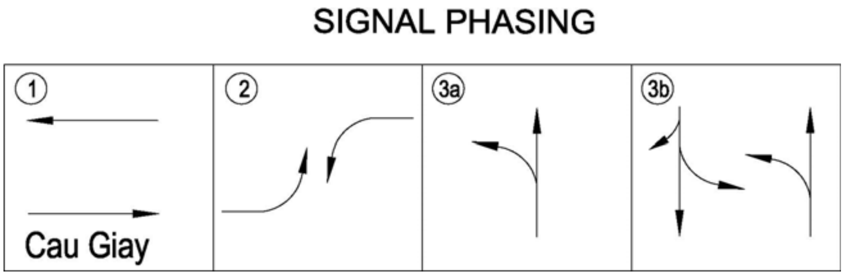


Figure 3: Signal phasing at Daewoo intersection

4. Methodology and results of the investigation

4.1. Crossing time $t_{ü}$

As discussed above, the crossing time is the interval since the green time ends until the last vehicle of the current phase reaches the stop-line.

This investigation focuses measuring the crossing time of the go-through and left-turning vehicles on Cau Giay approach. To do so, two cameras are located in front of the signal heads on Cau Giay approach as illustrated in **Figure 4** as follows:

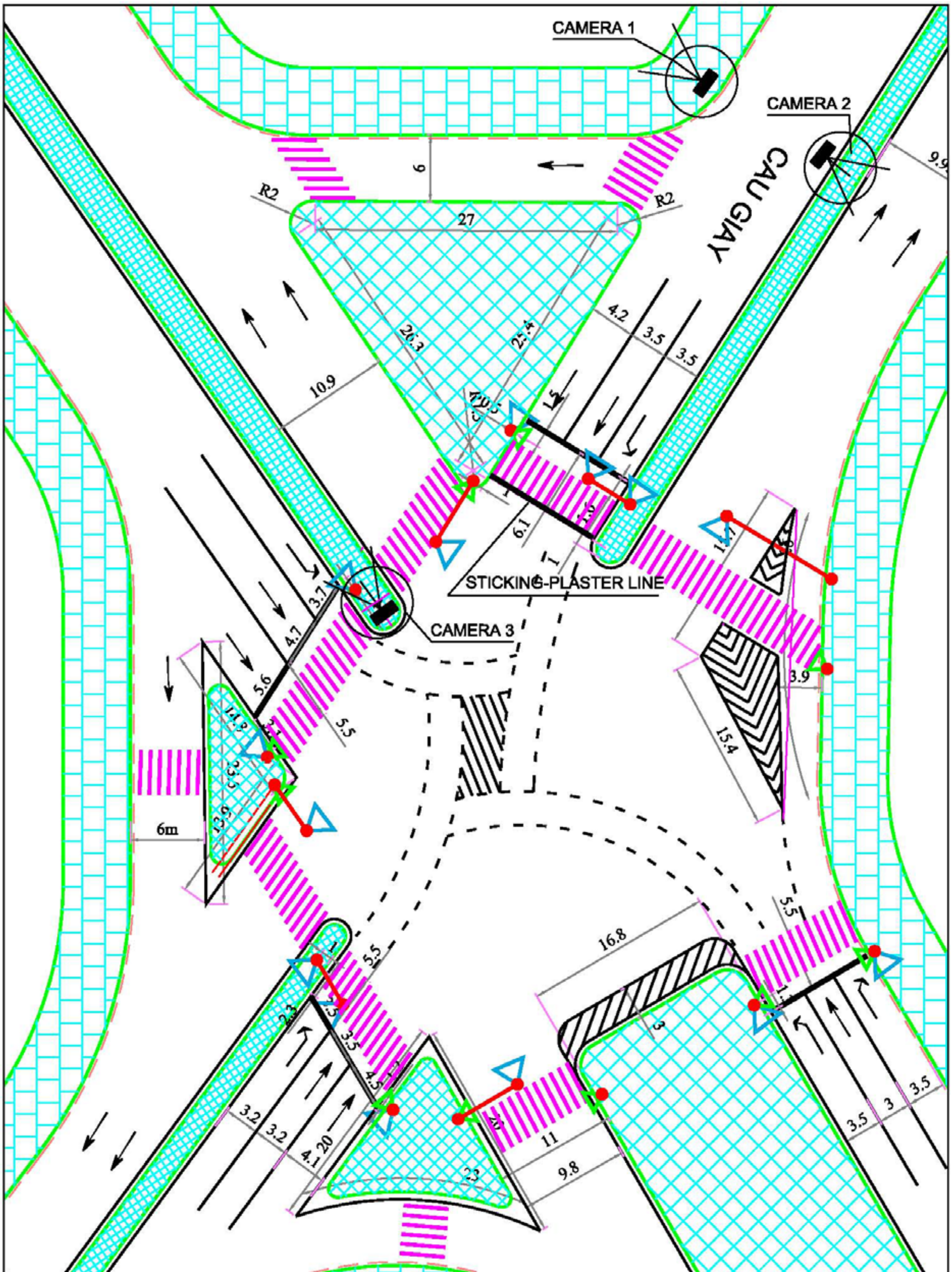


Figure 4: Camera arrangements for the investigation

Camera 1 is used to record movements of go-through vehicles and the signals showing for these vehicles. Similarly, Camera 2 is used to record the signals and movements for the left-turning vehicles.

Then, images from camera are transferred to computers. All of the measurements are implemented based on these images. By using the stop-watch with a degree of accuracy of 0.01 second, the maximal crossing time $t_{ü}$ in each cycle time is measured as the interval between twice presses on the button of the stop-watch (the first press is at the end of the green time, the second press is when the last vehicle crossing the stop-line).

The results of the maximal crossing time measurements are presented in detail in the next pages of this report. It can be summarized as follows:

For the left-turning vehicles, 35 values are recorded in which the maximal value is 2.11 s, the minimal value is 1.26 s, and the average value is 1.79 s with the standard deviation is 0.19 s. Therefore, it is reasonable to set the crossing time $t_{ü}$ at 2.0 s for left-turning vehicles.

For the go-through vehicles, 35 values are also recorded in which the maximal value is 3.0 s, the minimal value is 1.38 s, and the average value is 2.51 s with the standard deviation is 0.47 s. Therefore, it is reasonable to set the crossing time $t_{ü}$ at 3.0 s for go-through vehicles.

4.2. Clearing speed v_r

In order to measure the clearing speed of the last vehicle of the current phase, Camera 3 is located behind the signal heads of Cau Giay approach as illustrated in **Figure 4**.

A sticking-plaster line is temporarily pasted in the inner intersection area and behind the stop-line. The position of the sticking-plaster line is coincided with the further edge of the pedestrian crossing in order to easily recognize the vehicle reaching the sticking-plaster line (see **Figure 4**). The distance from the stop-line to the sticking-plaster is 6.1 m. Camera 3 records all movements of vehicles from the stop-line to the sticking-plaster line. The time interval for the last vehicle of the signal phase driving from the stop-line to the stick-plaster line is recorded by a stop-watch with a degree of accuracy of 0.01 second. Then, the value of the clearing speed is calculated based on the time interval and the distance of the vehicle's movement from the stop-line to the sticking-plaster line.

For the clearing speed of the go-through vehicles, the curve of clearing speed v_r and probability for ($v \leq v_r$) shows that the probability for ($v \leq v_r$) is quickly increased with the values of v_r from 2 m/s to 7.8 m/s, but quickly decreased with the values of v_r from 7.8 m/s to 12 m/s. Therefore, v_r = 8 m/s (probability for $v \leq v_r$ is approximately 85%) is considered to be the clearing speed in this investigation (this point on the curve is called the bend-point). This value is 2 m/s lower than that in Germany. This is reasonable due to the investigation is carried out under the mixed traffic condition, and the speed limit of motorcycle is lower than that in Germany. Therefore, it is reasonable to set v_r at 8.0 m/s for go-through vehicles.

For the clearing speed of the left-turning vehicles, the bend-point is at v_r = 5.2 m/s and probability for ($v \leq v_r$) at this point is approximately 85%. Therefore, it is reasonable to set v_r at 5.0 m/s for turning vehicles.

These results are shown in the next pages.

4.3. Amber time

At this intersection, all phases are being controlled with the amber time of 3 s. According to the observation at Cau Giay approach, there is no violation of the drivers at the beginning of the red time. In addition, under the mixed traffic condition, the speed of vehicles approaching the intersection is usually lower than the speed limit, especially in case of high traffic volume. Therefore, drivers usually stop safely in front of the stop-line. It means that, the amber time of 3 s. is reasonable.

4.4. Homogeneous motorcycle saturation flow

The goal of this investigation for the homogeneous motorcycle saturation flow is to check the value of 11.000 MCU/h/3.5 m, which was researched by Hien Nguyen and Frank Montgomery (2007).

Camera 3 records traffic flows on Cau Giay approach in peak hours. Then, the homogeneous motorcycle saturation flow of go-through traffic on the lane width of 3.5 m is analysed according to the images recorded.

As traffic is being operated under the mixed traffic condition, only periods of the green time during which only motorcycles passing the stop-line are recorded. In order to make sure that the traffic flow is saturated, only those periods of green time with full of motorcycles on the lane are analysed. Such situations of traffic occur very often because of a high proportion of motorcycles in the traffic flow. The number of motorcycles passing the stop-line is counted, and the respective periods of green time are also recorded by a stop-watch with a degree of accuracy of 0.01 second.

According to the HBS (FGSV, 2001), at least 20 values of the saturation flow have to be counted, then the average value will be calculated. This investigation collected 43 values of the saturation flow, and the average saturation flow is 10.960 MCU/h/3.5m. It means that the value of 11.000 MCU/h/3.5m for the homogeneous saturation flow proposed by Hien Nguyen and Frank Montgomery (2007) is acceptable.

All the detailed results are shown in the next pages.

Investigation Results for The Crossing Time

Intersection: Daewoo Intersection
Direction: Cau Giay -> Kim Ma **(go-through)**
Signal program: t_U = 98s ; t_R = 64s ; t_F = 31s ; t_G = 3s
Date: 19.12.2008
Time: 7:45 -> 8:45

The average value of $t_{ü}$: **2.51s**
Standard deviation: **0.47s**
The maximal value of $t_{ü}$: **3.00s**
The minimal value of $t_{ü}$: **1.38s**

Cycle time No.	Amber time (s)	Crossing time (s)	Crossing vehicle
1	3.00	2.63	Motorcycle
2	3.00	2.82	Motorcycle
3	3.00	2.98	Motorcycle
4	3.00	1.44	Bus
5	3.00	2.87	Motorcycle
6	3.00	2.99	Motorcycle
7	3.00	2.85	Motorcycle
8	3.00	2.57	Passenger Car
9	3.00	2.50	Motorcycle
10	3.00	2.96	Motorcycle
11	3.00	2.55	Motorcycle
12	3.00	2.47	Motorcycle
13	3.00	2.99	Motorcycle
14	3.00	2.34	Motorcycle
15	3.00	2.09	Motorcycle
16	3.00	2.07	Motorcycle
17	3.00	1.97	Motorcycle
18	3.00	1.75	Bus
19	3.00	2.25	Motorcycle
20	3.00	2.94	Motorcycle
21	3.00	2.95	Motorcycle
22	3.00	3.00	Passenger Car
23	3.00	2.99	Motorcycle
24	3.00	2.92	Motorcycle
25	3.00	1.75	Motorcycle
26	3.00	2.95	Motorcycle
27	3.00	2.34	Motorcycle
28	3.00	2.37	Motorcycle
29	3.00	2.53	Motorcycle
30	3.00	2.65	Motorcycle
31	3.00	2.97	Motorcycle
32	3.00	1.38	Motorcycle
33	3.00	1.97	Motorcycle
34	3.00	2.13	Motorcycle
35	3.00	2.97	Motorcycle

Investigation Results for The Crossing Time

Intersection: Daewoo Intersection
Direction: Cau Giay -> Lieu Giai **(left-turning)**
Signal program: t_U = 98s ; t_R = 64s ; t_F = 31s ; t_G = 3s
Date: 20.12.2008
Time: 7:45 -> 8:45

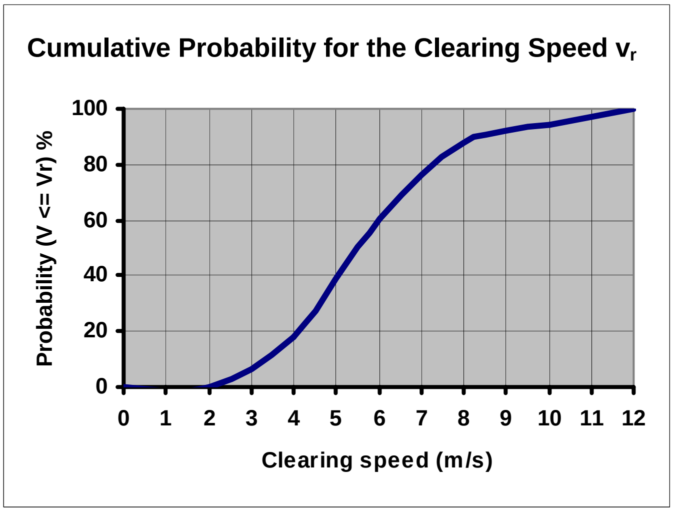
The average value of $t_{ü}$: **1.79s**
Standard deviation: **0.19s**
The maximal value of $t_{ü}$: **2.11s**
The minimal value of $t_{ü}$: **1.26s**

Cycle time No.	Amber time (s)	Crossing time (s)	Crossing vehicle
1	3.00	1.83	Motorcycle
2	3.00	2.02	Motorcycle
3	3.00	1.74	Motorcycle
4	3.00	1.98	Motorcycle
5	3.00	2.07	Motorcycle
6	3.00	1.82	Motorcycle
7	3.00	1.89	Motorcycle
8	3.00	1.38	Passenger Car
9	3.00	1.79	Motorcycle
10	3.00	1.88	Motorcycle
11	3.00	2.05	Motorcycle
12	3.00	1.67	Motorcycle
13	3.00	1.93	Motorcycle
14	3.00	2.11	Motorcycle
15	3.00	1.55	Motorcycle
16	3.00	1.89	Motorcycle
17	3.00	1.99	Motorcycle
18	3.00	1.67	Passenger Car
19	3.00	1.78	Motorcycle
20	3.00	1.94	Motorcycle
21	3.00	1.87	Motorcycle
22	3.00	1.92	Motorcycle
23	3.00	1.63	Motorcycle
24	3.00	1.86	Motorcycle
25	3.00	1.75	Motorcycle
26	3.00	1.88	Motorcycle
27	3.00	1.66	Motorcycle
28	3.00	1.89	Motorcycle
29	3.00	1.75	Motorcycle
30	3.00	1.26	Passenger Car
31	3.00	1.69	Motorcycle
32	3.00	1.48	Motorcycle
33	3.00	1.68	Motorcycle
34	3.00	1.45	Passenger Car
35	3.00	1.91	Motorcycle

Investigation Results for the Clearing Speed of go-through vehicles

Cycle time No.	Distance (m)	Time (s)	Clearing vehicle	Clearing Speed (m/s)
1	6.10	1.03	Motorcycle	5.92
2	6.10	1.44	Motorcycle	4.24
3	6.10	2.02	Bus	3.02
4	6.10	1.85	Bicycle	3.30
5	6.10	1.10	Motorcycle	5.55
6	6.10	1.93	Motorcycle	3.16
7	6.10	2.00	Motorcycle	3.05
8	6.10	1.40	Motorcycle	4.36
9	6.10	1.20	Motorcycle	5.08
10	6.10	1.20	Bus	5.08
11	6.10	1.09	Motorcycle	5.60
12	6.10	0.94	Motorcycle	6.49
13	6.10	0.65	Motorcycle	9.38
14	6.10	0.80	Motorcycle	7.63
15	6.10	1.03	Motorcycle	5.92
16	6.10	0.85	Motorcycle	7.18
17	6.10	1.60	Passenger Car	3.81
18	6.10	1.00	Motorcycle	6.10
19	6.10	0.95	Motorcycle	6.42
20	6.10	0.59	Motorcycle	10.34
21	6.10	0.97	Motorcycle	6.29
22	6.10	1.03	Motorcycle	5.92
23	6.10	0.85	Motorcycle	7.18
24	6.10	1.15	Motorcycle	5.30
25	6.10	1.20	Motorcycle	5.08
26	6.10	0.70	Motorcycle	8.71
27	6.10	1.05	Motorcycle	5.81
28	6.10	0.82	Motorcycle	7.44
29	6.10	1.70	Motorcycle	3.59
30	6.10	1.02	Motorcycle	5.98
31	6.10	0.95	Motorcycle	6.42
32	6.10	1.25	Passenger Car	4.88
33	6.10	0.53	Motorcycle	11.51

Intersection:	Deawoo Intersection
Direction:	Cau Giay -> Kim Ma **(go-through)**
Signal program:	t_U = 98s ; t_R = 64s ; t_F = 31s ; t_G = 3s
Date:	24.12.2008
Time:	7:45 -> 8:45
The 85th percentile clearing speed:	**7.8 m/s**

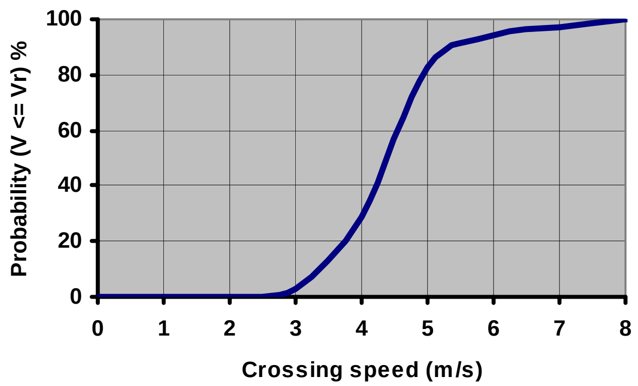


Investigation Results for the Clearing Speed of left-turning vehicles

Cycle time No.	Distance (m)	Time (s)	Clearing Vehicle	Clearing Speed (m/s)
1	6.10	1.88	Motorcycle	3.24
2	6.10	1.37	Motorcycle	4.45
3	6.10	1.44	Motorcycle	4.24
4	6.10	1.92	Passenger Car	3.18
5	6.10	1.20	Motorcycle	5.08
6	6.10	1.31	Motorcycle	4.66
7	6.10	1.50	Motorcycle	4.07
8	6.10	1.70	Motorcycle	3.59
9	6.10	1.10	Passenger Car	4.55
10	6.10	1.55	Motorcycle	3.94
11	6.10	1.60	Passenger Car	3.81
12	6.10	1.45	Passenger Car	4.21
13	6.10	1.44	Motorcycle	4.24
14	6.10	1.20	Motorcycle	5.08
15	6.10	0.70	Passenger Car	7.50
16	6.10	1.50	Passenger Car	4.07
17	6.10	0.87	Motorcycle	6.80
18	6.10	1.85	Passenger Car	3.30
19	6.10	1.34	Passenger Car	4.55
20	6.10	1.18	Passenger Car	5.17
21	6.10	1.40	Motorcycle	4.36
22	6.10	1.90	Passenger Car	3.21
23	6.10	1.25	Passenger Car	4.88
24	6.10	1.85	Motorcycle	3.30
25	6.10	2.60	Passenger Car	2.35
26	6.10	1.47	Motorcycle	4.15
27	6.10	1.84	Motorcycle	3.32
28	6.10	1.30	Passenger Car	4.69
29	6.10	1.45	Passenger Car	4.21
30	6.10	0.85	Passenger Car	4.77
31	6.10	1.40	Motorcycle	5.46
32	6.10	2.00	Bicycle	4.05
33	6.10	1.20	Motorcycle	4.08
34	6.10	1.10	Passenger Car	4.89
35	6.10	1.50	Passenger Car	4.07

Intersection:	Deawoo Intersection
Direction:	Cau Giay -> Lieu Giai **(left-turning)**
Signal program:	t_U = 98s ; t_R = 64s ; t_F = 31s ; t_G = 3s
Date:	24.12.2008
Time:	7:45 -> 8:45
The average clearing speed:	**5.2 m/s**

Cumulative Probability for the Clearing Speed v_r



Investigation Results for Motorcycle Homogeneous Saturation Flow

Intersection: Deawoo Intersection
Direction: Cau Giay -> Kim Ma **(go-through)**
Signal program: t_U = 133s ; t_R = 77s ; t_F = 53s ; t_G = 3s
Date: 18.12.2008 **Time:** 17:00 -> 18:00

The average saturation flow: **10,960** MCU/h
Standard deviation: **337** MCU/h
The maximal value: **11,956** MCU/h
The maximal value: **10,349** MCU/h

No.	Time (s)	Number of MCs (MC)	Saturation Flow (MCU/h/3.5m)
1	9.63	29	10,841
2	5.41	16	10,647
3	7.75	23	10,684
4	10.56	32	10,909
5	5.50	17	11,127
6	2.60	8	11,077
7	6.38	20	11,285
8	3.69	11	10,732
9	6.94	21	10,893
10	4.00	12	10,800
11	3.47	10	10,375
12	4.21	13	11,116
13	4.00	12	10,800
14	11.94	36	10,854
15	9.91	31	11,261
16	9.57	28	10,533
17	4.06	12	10,640
18	11.00	32	10,473
19	8.09	25	11,125
20	8.37	26	11,183
21	6.20	19	11,032
22	10.84	36	11,956
23	6.49	20	11,094
24	9.06	28	11,126
25	6.10	18	10,623
26	4.87	14	10,349
27	10.63	32	10,837
28	5.40	17	11,333
29	6.63	21	11,403
30	19.47	60	11,094
31	9.88	30	10,931
32	5.37	17	11,397
33	5.87	18	11,039
34	5.57	17	10,987
35	6.78	20	10,619
36	12.00	38	11,400
37	4.03	12	10,720
38	5.88	18	11,020
39	5.09	16	11,316
40	6.22	18	10,418
41	10.35	33	11,478
42	4.50	14	11,200
43	6.47	19	10,572

Curriculum Vitae

Personal Information

Full name:	Do Quoc Cuong
Date of birth:	03.01.1976
Place of birth:	Thaibinh, Vietnam
Nationality:	Vietnamese

Educational Backgrounds

1982 – 1987:	Primary school in Quynhcoi Town, Quynhphu District, Thaibinh Province.
1987 – 1990:	Junior high school specialized in Mathematics of Quynhphu District, Thaibinh Province.
1990 – 1991:	Junior high school specialized in Mathematics of Thaibinh City, Thaibinh Province.
1991 – 1994:	Senior high school specialized in Mathematics of Hanoi University of Science, Vietnam.
1994 - 1999:	The University of Transport and Communications, Vietnam. Civil Engineering Faculty, Gaining University Degree of Road and Highway Engineering. Thesis: "Applying the German method for designing Green Wave on some arterials in Hanoi".
2000 – 2003:	The University of Transport and Communications, Vietnam. Civil Engineering Faculty, Gaining Master Degree of Science in Civil Engineering (M.Sc). Thesis: "Applying the Japanese method for calculating the capacity of signalised intersections".
2006 – 2009:	Darmstadt University of Technology, Germany. Civil Engineering Faculty, Section Transport Planning and Traffic Engineering, Finalizing the doctoral dissertation in September 2009. Dissertation: "Traffic Signals in Motorcycle Dependent Cities"

Occupation

From 1999:	Teaching Assistant for Professors at the section Road and Highway Engineering. The University of Transport and Communications, Vietnam.

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