The continuous line continued...

A research to the effect of a continuous line at a highway access

Master thesis - Final report

B.G. (Bart) Leferink

17 June 2013

UNIVERSITEIT TWENTE.

ARCADIS
Infrastructure, environment, buildings

THE CONTINUOUS LINE CONTINUED...

A research to the effect of a continuous line at a highway access

Master thesis
Final Report

17 June 2013

B.G. (Bart) Leferink
Student number: 0137162
Telephone: 06 53 97 13 89
Email: b.g.leferink@alumnus.utwente.nl

University of Twente
Faculty of Engineering
Centre for Transport Studies
P.O. Box 217
7500 AE Enschede

USE System Engineering BV
Industriestraat 77
7482 EW Haaksbergen

Arcadis Nederland BV
Postbus 264
6800 AG Arnhem

Supervisors:
Dr T. (Tom) Thomas
Prof. Dr. E.C. (Eric) van Berkum

Ing. J.H.M. (Hans) Van der Kuil

Ir. M.E.J. (Martijn) Loot

UNIVERSITEIT TWENTE.

USE

ARCADIS
Infrastructure, environment, buildings

Source picture cover page: Ministerie van Verkeer en Waterstaat (2008).

PREFACE

The preface is written in Dutch

Op mijn eerste schooldag werd ik door mijn moeder naar school gebracht en bleef ze zelfs nog even zitten tijdens de eerste les. Nu, 20 jaar later, is het tijd om mijn studieperiode af te sluiten en wie zit er in de zaal...

Er gaat dus een einde komen aan mijn studieperiode waarin ik simpelweg veel plezier heb gehad. Na de middelbare school ben ik in 2006 begonnen met de studie Advanced Technology, waarna ik na een jaar ben ik overgestapt naar Civiele Techniek. Aan beide studies heb ik mooie herinneringen en goede vriendschappen overgehouden. Ook hiervoor geldt: de eersten die ik tijdens mijn studie tegenkwam zijn er ook vandaag weer bij.

Ik kijk terug op mooie buitenlandreizen, het bouwen van snelle betonkano's, leuke stageperiodes, een leerzame studieperiode in Wenen en op een fantastische studiereis naar Brazilië. Ik ben blij dat ik dit allemaal mee mocht maken.

Ook mijn afstudeertraject heb ik met veel plezier doorlopen, en zo hoop ik het ook af te ronden. Hiervoor wil ik in ieder geval alvast mijn begeleiders bedanken. Eric, bedankt voor de goede hulp bij vooral de voorbereiding en heldere kritiek. Tom, bedankt voor het enthousiasme en de interessante discussies, je hebt me aardig aan het denken gezet over kleine details in het onderzoek. Martijn, bedankt voor de spontane hulp. Het is fijn om uit het niets een goede begeleider erbij te krijgen die je de weg wijst als het nodig is. En Hans, bedankt dat je me de kans hebt gegeven om bij USE af te studeren. Je vertrouwen en de vrijheid die ik heb gekregen hebben tot een mooi resultaat geleid. Ook ben ik de medewerkers van Rijkswaterstaat, Munsterhuis, BAM Infra, Grontmij en de Gemeente Hengelo dankbaar voor de vrijwillige, enthousiaste en fijne medewerking.

Daarnaast iedereen van de "HP-groep" bedankt voor de steun en gezelschap bij het afstuderen, ik denk dat dit wederzijds is. Tot slot wil ik graag mijn familie en vooral vriendin bedanken voor de steun en fijne thuisbasis.

Tot slot zou ik, om vast het thema van dit rapport te introduceren, mijn studietijd kunnen omschrijven als een constante lijn die wat mij betreft gewoon doorgetrokken wordt!

SUMMARY

Congestion is a daily phenomenon at Dutch highways. One of the main causes are merges of on-ramps and highways. Only little research has been conducted to increase the highway performance at a merge. This research aims to evaluate to what extent a continuous line in the pre-merging section of the highway could increase the highway performance. This is done by performing a single case study at Hengelo-Zuid in combination with using a traffic simulation model.

A literature review shows the relation between traffic characteristics around a merge. At a macroscopic scale, these are capacity distributions, congestion, traffic flows and shock waves. At a microscopic scale, also headways, lane distributions and lane changing play an important role. Ramp metering is usually used to control on-ramp flows and to prevent highway congestion.

The study area at Hengelo-Zuid consists of a two-lane main carriageway. This area is analysed with data gathered from detection loops and road side video measurements. Data from the adjacent road network and traffic flow predictions for 2020 are used to determine effects at the wider network for now and the near future.

In the study area, a capacity drop of 19% is observed. Congestion takes on average 30 minutes, with highway and on-ramp delays up to respectively three and six minutes. Non-congested and congested traffic patterns are analysed. Cooperative lane changing is observed, which influences the lane distribution significantly.

The highway access at Hengelo-Zuid is modelled with the simulation software Fosim. Here, the capacity value is used as main performance indicator. The model is calibrated and validated with the observed microscopic and macroscopic traffic flow characteristics to improve the correctness of capacity calculations.

The effect of the continuous line is evaluated for three different on-ramp flows: 1) a signalised on-ramp flow; 2) a random on-ramp flow; and 3) a metered on-ramp flow (with Rijkswaterstaat-algorithm). All these on-ramp flows are theoretically and practically relevant.

The conclusion is that a continuous line can increase the capacity slightly but significantly. Though, for high signalised and metered on-ramp flows, this increase is constrained by negative effects at lane one, which is the left lane from the highway. For these situations, a shorter version of the line is preferable. The capacity increase is due to an increased share of vehicles at lane one. Aim of the line is to reach an optimal lane distribution. The amount of lane changes does not increase with a line, neither before the merge, nor after the merge. With a line, the ramp meter release rate could be increased significantly without increasing the congestion probability.

In Hengelo-Zuid, the continuous line can reduce the congestion probability. The reduction of highway congestion duration is estimated to be ten minutes; reduction in on-ramp delay is 1.5 minutes. The results imply that a continuous line could reduce congestion on several places in The Netherlands.

SAMENVATTING

Congestie komt dagelijks voor op de Nederlandse snelwegen. Een van de belangrijkste oorzaken hiervan is het invoegproces. Er is weinig onderzoek gedaan naar het verbeteren van dit soort knelpunten. Dit onderzoek gaat over het verbeteren van de prestatie van een invoeger met een verlengde doorgetrokken streep links. Het onderzoek is uitgevoerd met een single case study op de locatie Hengelo-Zuid gecombineerd met het gebruik van een verkeerssimulatiemodel.

In een literatuuronderzoek zijn verbanden gelegd tussen kenmerken van het verkeer rond een invoeger. Kenmerken op een macroniveau zijn capaciteitsverdelingen, congestie, verkeersstromen en schokgolven. Kenmerken op een microniveau zijn hiaatverdelingen, rijstrookverhoudingen en rijstrookwisselingen. Daarnaast worden op knelpunten veroorzaakt door een invoeger vaak toeritdoseringen gebruikt om de verkeersstroom te beheersen en file op de snelweg te voorkomen.

Het studiegebied in Hengelo-Zuid bevat een tweestrooks rijbaan (in de noordelijke richting). Het studiegebied is geanalyseerd met data uit detectielussen en video opnames. Met data van het bredere wegennetwerk, onderliggend wegennet en verkeersvoorspellingen van 2020 kunnen effecten van de doorgetrokken streep voor nu en de toekomst worden bepaald.

In het studiegebied is een capaciteitsval van 19% waargenomen. De dagelijkse file duurt ongeveer een half uur, wat vertragingen op de hoofdrijbaan en toerit van respectievelijk drie en zes minuten oplevert ten opzichte van het moment vóór de congestie. De verkeersstromen zonder én met congestie zijn geanalyseerd. Ook is coöperatief rijstrookwisselen waargenomen, een verschijnsel dat de rijstrookverhoudingen significant beïnvloedt.

De invoeger bij Hengelo-Zuid is gemodelleerd met het simulatieprogramma Fosim. De capaciteitsverdeling is gebruikt als prestatie-indicator. Om de kwaliteit van de capaciteitsberekeningen te verbetren is het model gecalibreerd en gevalideerd met de waargenomen microscopische en macroscopische verkeerskenmerken.

Het effect van de doorgetrokken streep links is beoordeeld voor drie verschillende stromen op de toerit. Dit zijn 1) een stroom gestuurd door een verkeersregelinstallatie (VRI); 2) een willekeurige verkeersstroom; en 3) een stroom gestuurd door een toeritdoseringsinstallatie (TDI) met Rijkswaterstaat-algoritme. Deze drie verkeersstromen zijn theoretisch en praktisch gezien relevant. Meerdere lengtes van de lijn zijn onderzocht, evenals negatieve effecten op de linker rijstrook.

De conclusie van dit onderzoek is dat een doorgetrokken streep links de capaciteit in Hengelo-Zuid enigszins, maar wel significant, kan verbeteren. Voor een hoge verkeersvraag op de toerit (van de TDI en VRI) wordt de capaciteitstoename beperkt door negatieve effecten op de linker rijstrook. Een kortere doorgetrekken streep links heeft dan de voorkeur. Voor andere verkeersvragen geniet een lange lijn de voorkeur. De capaciteitstoename komt door het hogere aandeel voertuigen op de linker strook. Het doel van de lijn is dan ook om te streven naar een optimale rijstrookverdeling. Het aantal rijstrookwisselingen neemt niet toe met de maatregel: niet voor, en ook niet na de invoeger. Met een doorgetrokken streep links zou de toe te laten intensiteit van een TDI verhoogd kunnen worden zonder de kans op file te vergroten.

Ook in Hengelo-Zuid kan een doorgetrokken streep links de kans op file verkleinen. Een grove schatting is dat de fileduur met ongeveer tien minuten verkleind kan worden. Vermindering van vertraging op de toerit is naar schatting anderhalve minuut. De resultaten laten zien dat een verlengde doorgetrokken streep links de kans op file op meerdere plaatsen in Nederland kan verkleinen.

CONTENTS

1. INTRODUCTION AND GOALS

This chapter introduces the background, subject and goal of this master thesis. Common used terms in this report are explained and the research is split up in research questions. The used strategies are also elaborated. All things considered, this chapter forms the basis for the research.

1.1 Introduction

Congestion is one of the main problems on the Dutch road network. The yearly costs of travel time losses were around 1 billion Euros in 2010. The amount of vehicle loss hours has increased by 50% during the period 2000-2010, and are about 9% of the total travel time in 2010 (Ministerie van Infrastructuur en Milieu, 2011). Building new roads does not always seem to be a solution anymore. For the period 2000-2010, traffic management provided a 6% decrease in vehicle loss hours in the national road network (Ministerie van Infrastructuur en Milieu, 2011). Solution provider USE System Engineering in Haaksbergen (USE) focuses on optimising highway operations by developing a.o. smart traffic management solutions, and aims to evaluate potential measures to achieve this.

One of the main bottlenecks is the merge of on-ramps and highways (Van Toorenburg, 1988; VID, 2012). This causes not only highway delays, but also large delays on the adjacent road networks. Chen et al. (2001) state that the major cause of congestion is an inefficient operation of highways during periods of high demand. The merging process causes speed adjustments which affect the traffic flow, and congestion occurs.

The main accepted solution to manage traffic demand at the merge of on-ramps and highways is ramp metering. Special traffic lights allow vehicles to enter the highway one by one, in such a way that the probability for highway congestion is minimised. Ramp metering seems to be an effective instrument to control the traffic flow. Several assessment studies showed that the capacity increased up to 5% (Middelham & Taale, 2006).

In some cases, the merge stays a bottleneck for the adjacent road network. A ramp meter has limitations, and dependent on the network traffic conditions, the merging process can be optimised further. USE is looking for possibilities to achieve this by looking for applications of dynamic road markings, which is an own developed product.

Only little research focused on influencing traffic conditions before the merge in order to improve the merging process. At the A35 highway access Hengelo-Zuid, Rijkswaterstaat already implemented a continuous white line along an on-ramp for safety reasons, as shown in Figure 1.1 (Ministerie van Verkeer en Waterstaat, 2008). In reference to the dynamic road markings from USE, this research is about improving the efficiency of a merge by extending this continuous line. Specific, this research focuses on extending the continuous line along the pre-merging section to provide more space for the merging traffic. For this, mainly the effect of the line is relevant, including the combination with a ramp meter, rather than the fact that the line can be dynamic. The used case is the situation in Hengelo-Zuid: the continuous line continued...

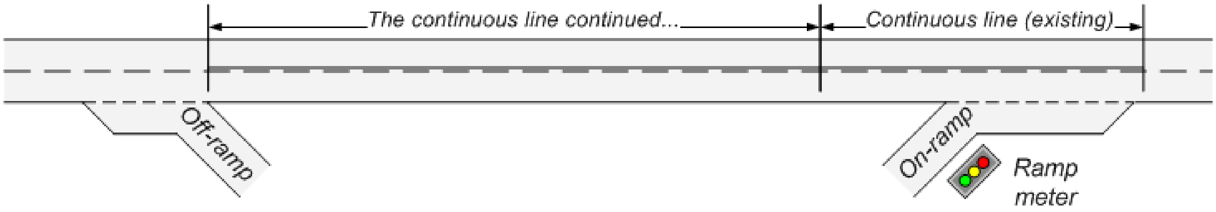


Figure 1.1: A sketch of the situation in Hengelo-Zuid with the proposed continuous line

1.2 Goal

This research has the following aim:

> *The aim of this research is to evaluate to what extent the highway performance* at Hengelo-Zuid could be increased, by analysing the effect of a continuous line in the pre-merging area on the highway and giving possible implications for a ramp meter.*

* The performance indicators are chosen later in this research.

1.3 Definitions

This section gives the definitions of common used terms in this report.

- Active bottleneck: a bottleneck at the highway which is not subject to a downstream bottleneck, i.e. the downstream flow is not constrained by the downstream supply.
- Capacity: the maximum amount of vehicles per time unit that is able to pass a cross section during a certain time period under the applying road-, traffic- and management conditions (DVS, 2011).
- Continuous line: a road marking between lane one and lane two, with a continuous white line on the side of lane one and a dashed line on the side of lane two, such that overtaking is allowed, and changing lane to lane two is prohibited.
- Delay: Time-delay of vehicles in a network, which is the difference between the travel time and the travel time under maximum flow.
- Headway: the time difference between two successive vehicles on a lane, measured between head and tail of the vehicle.
- Highway: a two-lane highway according to Dutch standards, including main carriageway, on- and off-ramps.
- Lane one: the left lane on a two-lane highway.
- Lane two: the right lane on a two-lane highway.
- Merging section: the longitudinal section on the highway along the acceleration lane.
- Off- and on-ramp: a one-way connecting road from/to the main carriageway.
- Performance: an optimal traffic situation, which can be expressed with different indicators.
- Pre-merging section: the section on the main carriageway before the on-ramp and main carriageway meet.
- Ramp meter: special traffic lights that control the traffic flow on an on-ramp to the highway
- Traffic flow: the actual flow of vehicles on a road section, including its characteristics.

The definitions are visualised in Figure 1.2.

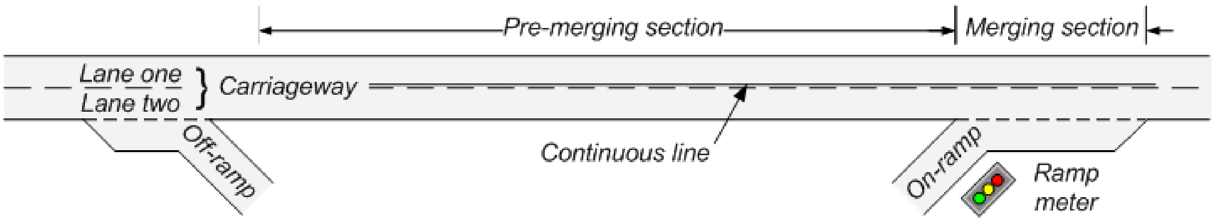


Figure 1.2: Visualisation of definitions

The traffic flows are in this research defined as shown in Figure 1.3.

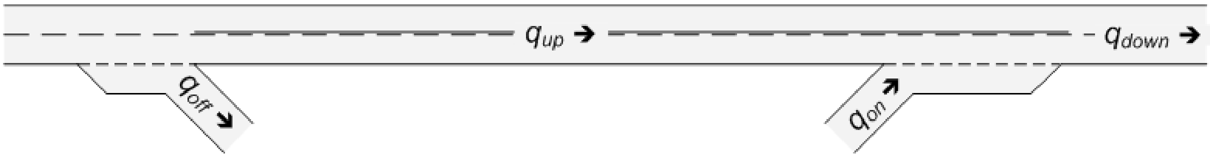


Figure 1.3: Denotation for traffic flows

1.4 Research questions

The research is set up with three main research questions, which are divided into several sub questions.

1. **What does the literature tell us about performance at merges and ramp meters?**
    1. What macroscopic and microscopic traffic flow characteristics are relevant in respect to highway performance, congestion and a continuous line?
    2. What is the effect of different ramp meter strategies at a merge?
2. **What is the current traffic situation and performance in the study area?**
    1. What are the macroscopic and microscopic traffic flow characteristics, found in research question 1.1?
    2. What performance indicator can be used to evaluate highway performance?
    3. What is the travel time delay at the highway and at the adjacent road network?
    4. To what extent is the access at Hengelo-Zuid an active bottleneck at the A35?
3. **What is the effect of a continuous line at the highway?**
    1. What is the effect of multiple on-ramp demands and flow patterns at the highway performance, including that from a ramp meter?
    2. What is the effect of different line lengths on highway performance?
    3. What is the effect of different line lengths on the traffic conditions at separate lanes?
    4. What is the effect of different line lengths on the macroscopic and microscopic traffic flow characteristics, found in research question 1.1?
    5. What is the effect on travel time delay at the highway and the adjacent road network?

1.5 Strategy

This research is executed according to the following strategy.

A single-case study is used to evaluate the effect of the continuous line. This strategy is chosen because the effect of the line is very location dependent. The local traffic situation should be analysed very carefully before effects can be measured. The single-case strategy places the research in a context which preserves the relation with the practical relevance. This makes effects better understandable, and effects can be quantified directly. The main disadvantage of the single-case study is a low external validity.

A literature study describes the state-of-the-art of the subjects related to this research. This is elaborated in research question one.

The next research question contains a situation analysis. For this, the traffic characteristics in the study area are elaborated. This is done with empirical traffic data like detection loop data, video measurements and traffic light log files. Also existing modelling software is used to estimate current and future traffic patterns.

After that, the effect of the continuous line is evaluated with a traffic simulation model. The traffic simulation model is calibrated and validated with data from detection loops and video measurements. Validation determines eventually the reliability and constraints of the model.

Different on-ramp flows, including the flow from a ramp meter, are eventually simulated to evaluate the effect of the continuous line.

2. THEORETICAL FRAMEWORK

The theoretical framework describes the state of the art of traffic characteristics around a merge, and forms the foundation for the research. The first section describes macroscopic traffic flow characteristics. The next two sections describe the microscopic traffic flow characteristics. Sections 2.4 and 2.5 respectively describe ramp metering and effects of ramp metering on the traffic flow. The state-of-the-art of the continuous line is discussed in Section 2.6, and the chapter ends with a summary.

2.1 Macroscopic traffic flow characteristics

This section describes the macroscopic traffic flow characteristics, and classifies the aspects capacity, congestion, shock waves and variances in the traffic flow. These aspects are all closely related to each other and play an important role in the research.

2.1.1 Capacity analysis

The research is about improving the highway performance at a highway merge. In this research, the capacity is an important performance indicator, which is explained later in this research. Several authors described the performance of highway merges. Liu & Hyman (2012) stated that the performance depends on three main factors: (1) geometric design; (2) traffic conditions, such as traffic flow volumes, temporal profiles and traffic composition; and (3) interactive behaviour between vehicles on the carriageway and from the on-ramp.

Highway capacities are dependent on a lot of factors, like weather conditions, slopes, road conditions, design, and traffic conditions. Though, capacities are often prescribed. The Dutch Handbook 'Capacity values Infrastructure Highways' (Handbook CIA) determines the capacity at merge at 4200 veh/h for a 2-lane highway, with 15% freight traffic. The reduction factor to respectively 0%, 5% and 10% heavy vehicles is 1.15, 1.10 and 1.05, considering a pcu value (passenger car unit) of 2.0 (DVS, 2011). In this research, only traffic situations with ideal weather conditions are taken into consideration.

The capacity is defined as the maximum amount of vehicles per time unit that can cross a certain section. This amount can be different per day. An example of measured capacities is shown in Figure 2.1. In the Handbook CIA, the median of this distribution is used as capacity value (DVS, 2011).

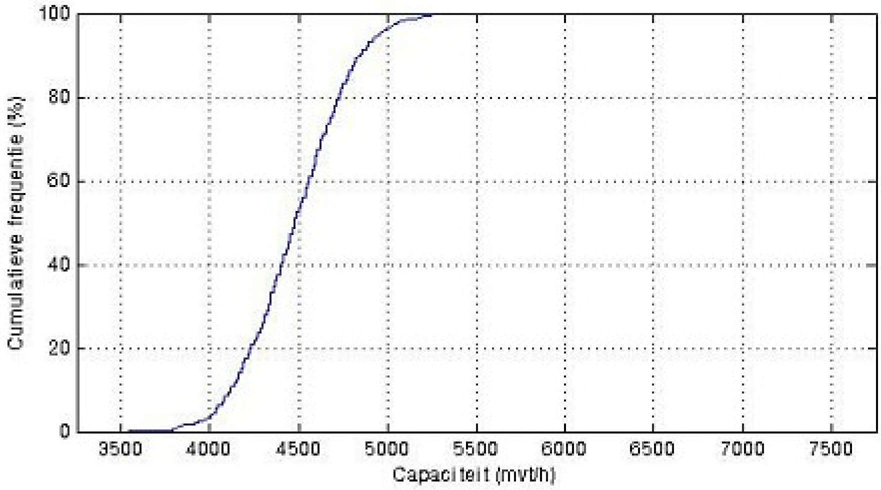


Figure 2.1: An example of a capacity distribution (DVS, 2011)

According to the CIA Handbook, the capacity value can be calculated with different methods.

1. The method Brilon (which is a product-limit-method) considers the intensity at the interval before congestion is detected at a detector upstream of the concerning cross section as a capacity observation. The median of a large number of measurements is considered as the capacity.

2. The Fosim method considers the median from the capacity distribution from the traffic simulation model 'FOSIM' as a capacity value.

3. The empirical-distribution method is commonly used to determine the discharge capacity. This is done by measuring the intensities downstream at moments where congestion is measured at a detector upstream.

Differences in the capacity values gathered with the Fosim method and method Brilon are in most cases between -10% and 10%. On average, the capacity values gathered with the Fosim method are 2% lower than capacity values calculated with the method Brilon (Grontmij, 2009).

2.1.2 Congestion at a merge

In this research, congestion is defined as traffic conditions where the average speed during an interval has dropped below a certain threshold, and can have different causes. This research focuses on causes where demand apparently exceeds the capacity, according to Equation 1:

$$q_{up}+q_{on}>C \quad (1)$$

where q_{up} is the upstream intensity, q_{on} the on-ramp intensity, C and the capacity. Here, the capacity depends on the proportion q_{up} and q_{on}, according to the Newell-Daganzo model (Newell, 1982; Daganzo, 1995). This model hypothesises that the capacity increases for high q_{up} / q_{on} ratios (see Figure 2.2).

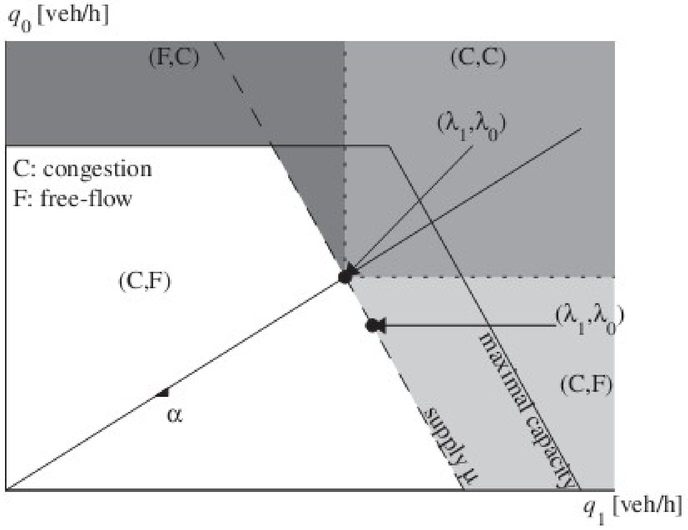


Figure 2.2: Merge diagram for the Newell-Daganzo (ND) model. q_0 represents the on-ramp flow, q_1 the downstream flow (Leclercq, Laval & Chiabaut, 2011)

Congestion due to a too high demand occurs due to unstable traffic patterns. Small speed adjustments are intensified by following traffic which causes shock waves (May, 1990). These disruptions can occur at the merge itself, due to speed differences of merging traffic, but also after the merge, due to relaxation.

Relaxation is the phenomenon that drivers accept shorter spacings at the moment a lane change is executed, which relaxes to normal values after a short period, usually 20 to 30 seconds (Daamen, Loot, & Hoogendoorn, 2010). Laval & Leclercq (2008) mention relaxation as the most important parameter describing the effect of lane changing on traffic streams. Loot (2009) observed shock waves starting approximately 2km downstream, plausible due to relaxation.

2.1.3 Shock waves at merging

An interesting phenomenon is the shock wave theory, which can describe certain traffic effects at a merge. The equation that describes the shock wave speeds between two successive traffic conditions is (Equation 2):

$$v_\omega = \frac{q_2 - q_1}{k_2 - k_1} \quad (2)$$

Before congestion occurs, there are basically two traffic flows on the highway: 1) the upstream traffic flow q_2 (q_{up} in this research); and 2) the downstream traffic flow q_1 (q_{down}), which is the upstream traffic flow plus the on-ramp traffic flow. The relative shock wave speed before congestion is, according to Equation 2, positive. Due to the merge the shock wave does not move forward.

Congestion at a merge is usually measured upstream of the merge location. Then, the relative speed is smaller than 0, and the shock wave has to move backwards. Whether congestion occurs or not depends on the speed at merge and upstream flow. Thus, not every disruption at the merge leads to a backward shock wave. Solving Equation 3 gives the constraints for a backward shock wave at a merge.

$$\frac{q_{up} - q_{down}}{k_{up} - k_{down}} < 0 \tag{3}$$

Since:

$$k = \frac{q}{v} \tag{4}$$

the following constraints must apply for a backward shock wave.

$$\begin{aligned} u_1: q_{up} &> q_{down} \\ u_2: q_{down} / k_{up} &> v_{down} \end{aligned} \tag{5}$$

The boundaries can be seen in the q/v diagram in Figure 2.3. In the figure, the green area represents the constraints from Equation 5. The upstream intensity q_{up} represents the first constraint. If the upstream intensity is higher than the intensity at merge, this constraint is satisfied. The speed at merge v_{down} represents the second constraint. If the speed at merge is in the lower part of the q/v diagram, also this constraint is satisfied.

The speed at merge is qualifying for whether backward shock waves occur or not. If the speed at merge is low, and thus the intensity at merge too, relative low upstream intensities are sufficient for backward shock waves. For higher speeds at merge, and thus also higher intensities at merge, the backward shock waves are less likely to occur.

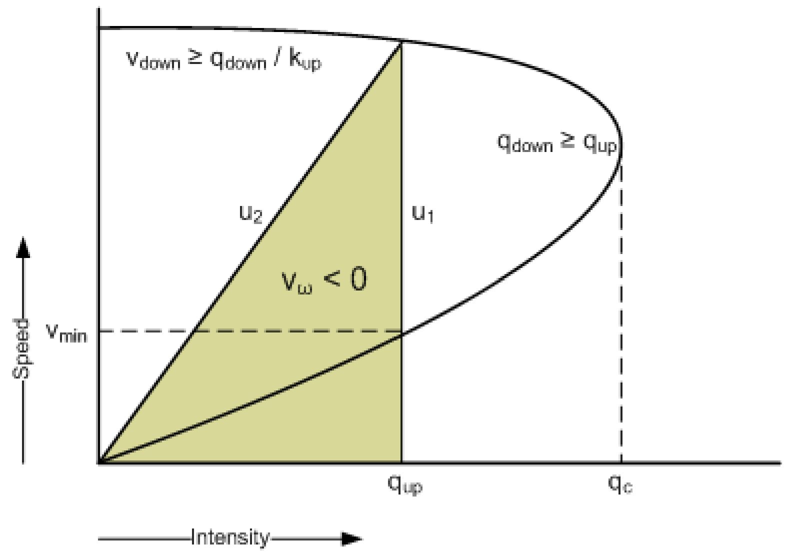


Figure 2.3: Shockwave effects in the fundamental q/v diagram.
Congestion occurs if the conditions at the merge are in the marked area.

The figure can also be used for disruptions at the merge. At high upstream intensities, small disruptions can cause backward shock waves.

Congestion can also recover according to the same theory. Backward recovery can occur if the upstream intensity drops. Forward recovery can occur if the speed at merge increases. The latter can occur if 1) the on-ramp flow decreases; or 2) a more fluent merging process, for example less trucks which causes disruptions.

Example
The shock waves at a merge are illustrated in a time-distance diagram in Figure 2.4. Here, dark colours represent higher intensities. The following stages occur:

- At t = 0, the traffic conditions are stable, and the flow downstream (F) reaches the capacity.
- At t = 1, a backward shock wave occurs (D) due to an increased on-ramp flow. The downstream flow is equal to the discharge capacity (E)
- At t = 2, the on-ramp flow dropped, the shock wave recovers (H) and the downstream flow reaches the capacity again (F)
- At t = 3, the upstream intensity increased (B). This shock wave grows much faster than at t = 1 (D).
- At t = 4, the intensity at the highway has dropped (C), such that the speed at the merge is higher than q_{down}/k_{up} which results in a backward recovery of the shock wave (D).
- If the on-ramp flow is 0 (t=5), the shock wave recovers fast.

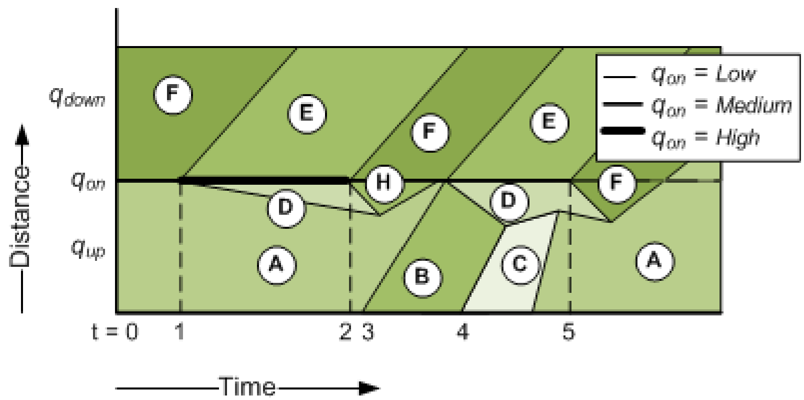


Figure 2.4: Time-distance diagram round a merge.

The relative speeds of the shock waves are shown in the fundamental q/k diagram in Figure 2.5. The descending lines represent a negative relative speed and can represent a backward forming shock wave or a forward recovery. This figure makes clear that congestion grows fast if the upstream flows are high and downstream flows are low.

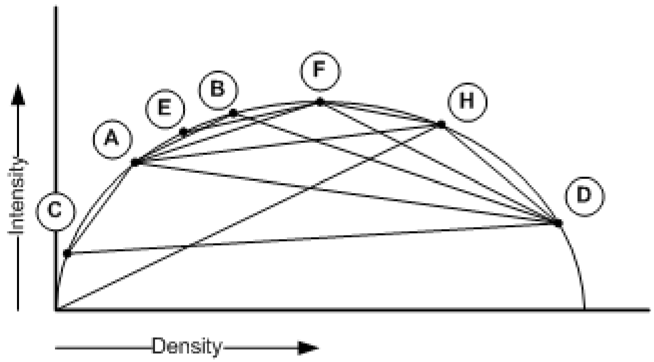


Figure 2.5: The lines in the fundamental q/k diagram represent shock wave speeds in the time-distance diagram from Figure 2.4.

2.1.4 Intensities and variances in the traffic flow

May (1990) describes the relationship between variances and volume-capacity ratios, based on two intensity distributions: random and single-valued count distributions. The random count distribution considers a traffic flow with a variance equal to the average mean flow. The single-valued count distribution considers a constant traffic flow with a variance equal to zero. The range of likely variances over the volume-capacity-range is a parabola-shaped area between these two distributions. If the highway intensity reaches the capacity, the variance in the flow is probably also relatively low. A traffic flow which is about equal to half the capacity has a relatively large variance.

Traffic flows with high variances can cause and also recover small shock waves. The low peaks in the flow can function as a buffer for the disruptions caused by the high peaks. More constant traffic flows do not have this buffer behaviour.

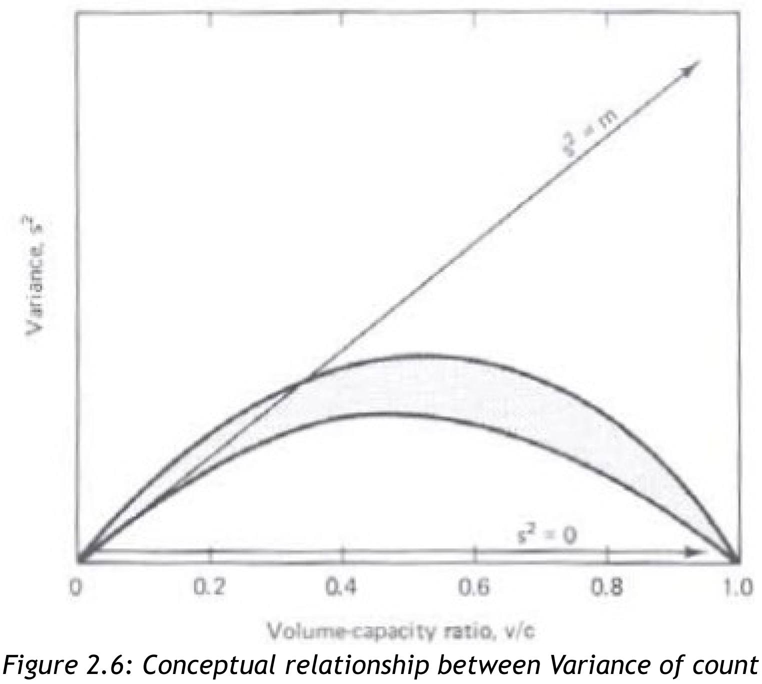


Figure 2.6: Conceptual relationship between Variance of count distribution and Volume-Capacity ratio (May, 1990)

2.2 Microscopic longitudinal traffic flow characteristics

2.2.1 Headway distributions

May (1990) describes three headway states. These are 1) the random headway state, in which headways are not correlated to each other and thus completely random; 2) the constant headway state, in which the headways are normal distributed among the mean headway; and 3) the intermediate headway state, which is a mixture between the random and constant headway state. Traffic flows with a low intensity tend to have a more (negative exponential distributed) random headway state, and traffic flows with a high intensity tend to have a more (normal distributed) constant headway state. The situation most encountered in practice is the intermediate headway state.

An example of a generalized mathematical model approach that has been proposed is the Pearson type III distribution, which is shown in Equation 6.

$$f(t)=\frac{\lambda}{\Gamma(K)}[\lambda(t-\alpha)]^{K-1}e^{-\lambda(t-\alpha)} \tag{6}$$

Where $f(t)$ is the probability function; λ is a parameter that is a function of the mean time headway and the two user-specified parameters, K and α; K is a user selected parameter between 0 and infinity that affects the shape of the distribution; α is a user selected parameter greater than or equal to zero that affects the horizontal shift of the distribution; t is the time headway being investigated and $\Gamma(K)$ is a gamma function, equivalent to $(K-1)!$. The value of K can be estimated with:

$$\hat{K}=\frac{\bar{t}-\alpha}{s} \tag{7}$$

The value of λ can be calculated with:

$$\lambda=\frac{K}{\bar{t}-\alpha} \tag{8}$$

The value for α is usually 0.5. This is the minimal time headway. The values for $\bar{t}$ and s can be determined with empirical traffic data.

2.2.2 Lane distributions

There are several examples of influencing lane distribution in (pre)merging areas. Knoop, Duret, Buisson & Van Arem (2010) researched the influence of variable speed limits on the lane distribution of traffic near merging zones. Knoop et al. (2010) found that a variable speed limit of 60 km/h increases the flow on the initially underutilized lane two. This leads to smaller gaps in the traffic at lane one, which causes the merging process to be more difficult.

Sarvi & Kuwahara (2008) did a study to improve the capacity of freeway merging sections by transferring these heavy vehicles from lane two to lane one. They concluded that by moving 10% of heavy vehicles to lane one, the total throughput of the merging section could be improved by 1%. The capacity of the freeway nearside lane was improved by 3%. Transferring 50% of the heavy vehicles to lane one could provide a capacity increase of 4%.

The lane distribution could thus have a large influence on the throughput of the traffic.

2.3 Microscopic lateral traffic flow characteristics

2.3.1 Lane changing

Daamen et al. (2010) stated that there has never been given much attention to lateral driving behaviour, such as lane changing. Based on a short literature study they state the effect of lane changes on traffic conditions is not negligible, and that lane changes may trigger a capacity drop between free flow and congested flow (Laval & Leclercq, 2008).

There are two types of lane change:

1. Voluntary lane change: vehicles can decide on their own whether they want to change lane or not;
2. Mandatory lane change: vehicles must change lane due to a merge or end-of-lane.

In this research, both types of lane changing are relevant.

2.3.2 Voluntary lane change

Knoop et al. (2010) state that there are two processes in lane distribution. These are the desire to change lanes and the possibility to change lanes. Daganzo (2002) gave a theoretical basis for the desire to change lanes. He distinguished drivers into two categories: aggressive ones (rabbits) and less aggressive ones (slugs). This mix can create congested patterns.

Another theory describes the utility of changing lanes. The utility of a higher speed can be weight against the disutility of acceleration. This consideration leads to a decision to change lanes (Kesting, Treiber, & Helbing, 2007). The combined decisions of all drivers lead to a lane distribution (Knoop et al., 2010).

Furthermore, only little research has been done on lane distribution in (pre-)merging areas, as Knoop et al. (2010) stated that most studies on lane distribution focus on an equilibrium without the influence of merging traffic. Research however did find out that the presence of a heavy vehicle ahead as an important factor of lane selection (Hidas, 2005). Though, Sarvi & Kuwahara (2008) stated that there have been very few studies that are concerned with the traffic behaviour and characteristics of heavy vehicles in these situations.

2.3.3 Merging

The merging manoeuvre is a specific type of lane changing, namely a mandatory lane change. Daamen et al. (2010) stated that this merging manoeuvre depends on the accepted gap, which eventually determines the merge location. The acceptance of gaps here is based on the size of available gaps, road layout, traffic conditions, the individual critical gap, relaxation and cooperative lane change.

Merging is thus a mandatory lane change. Hidas (2005) modelled vehicle interactions in merging and weaving traffic, and described three types of mandatory lane changes:

1. free lane change, where there is no noticeable change in the relative gap between leader and follower before and after the lane change;
2. forced lane change, where the vehicle is forced to change lane such that leader and follower in the target lane have to adjust their speeds; and
3. a cooperative lane change, where the follower slowed down to allow a vehicle to enter the lane.

Latter is an important phenomenon in the lane distribution theory.

Cooperative lane changing
Traffic on the main carriageway tends to create space for merging traffic. Van Toorenburg (1988) explains this as following. Normally, traffic on the highway has right of way over traffic on the on-ramp. In (almost) congested traffic conditions the opposite occurs. Merging traffic must change lane to the main carriageway, and therefore the drivers on the main carriageway provide space for the merging traffic. In these situations, on-ramp traffic has de facto priority due to the forced lane change and cooperative behaviour of traffic.

There are two types of cooperative lane changing:

1. A lane change manoeuvre of the lag vehicle to provide space for the merging vehicle;
2. A deceleration manoeuvre of the lag vehicle to provide space for the merging vehicle. This phenomenon is called courtesy yielding.

Lane changing has clearly an effect on the lane distribution, in contradiction to courtesy yielding.

2.4 Ramp metering
A common measure to prevent the disruptions in the merging process is ramp metering. Ramp metering is the control of a traffic stream from an on-ramp to the highway, which is done by using special traffic lights that allow vehicles to enter the highway one by one. A fraction of the delay on the highway is transferred to a delay on the on-ramp; the rest of the delay is eliminated (Chen et al., 2001).

2.4.1 Classification
There are multiple ramp meter strategies. Most applied strategies are reactive, which means that they operate at a tactical level and have the aim to maintain the highway traffic conditions close to desired values by the use of real-time measurements (Papageorgiou & Kotsialos, 2002). Reactive strategies are commonly applied worldwide, and can be divided in local and coordinated strategies. Local strategies focus on a single highway entrance. Coordinated strategies manage several successive highway entrances in order to manage the flow on an entire highway section (Bie, 2011).

The local strategy also has sub strategies. These are the release-to-gap strategy, demand-capacity (DC) strategy, the occupancy strategy and the ALINEA strategy (Bie, 2011). The release-to-gap strategy aims to release vehicles into local gaps on the traffic flow on the main carriageway. The demand-capacity strategy attempts to add to the measured upstream flow as much as ramp flow necessary to reach the downstream highway capacity. The occupancy strategy is based on the same philosophy as the DC strategy, but it uses upstream occupancy-based estimations. The ALINEA strategy is also occupancy-based, but relies on the downstream occupancy, which makes it a closed-loop strategy (Papageorgiou & Kotsialos, 2002).

2.4.2 Control strategies in The Netherlands
The most Dutch local ramp meters are equipped with the Rijkswaterstaat (RWS) strategy. This is a form of the DC strategy. The RWS strategy has other turning on and off rules than a regular DC strategy. A DC ramp meter turns on if the downstream occupancy exceeds a critical value. A RWS ramp meter turns on if the upstream intensity or up/downstream velocities exceed a threshold (Bie, 2011). The release rate r_k is calculated as

$$r_k = C - I_{k-1} \tag{9}$$

where r_k is the release rate (the amount of vehicles that is allowed to enter the highway in time interval), C is the pre-specified capacity of the highway downstream the on-ramp and I_{k-1} is the measured and smoothed upstream flow in the previous time interval (Middelham & Taale, 2006). The measure location is usually 500 meter before the start of the merging section (Vlek, personal communication, 2012).

Other Dutch ramp meters are equipped with the ALINEA algorithm (Traag, personal communication, 2012). The release rate here is calculated as

$$r_k = r_{k-1} + K(O_s - O_{k-1}) \tag{10}$$

where K is a constant, O_s the occupancy set point and O_{k-1} the occupancy measured downstream the on-ramp in the previous time interval (Middelham & Taale, 2006).

During the 1990s, several tests have been performed in the Netherlands about a FUZZY strategy, which is a form of the release-to-gap strategy (Noordmans, personal communication, 2012). The strategy is based on three input variables: the speed upstream the on-ramp, speed downstream the on-ramp and the time a queue is present on the on-ramp. Certain rules classify the input variables and determine the cycle time (Middelham & Taale, 2006). The FUZZY strategy is never implemented due to difficulties with the longer green times and problems with switching on and off (Noordmans, personal communication, 2012; Taale, Slager, & Rosloot, 1996).

Both ALINEA and RWS strategies use 5-minute data as input value. The algorithms use the formula in Equation 11 to calculate the cycle time (Ministerie van Verkeer en Waterstaat, 2007).

$$t = \frac{n_{lanes} \cdot n_{vehicles} \cdot 3600}{C_{RW} - I_{RW}} \tag{11}$$

Where I_{RW} is the smoothed hour intensity for the main carriageway upstream (q_{up}), according to Equation 12 (Ministerie van Verkeer en Waterstaat, 2007).

$$I_{a,new} = a \cdot I + (1-a) \cdot I_{a,old} \tag{12}$$

Where I is the 5-minute intensity. The values of a are 0.1 for $I_{a,new} < I_{a,old}$ and 0.4 for $I_{a,new} > I_{a,old}$.

The capacity C_{RW} which is used can be either the discharge capacity, or the maximum capacity. In the case the maximum capacity is used, a threshold is used such that congestion is prevented. If we look at the capacity distribution in Figure 2.1, the threshold must prevent the congestion that occurs at the lower intensities (in the bottom left of the figure).

2.4.3 Ramp meter limitations

Most ramp meters, such as RWS ramp meters, use the 5-minute average flow rate. If the traffic flow has a small variance, this is useful. Though, for traffic flows with high variances, the ramp meter release rate is quite inefficient. Variances in the traffic flow are ignored, which possibly could affect the stability of the traffic. The generalisation of the peaks leads to an inefficient use of the traffic demands, which may lead to unnecessary delays at the on-ramp.

In the United States tests have been performed regarding release-to-gap algorithms. Dependent on the availability of a gap at the highway traffic flow, on-ramp traffic was released. The method turned out to be too unreliable (Van Toorenburg, 1988). Tests with release-to-gap theories based on larger intervals than single headways, but smaller than 5-minutes, are unknown.

2.4.4 Ramp meter evaluation

Several indicators can be used to evaluate ramp meters. Chen et al. (2001) defined a general indicator for congestion, as they defined congestion as the delay between the travel time and travel time under maximum flow. Papageorgiou, Hadj-Salem, and Middelham (1997) analysed the ALINEA and RWS strategies and used the evaluation criteria: total travel time on the main carriageway; total waiting time at the ramp; total time spent; total travel distance; mean speed; and mean congestion duration. Middelham & Taale (2006) used the capacity of the bottleneck (capacity at merge), use of on-ramp, total delay, and the amount of red light violations as indicators.

In their study, Papageorgiou et al. (1997) summarised field results from ramp meters, and found that the ALINEA strategy was the most efficient. A test case on the A10 showed that the total time spent with the ALINEA strategy was 8.3% less than using the RWS strategy (considering both highway and on-ramp). The total travel distance was 1.3% higher and the mean speed was 8.2% higher. The main difference between the strategies is that ALINEA reacts smoothly even on slight differences between the downstream occupancy and an occupancy set level, whereas the RWS strategy only react on excessive occupancies, only after a threshold value is reached. If the upstream flow varies, the ALINEA strategy seems to work as a smoothing filter.

Several assessment studies showed that the RWS strategy provided a capacity increase up to 5% in 7 out of 10 cases (Middelham & Taale, 2006). Middelham & Taale (2006) also stated that the FUZZY strategy gave better results than the RWS and ALINEA strategy as the capacity increased with about 5% (however not significant), lead to higher speeds and lower travel times. This implies that improvement of both RWS and ALINEA is possible. The FUZZY strategy is not applied due to difficulties with longer green times (Noordmans, personal communication, 2012; Taale et al., 1996).

2.5 Effects Ramp metering on traffic flow

2.5.1 Effects on lane distribution

Wu, McDonald, and Chatterjee (2007) studied the effect of ramp metering on the traffic behaviour. They found that ramp metering only neither has significant effects on speeds and headways on lane two and lane three (considering a three-lane highway) in the pre-merging zone, nor on traffic speeds, headways and acceleration/deceleration rates in the merging section. Though, ramp metering has an effect on the lane change in the pre-merging zone, as described in Section 2.5.2.

Wu et al. (2007) also observed accepted gap sizes in situations where ramp metering is turned off and on. They found that the accepted gap size was much larger in situations with ramp metering turned on. Unfortunately they did not make clear why.

2.5.2 Effects on lane change

Wu et al. (2007) found that ramp metering does not have significant effects on traffic speeds, headways and acceleration/deceleration rates for passing traffic. Though, there is a significant increase of the number of lane changes from lane one to lane two in the pre-merge zone with ramp metering turned on. This resulted in significant increases in headways of traffic on lane one in the pre-merge and merge sections. Though, this change only happened in a very limited area. The reason for this is that merging vehicles have a significant lower speed when ramp metering is turned on. The length from the traffic light to the merging point is namely not long enough to accelerate to the same speed levels of that when ramp metering is turned off.

Cassidy and Rudjanakanoknad (2005) revealed capacity drops at an on-ramp bottleneck equipped with ramp metering, based on empirical observations. They found that "(i) the capacity drop occurs simultaneously with an increase in lane-changing counts and shoulder lane vehicle accumulation, and that (ii) controlling the ramp-metering rate could mitigate this lane changing and accumulation, so that high merge capacities could be restored" (Laval & Leclercq, 2008).

2.6 Continuous line

Hardly any studies have been performed to the effect of road markings on merging. Rijkswaterstaat however researched and implemented a continuous line to improve safety at merging sections, as visible in Figure 1.1. This continuous white line stretches from a few hundred meters upstream the merging point to the end of the acceleration lane of the on-ramp. The line prohibits traffic to change lane to lane one, which provides more space for the merging traffic. The concept is studied as a part of the RWS program ‘Fileproof’ and is implemented on 48 locations in The Netherlands in 2008 (Ministerie van Verkeer en Waterstaat, 2008). According to Molenkamp (2008), the measure provided a speed increase of 5 km/h.

Tests about the continuous line as implemented by Rijkswaterstaat (Figure 1.1) showed a steady ignorance of the line during peak periods. It seemed that there was a relation between the traffic demand and the negations of the continuous line. In relative calm morning peak periods there were about five negations per hour, and in the busy evening peak period between ten and twenty negations per hour (Ministerie van Verkeer en Waterstaat, 2006).

2.7 Summary

This research focuses on congestion at merge due to a too high capacity. This kind of congestion is characterised by speed disruptions, followed by backward shock waves. These backward shock wave are determined by two constraints. The behaviour of backward shock waves also depends on variances in the traffic flow.

Traffic flows usually have intermediate headway states. This is a mixture between the random headway state (where headways are randomly distributed) and the constant headway state (where headways are normal distributed among the mean headway). An intermediate headway state is characterised by a Poisson distribution. An intermediate headway state also refers to a traffic flow with relative high variances. Herewith the link is made between the headway state and shock wave behaviour.

Lane distributions can have an influence on the merging process. Lower speeds for example increase the traffic flow at lane two, which leads to smaller gaps and a more difficult merging process. The effect on lane changes on traffic conditions is not negligible. Furthermore, only few studies focused on influencing lane distributions for merging traffic. Though, drivers also automatically influence the lane distributions at a merge, by cooperative lane changing.

Ramp meters are often used at a merge to prevent congestion at the highway. In The Netherlands, mainly RWS algorithms are used. RWS ramp meters only take 5-minute flows into account, which ignores traffic variances within this interval. The maximum throughput with a ramp meter is equal to the discharge capacity, which does not undo the capacity drop. Release-to-gap algorithms have been tested in the United States, but seemed to be too unreliable. Ramp metering does have an effect on the lane distribution. Due to the slower speed at on-ramps, more cooperative lane changing occurs.

The effect of a continuous line on lane distributions is not studied before. Shorter versions of the line refer to a speed increase at the merge of 5 km/h, but involved 20 negations per hour in the evening peak. This literature study showed that influencing lane distributions is not studied before, but could have an effect on the merging process.

3. STUDY AREA

This chapter describes the layout and of the study area at Hengelo-Zuid, and the data sources within this area for the research. The theory described in Chapter 2 is applied on this area. The first section describes the layout of the study area, followed by a section about the description of the used data sources in the study area. The validation of the used data sources is discussed, even as the external validity of the study area. The chapter ends with a short summary.

3.1 Layout

The study area in this research is the 'Rijksweg 35' (RW35) access 27 Hengelo-Zuid. The RW35 is a national road between Zwolle and the German border near Enschede. From Wierden to Enschede, the road is a 2×2 lane highway (A35) and a 3×3 lane highway between interchanges Azelo and Buren, where the road is combined with the A1 (Figure 3.1 a and Appendix II.).

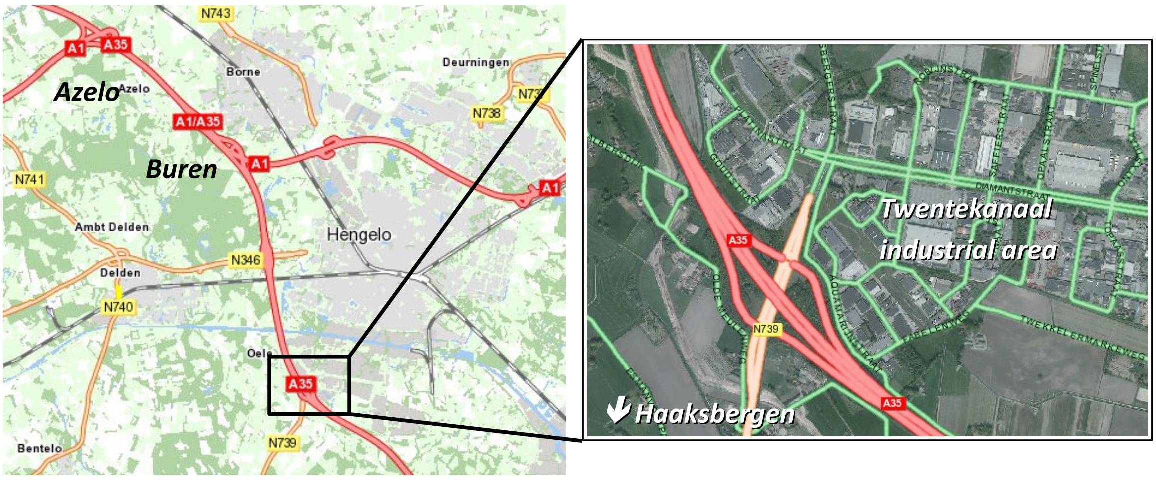


Figure 3.1 a and b: RW35, access Hengelo-Zuid and the adjacent road network (derived from Eurosense, 2008)

The access Hengelo-Zuid serves the industrial area 'Twentekanaal' and the provincial road N739 towards Haaksbergen (Figure 3.1 b). The northbound of the access – which is the left side of the highway – heads for the directions Almelo / Deventer / Oldenzaal (towards interchange Buren) and is equipped with a ramp meter and a short continuous line. This research focuses on this side of the highway access (RW35 access 27 – Left) (see Figure 3.2).

The continuous line continued...

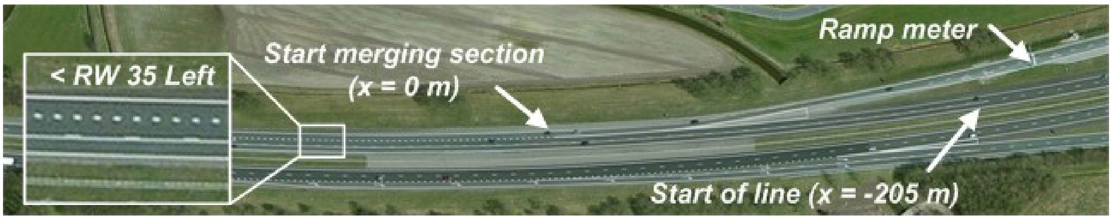


Figure 3.2: Merging area and continuous line at access Hengelo-Zuid (derived from Nokia & Microsoft Corporation, 2013).

Recently, the situation on this highway had been changed. Since 1 September 2012 the maximum speed from the section Enschede-West until intersection 'Buren' was set from 120 km/h to 130 km/h (Schultz van Haegen, 2012). The maximum speed from intersection 'Buren' to intersection 'Azelo' stayed 120 km/h.

The ramp meter however, does not operate due to an error. This was the case during the whole study period. Therefore, the situation analysis does not include situations with a ramp meter. The ramp meter is separately evaluated separately in Chapter Error: Reference source not found.

Appendix II. shows a complete overview of the RW35 – Left with names, accesses and locations of induction loops.

3.2 Data sources

Evaluating the effect of the continuous line requires a variety of data, which is described in this section.

At first, data is required for the situation analysis. The situations at the access Hengelo-Zuid is analysed on both a macroscopic level (for example intensities) and microscopic level (for example headways). The situations at the adjacent and wider road network are also elaborated. This forms a foundation for 1) the effect of the line at the adjacent road network; and 2) the effect of the line at the wider network. This analysis includes future predictions about the traffic flow. The calibration and validation of the traffic simulation model requires both macroscopic and microscopic traffic data from the access Hengelo-Zuid. Historical weather data is used to obtain only data with ideal weather conditions.

The required data is gathered from the following sources:

1. detection loops at the A35 provide the macroscopic traffic data;
2. video measurements next to the on-ramp provide the microscopic traffic data;
3. traffic counts and traffic light log files provide traffic flows from the adjacent road network;
4. the regional traffic model provides current and future traffic flow patterns; and
5. measurements from weather station Twente provide historical weather data.

The next sections evaluate the data sources.

3.2.1 Detection loops

Data from different detection loops at the A35 is available. The locations of the detection loops of Hengelo-Zuid are shown in Figure 3.3, locations from the detection loops at the A35 in Appendix II.

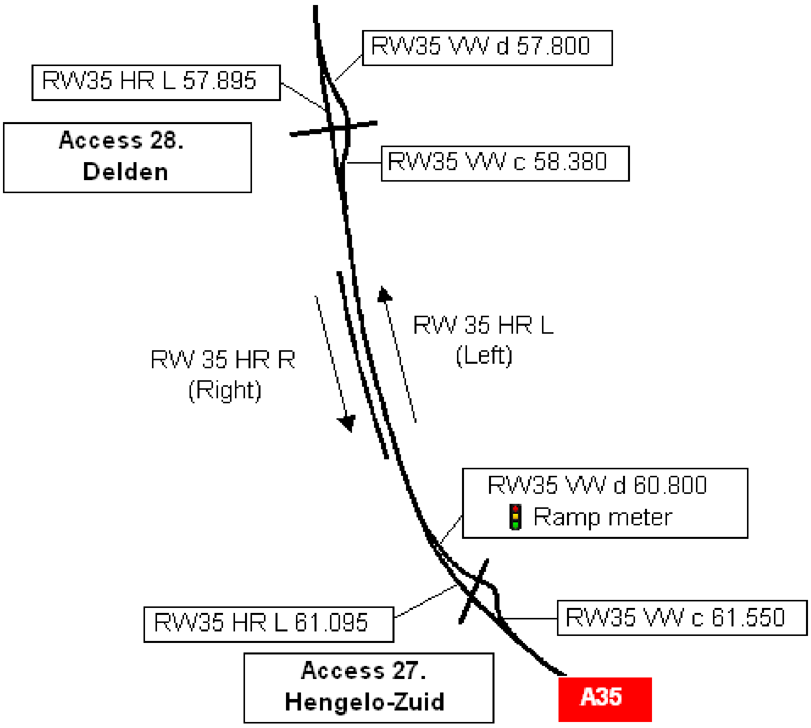


Figure 3.3: Detection loops in the study area. The full image is shown in Appendix II.

The detection loops provide minute-average intensities and speeds. Three vehicle categories are distinguished, according to the Dutch standards (Geerarts & Van Bergen, 2003). These vehicle categories are:

- Category 1 (CAT I) vehicles: <5.6 meter
- Category 2 (CAT II) vehicles: 5.6 – 12.2 meter
- Category 3 (CAT III) vehicles: >12.2 meter.

The data also contains information about the reliability of each minute-sample. This is indicated with a 'j' (reliable) or 'n' (not reliable). The data showed that the detection loops at the access Hengelo-Zuid have a quite high downtime, which results in a relative low amount of available data from these loops:

- The detection loop RW35 VW d 60.800 (on-ramp) hardly gives reliable data;
- The detection loop RW35 HR L 61.095 (highway) only gives useful data less than 50% of the time;

The other detection loops provide a sufficient amount of reliable data.

Only Mondays, Tuesdays, Wednesdays and Thursdays are selected. There are namely indications that the traffic composition during these days is different than at other days (Van der Kuil, 2012). Furthermore, only data has been selected from days after the new speed limit has been set (at 1 September 2012).

3.2.2 Video measurements

Video measurements were used to gather microscopic vehicle data from the access Hengelo-Zuid. Video cameras were installed at a car dealer next to the highway access, with sight on the pre-merging location and on-ramp (Figure 3.4 and Figure 3.5). The cameras have a view at the section approximately 250 meters before the merge.

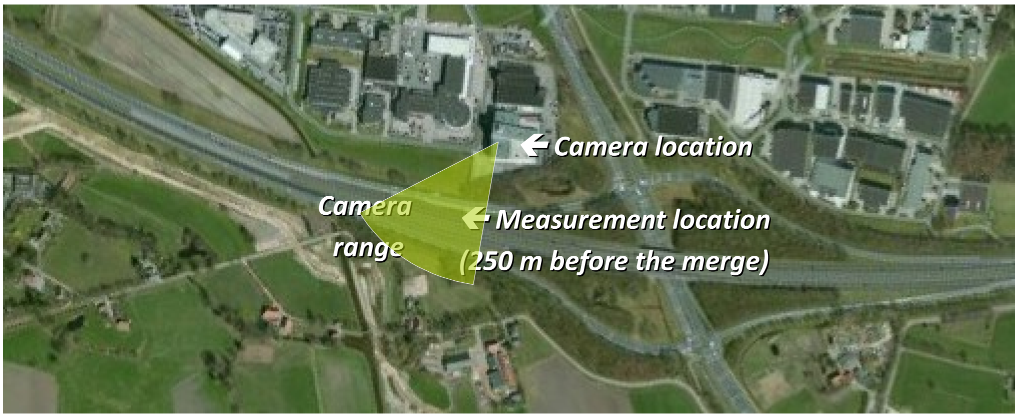


Figure 3.4: Camera location, range and measurement location of the video dataset (derived from Google, 2013).

The frame rate of the video camera is 25 frames per second. During the analysis of the video recordings, reference points are created on the screen. Per frame is determined whether a vehicle crosses that reference point or not. The method resulted in a precision of 0.04 seconds, which is important for a precise headway analysis.

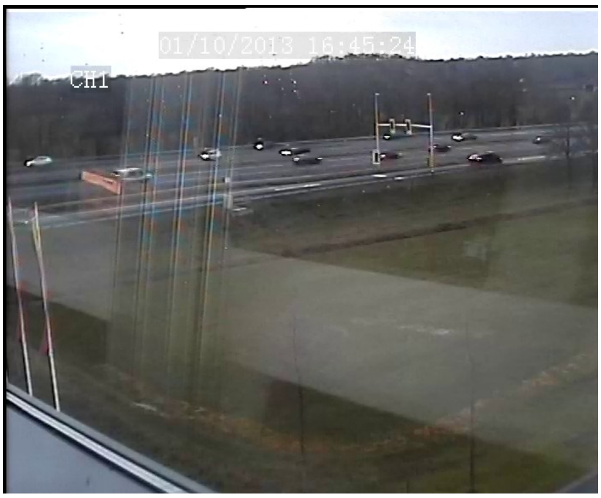


Figure 3.5: Snapshot from the camera view

Unfortunately, the resolution of the used camera system was not sufficient to record the complete merging section. The recordings were performed in evening peak hours, on dayparts without frost and rain. Data is used from Thursday 10 January 2013 and Monday 28 January 2013, with recording times from respectively 15:25 – 17:20 and 16:25 – 17:15.

3.2.3 Traffic counts and traffic light log data

The continuous line eventually aims for a higher highway performance, which also means more throughput at the on-ramp. This has an effect on the adjacent road network, and this situation is therefore also analysed.

Traffic data is gathered from the two intersections at the adjacent road network: 1) the intersection N739 – A35, which is the access to the A35 at Hengelo-Zuid; and 2) the intersection Haaksbergerweg – Diamantstraat, which connects the industrial area 'Twentekanaal' to the provincial road N739 (see Figure 3.6).

Figure 3.6: Intersections at the adjacent road network (derived from Eurosense, 2008).

Two data sources are used:

1. Traffic light log files from the intersection Haaksbergerweg – Diamantstraat, from 8 – 12 April 2013 (Gemeente Hengelo, 2013).
2. Visual traffic flow counts intersection A35 – N739, from 21 September 2006 (Provincie Overijssel, 2006)

The second data source is relatively old. This data source is therefore mainly used to estimate flow patterns, rather than absolute traffic flow values.

3.2.4 Regional traffic model

The regional traffic model is a model which estimates the traffic flow patterns within the region of Twente. It is developed by Goudappel Coffeng, calibrated with observed traffic counts and estimates future traffic flow patterns based on historical trends.

The regional traffic model provided the following information:

- Selected link analysis of the A35 (after the on-ramp at Hengelo-Zuid) for 2012 and 2020
- Selected link analysis for the on-ramp at Hengelo-Zuid for 2012 and 2020
- Traffic flow patterns of the A35 for 2012 and 2020

The model estimates the two-hour flow rates per link in the network in terms of vehicles per 2 hours. The model for 2020 includes planned roads, such as the A18 from Haaksbergen to Enschede. This road for example discharges the flow at the provincial road N739. The model uses an average increase of traffic flow of 2% (Goudappel Coffeng, 2013).

3.2.5 Weather data

The effect of the continuous line is evaluated with data from ideal weather conditions. Historical weather data is used to select the traffic data under which these circumstances apply.

Only dry periods (less than 0.1 mm precipitation during a 24-hour period) without frost (+3°C during the measurement period) are selected for data analysis. The data is gathered from historical weather data measured at weather station Twenthe from the Royal Dutch Meteorological Institute (KNMI, 2013).

3.3 Validation of the data

Since a large amount of the detection loop data from Hengelo-Zuid is not reliable, this data can be validated by comparing it with the video measurements.

For a relative large sample, data is available from both video measurements, and detection loops at Hengelo-Zuid and Delden (28 January 2013 16:26 – 16:54). Within this sample, the following comparisons are made:

- The first two comparisons are made between minute intensities and 5-minute intensities from the video measurements and detection loops at Hengelo-Zuid (q_{up}). The distance between both measurement locations is 250 meter.
- The third comparison is made between 5-minute intensities from the video measurements and intensities measured at the highway access Delden (q_{down}). The distance between both locations is approximately 3 km. Considering a speed of 120 km/h, the travel time between both locations is approximately 90 seconds. Considering a speed of 90 km/h, the travel time is 30 seconds longer. Therefore, there is a time difference of 2 minutes in the intensities that are compared.

The observed flows are tested with hypothesis 13, according to the T-test (Appendix I.). The average difference between the samples is calculated according to Equation 14.

$$H_0 : \Delta\bar{I}_m = 0 ; H_1 : \Delta\bar{I}_m \neq 0 \text{ . Accept } H_0 \text{ if } T_{n-1} < T_{0.05} \tag{13}$$

$$\Delta\bar{I}_m = \frac{\sum_{m=1}^{n} \left| I_{m,Video} - I_{m,Loop} \right|}{n} \tag{14}$$

The results of the validation are shown in Table 3.1.

Table 3.1: Traffic flows from the video measurements and detection loops compared

I_m	**Video**	**Detection loop**	**Interval**	**n**	$\Delta\bar{I}$	$S_{\Delta\bar{I}}$	T_{n-1}	$T_{0.05}$	**Accept** H_0**?**
$q_{up,\ min}$	Highway	HR L 61.095	1 min	27	279 veh/h	173 veh/h	1.62	1.70	Yes
$q_{up,5\text{-}min}$	Highway	HR L 61.095	5 min	25	47 veh/h	40 veh/h	1.20	1.71	Yes
$q_{down,5\text{-}min}$	Highway + on-ramp	HR L 57.895 + VW c 58.380	5 min	23	62 veh/h	51 veh/h	1.23	1.72	Yes

The table shows that the flows from both datasets match within a 95% confidence interval. The comparison is also visualised in Figure 3.7.

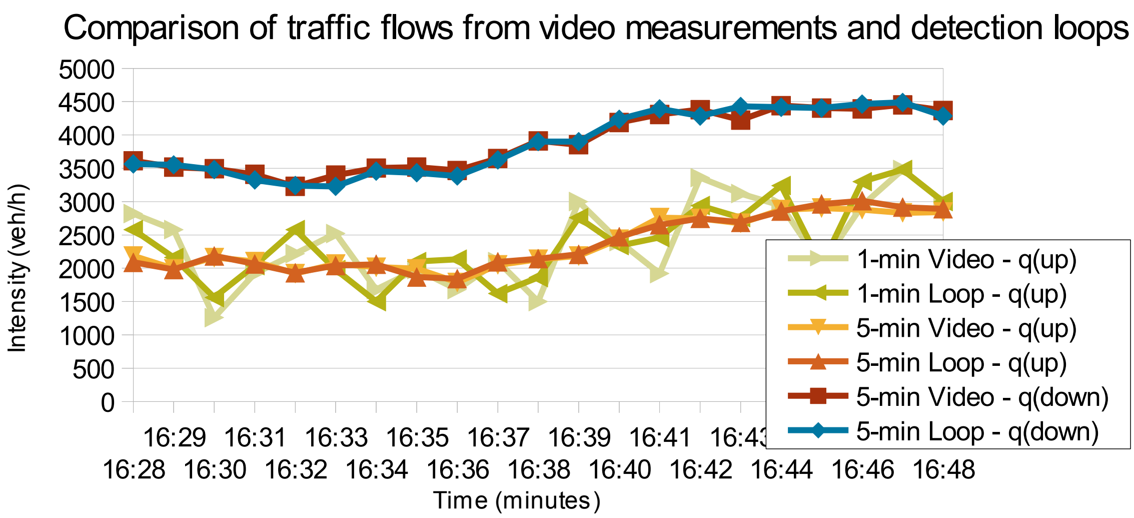


Figure 3.7: Comparison of traffic flows visualised

The figure shows the 1-minute traffic flows match relatively good. No clear offsets or shifts between both minute-intensities can be observed. The same applies for the 5-minute traffic flows.

3.4 External validity

The used case in this research is the situation in Hengelo-Zuid. A main disadvantage of a single case study is the low external validity. At almost any location, traffic conditions and road characteristics are different. Though, there are multiple places in The Netherlands with comparable conditions. Comparable conditions are:

- the merge is an active bottleneck;
- the highway has the layout of a two-lane highway with a speed limit of 130 km/h;
- the vehicle distributions and driver behaviour are comparable.

At first, merges are often an active bottleneck (Van Toorenburg, 1988; VID, 2012). Two-lane highways are also common. Speed limits however differ very often (Schultz van Haegen, 2012). The vehicle distributions and driver behaviour can also differ per location (see a.o. ARCADIS & Bureau Onderweg, 2011).

3.5 Summary

This chapter evaluated the study area, available data sources, validity of the used data sources and external validity.

The study area Hengelo-Zuid consists of the left-side of the highway A35, which is a two-lane carriageway heading for the directions Almelo and Amsterdam. The highway access is equipped with a short continuous line, implemented for safety reasons. A ramp meter is installed, which however does not function due to an error.

Detection loop data from the A35 and video measurements are used to evaluate the highway access at Hengelo-Zuid. The downtime of some detection loops at Hengelo-Zuid is relatively large, which makes data unreliable during 50% of the time. It's true that the reliability of the data is indicated by the dataset, though the high downtime makes analysis more difficult. Furthermore, data from adjacent road network and a regional traffic model are used. Weather data is used to select traffic data under which ideal weather conditions apply.

The data from detection loops and video measurements are compared and are found to be valid. The external validity of the location is however questionable due to many speed limits and different vehicle distributions and driver behaviour per location.

4. Situation analysis

This chapter describes the traffic situation in the study area. The first section describes the selection criteria for data analysis. The highway capacity is determined in Section 4.2. Section 4.3 describes the traffic flow characteristics at highway and the on-ramp. The next section describes other relevant traffic characteristics. Sections 4.5 and 4.6 contain an analysis of the upstream and downstream conditions (adjacent road network and wider network) of the study area. The chapter ends with a summary about the traffic situation and performance of the study area.

4.1 Selection criteria for data analysis

The merge at the R35 highway access Hengelo-Zuid is a bottleneck for the traffic flow on both the A35 and adjacent roads. This bottleneck mainly occurs during evening peak hours. This section describes the selection criteria for analysing data from this bottleneck.

The effect of the continuous line is studied for traffic conditions where the demand almost reaches the capacity, or in other words: in traffic conditions with a high risk for congestion, but without actual congestion. The risk for congestion is related with the capacity distribution, as described in Section 2.1.1. Risk for congestion basically occurs if the traffic flow exceeds the discharge capacity. For this analysis, situations with a 5-minute average flow higher than the discharge capacity form the first criterion (see Equation 15). The 5-minute interval excludes coincidentally high minute-flows from the analysis.

$$u_1 = \{ \bar{q}_{up,5\min} + q_{on,5\min} \geq C_{Discharge} \} \tag{15}$$

There is a grey area between congested conditions and free flow conditions. Therefore both conditions are distinguished in this research. Congestion is defined as minute-average speeds lower than 40 km/h (similar to the definition in the handbook CIA). The free flow conditions are defined as minute-average speeds higher than 70 km/h. At this speed, the flow is at least not congested, and includes slower driving heavy vehicles. The free flow and congested conditions are shown in Equations 16 and 17. Since congestion must be prevented with the continuous line, only free flow conditions are analysed. Equation 17 forms thus the second criterion for data analysis.

$$u_{Congestion} = \{ \bar{v}_{minute} < 40\, km/h \} \tag{16}$$

$$u_{Free\ flow} = u_2 = \{ \bar{v}_{minute} > 70\, km/h \} \tag{17}$$

Furthermore, only situations within peak hours are analysed. The traffic composition can during peak hours namely be different than outside peak hours. The amount of daily traffic and home-work traffic is much higher (Van der Kuil, 2012). Analysing only peak hour periods provides more certainty about equal traffic behaviour. Since the merge at Hengelo-Zuid mainly causes problems during evening peak hours, only the evening peak hours are considered in this research. Therefore, Equation 18 forms the third criterion for data analysis.

$$u_3 = \{ 16:00 > t > 19:00 \} \tag{18}$$

The criteria for data analysis is called u_{Line} and expressed in Equation 19.

$$u_{Line} = \begin{Bmatrix} q_{up,5\min} + q_{on,5\min} \geq C_{Discharge} \\ v_{minute} > 70\ km/h \\ 16:00 < t < 19:00 \end{Bmatrix} \tag{19}$$

Only data that meet these criteria is used to analyse the current traffic situation and evaluate the effect of the continuous line.

4.2 Capacity analysis

The capacity of the road is an important indicator for the current highway performance. Both the free capacity and discharge capacity are analysed. It may change after a continuous line is implemented. The difference between the discharge and free capacity can show the importance of postponing congestion.

The capacity of the study area can be determined multiple ways. It can be calculated empirically (using the detection loop data) and using literature (the Handbook CIA).

4.2.1 Capacity determination with literature

The capacity of the highway according to the Handbook CIA is 4200 vehicles/hour for 15% heavy vehicles (DVS, 2011). Heavy vehicles are defined as all vehicles longer than six meter with an equal distribution of light and heavy trucks. Compared with the categories, this is equal with category 2 and 3 together (see Section 2.1.1). The amount of heavy vehicles at Hengelo-Zuid is 5% (see Section 5.4). The reduction factor to 5% heavy vehicles is 1.10 (considering a pcu value of 2.0). The capacity of the road is then 4620 veh/h. This capacity is the free capacity, and is defined as the capacity measured just before congestion occurs.

The capacity expressed in passenger car units per hour is 4830 pcu/h. Per lane, the capacity is then 2415 veh/h.

4.2.2 Empirical capacity determination

The Handbook CIA explains a method to determine the free capacity, which is the Brilon method (see Section 2.1.1). Here, an observation of the capacity is the 5-minute downstream intensity in the interval before the interval that congestion occurs. Congestion is defined as a speed detection below a certain threshold at an upstream detector (usually 500 meter upstream). This could cause a delay in congestion occurrence and congestion detection, dependent on the relative shock wave speed. If this speed is low, the delay between congestion occurrence and detection is relatively big, which makes the Brilon method unreliable. In addition to that, the detection loops at Hengelo-Zuid have a high downtime (see Section 3.2.1) which would make the sample size too low.

Therefore, the highest 5-minute intensity measured during u_{Line} is used as a capacity observation. The advantage is that always the highest intensity is chosen per day. Though, if no congestion occurs, the capacity can be underestimated because the intensity possibly can increase further. Latter effect is probably small, since congestion almost daily occurs.

The 5-minute interval is chosen because: 1) this interval is prescribed in the Handbook CIA; 2) larger intervals do not include structural variances in the demand (see Section 4.3); 3) smaller intervals include too much coincidental variances in the demand; and 4) there is too much data missing to provide sufficient reliable samples of larger intervals.

It is not possible to gather direct loop detection data from the highway. Data from the next highway access (28. Delden) must be used (detection loops 57.895 HR + 58.380 VW c, see Appendix II.). These detectors are located three kilometres downstream of the merge. It takes about 1.5 minutes to get there (considering a speed of 120 km/h). A vehicle with 90 km/h arrives 30 seconds later. For measuring the highest intensity, this makes no difference. But for detecting the discharge capacity, this difference should be taken into account. Since the interval is five minutes, the effect of this delay is only small.

The free capacity is measured as the median of maximum observed 5-minute intensities per working day (Mondays – Thursdays). The 15-minute average is added as comparison. The discharge capacity is the median of 5-minute intensities downstream if congestion upstream is detected.

The observed capacity downstream of Hengelo-Zuid is estimated to be:

- Observed free capacity: $\bar{I}_{5\min}=5016\,veh/h$, $S_I=293\,veh/h$, $n=35$.
- Observed free capacity over a 15 minute interval: $\bar{I}_{15\min}=4720\,veh/h$, $S_I=304\,veh/h$, $n=35$.
- Discharge capacity: $\bar{I}_{5\min}=4080\,veh/h$, $S_I=327\,veh/h$, $n=43$.

The capacity distributions are shown in Figure 4.1.

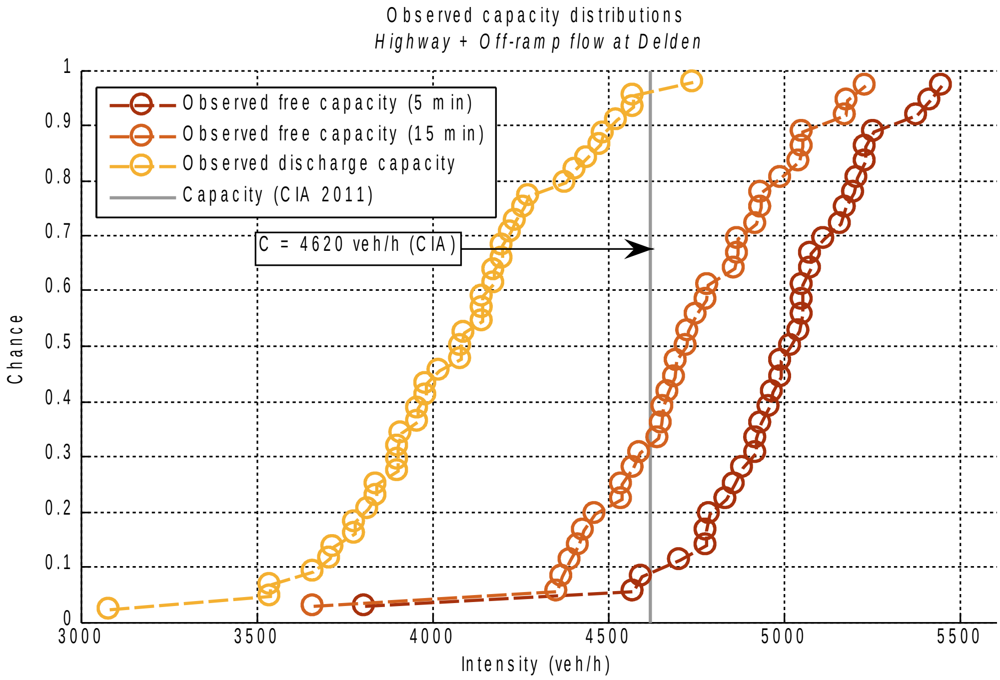


Figure 4.1: Observed capacity distributions

The figure shows that the observed 5-minute capacity is significant higher than the prescribed capacity by the Handbook CIA. The difference is approximately 9%. The following factors can declare this difference:

1. Local traffic conditions are at every location different, as described in Section 2.1.1. These conditions can influence capacity values.

2. The capacity in the Handbook CIA is calculated with the simulation software Fosim (DVS, 2011). Studies from Grontmij (2009) showed that the empirically measured capacity differs up to 10% from the modelled capacity with Fosim. The average difference is approximately 2%. According to this research, the observed capacity distribution is plausible.

3. If the downstream intensity is constant, a high on-ramp flow (1664 veh/h) is coupled with a relatively low upstream flow. This means that disruptions at the merge, and thus backward shock waves at the highway, are less likely to occur. The highway flow can then still increase before congestion occurs, which could thus be a reason for a capacity increase. Looking at the Newell-Daganzo model (Figure 2.2), this implies that the capacity for high on-ramp flows may slightly increase, which thus hypothesises a minimum capacity value (expressed in on-ramp / highway flow ratio).

4. Observations of Loot (2009) made it plausible that relaxation occurs up to two kilometres downstream of the merge. In the study area, the next off-ramp is three kilometres downstream. Since only little research is done to the relaxation phenomenon, it may be that relaxation plays a role in the high capacity observation. Though, Appert-Rolland & Du Boisberranger (2011) observed only relaxation phenomenons up to 400 meters downstream.

The figure shows also a large capacity drop, namely approximately 19%. According to the theory, these values commonly occur. Leclercq et al. (2011) listed literature with observed capacity drops at merges between 10% and 30%. Hereby, merges are an active bottleneck. The large capacity drop indicates that it is important to postpone congestion and provide a flow higher than the discharge capacity. Though, this capacity drop is measured during ideal (weather) circumstances. During rain, the capacity drop could be lower.

The difference between the 15-minute-average and 5-minute-average capacity distribution is as expected. Figure 4.2 shows an example for how the difference can be explained. Here, the 15-minute central moving average does not show variances in the 5-minute demand. For this sample, the maximum observed 15-minute intensity is about 10% lower than the maximum observed 5-minute intensity. The trend in Figure 4.2 is not structural, though it shows the difference of both intervals.

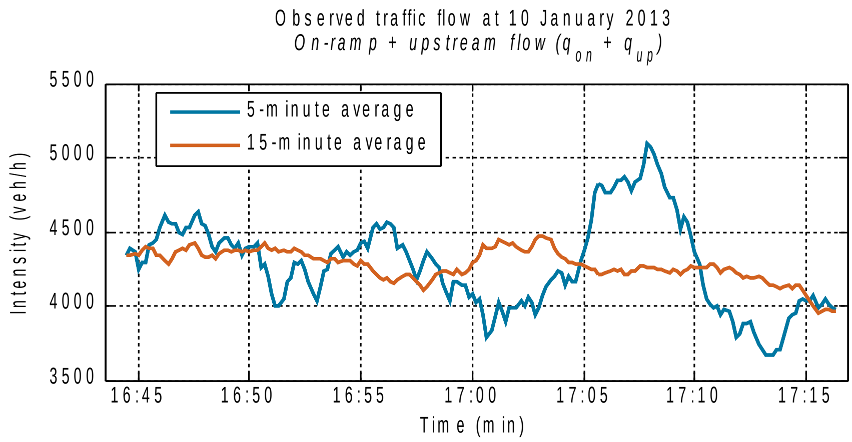


Figure 4.2: Example of a traffic flow in an evening peak hour: q_{down} (= q_{up} + q_{on}, measured with video) at 10 January 2013 for 5- and 15-minute central moving averages.

With the available data, the capacity value is relatively easy to observe and analyse. Therefore, the capacity value is used as performance indicator in this research.

4.3 Traffic flows

This section describes the flow characteristics at the on-ramp and highway during free flow and congested conditions.

4.3.1 Average flows and variances

The average traffic flow is shown in Figure 4.3. This figure shows the average traffic demand of samples from multiple days at the Hengelo-Zuid before the merge (q_{up}). Due to a relative low sample size, the 15-minute central moving average shows the average traffic flow the most fluently.

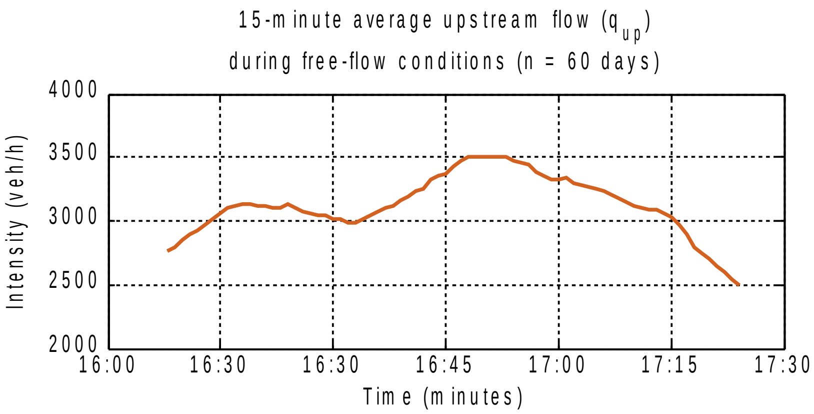


Figure 4.3: Average traffic flow at Hengelo-Zuid during free flow conditions

However, the situation is different per day. An example of a daily traffic flow is shown in Figure 4.4. The figure shows the 1-, 5- and 15-minute intensities. The minute intensities show the fluctuations in the traffic flow. The 5-minute intensities are shown because these values are also used in the capacity distributions. The 15-minute intensities show the central moving average of the traffic flow. With the central moving average, the variance in the traffic flow is calculated.

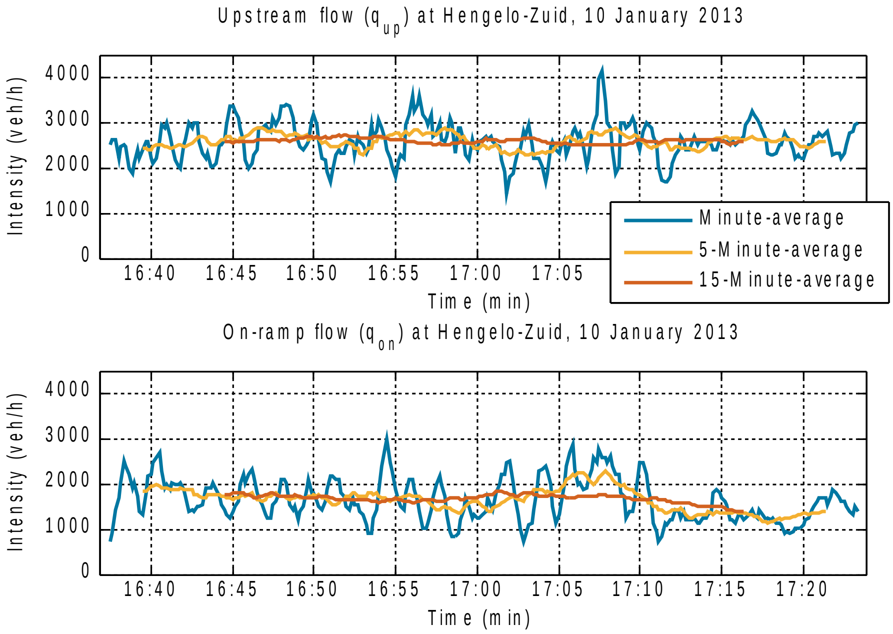


Figure 4.4: Average highway free flow

In the figure, congestion occurs at approximately 17:10. The 15-minute average seems a good indicator for the central moving average because the line is relatively fluent. The 5-minute average has still relative large fluctuations. There are large fluctuations in the minute-flows and can therefore be used for determining the variance in the traffic flow. The percentage heavy vehicles during u_{Line} is on average 5%: 3% category 2 vehicles and 2% category 3 vehicles.

Table 4.1 shows the average highway traffic flow during u_{Line}. The standard deviation is calculated as deviation of the minute-intensity subject to the central moving average (which is the 15-minute intensity trend line). Situations with and without congestion are distinguished, both in the minute-average and in the central moving average.

Table 4.1: Observed traffic flows at Hengelo-Zuid (before the merge)

Flow	*Quantity*	*Value (veh/h)*	*n*	*Source*
Upstream Free flow (q_{up})	Intensity	3156	921	Detection loop HR L 61.095
	Variance	483^2 *	569	
Upstream Congested flow (q_{up})	Intensity	2639	96	Detection loop HR L 61.095
	Variance	363^2	96	
	Intensity	2605	12	Video measurements 10 January 2013
	Variance	237^2	12	
On-ramp Free flow (q_{on})	Intensity	1664	30	Video measurements 10 January 2013
	Variance	515^2	30	
On-ramp Congested flow (q_{on})	Intensity	1345	12	Video measurements 10 January 2013
	Variance	227^2	12	

** Compared to the central moving average*

The deviation in the minute-intensities should, according to the literature in Section 2.1.4, lay between $s^2 = m$ and $s^2 = 0$. Since the minute-flow for a flow of 3156 veh/h is 53 veh/min, the value of s^2 clearly approaches 0.

The congested on-ramp flow plus congested upstream flow together should be equal to the discharge capacity. This can be tested with a T-test (Appendix I.). The congested on-ramp flow plus congested upstream flow together seem to match with the discharge capacity (within a 95% confidence interval).

4.3.2 Congestion

Highway congestion is a daily phenomenon at Mondays until Thursdays. Source data clearly shows that congestion usually starts between 17:00 and 17:15, with an average duration of approximately 30 minutes. The average speed during congestion is 37 km/h with a variance of 11^2 km/h (measured as minute-average of 96 minutes). Assuming a capacity of 5016 veh/h and an on-ramp flow of 1664 veh/h, the estimated upstream highway intensity (q_{up}) at which congestion starts is 3352 veh/h. Video measurements show that congestion at the highway starts at the same time as on-ramp congestion.

The fact that the congested highway flow (before the merge) is approximately 2639 veh/h, means that at upstream intensities higher than 2639 veh/h cause backward shock waves during congestion. Congestion only resolves if the upstream flow is during a longer period lower than 2639 veh/h. Looking at the average highway flow in Figure 4.3, the intensity is on average during the whole period between 16:30 and 17:15 higher than 2639 veh/h. Thus, if congestion occurs, congestion grows fast. This means that it takes some time before congestion is resolved.

> ***Example:*** If congestion starts at 17:00 at an upstream intensity of 3352 veh/h, the backward shock wave speed is 19 km/h (considering a congested flow of 2639 veh/h and speed of 37 km/h). If the upstream demand decreases linear to 2639 veh/h at 17:20, the maximum length of congestion is approximately 3 km. The congestion will be resolved after 40 minutes. The maximum delay on the highway is approximately 3 minutes (considering a free-flow speed of 100 km/h).

The observed congestion patterns can eventually be used to estimate the effect of the continuous line in travel time delay.

4.4 Other traffic characteristics

Literature showed the importance of headway distributions, lane distributions and lane change behaviour in this research. Observations show this behaviour in the study area.

4.4.1 Headways

An important aspect in the research are the headways. The headways according to the theory are calculated with the Pearson type III equation (Section 2.2.1), and shown in Figure 4.5.

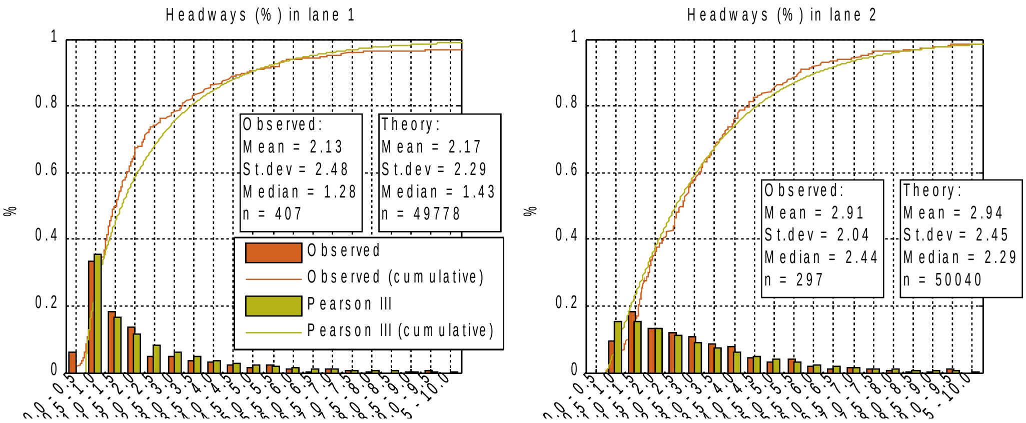


Figure 4.5: Observed headways compared to the theory

The figure shows that the sample of the headways corresponds with the theory. This indicates that no exceptional traffic behaviour can be observed in the headway distributions.

4.4.2 Lane distribution and cooperative lane change

The intended effect of the continuous line is to shift more vehicles from lane two to lane one. This lane changing behaviour already (partly) occurs due to cooperative lane changing manoeuvres, which is described in the literature in Section 2.3. The lane distributions and (cooperative) lane changing can also be observed in the source data.

The video observations showed the lane distribution within a range of 253 and 190 meters before the merging point. Since the distance to the merge is quite short, the difference between the lane distributions of two points in the video observations can be an indication of cooperative lane changing.

The detection loop is located 500 meters before the merging point. For only one video observation period, data from the detection loop is also available. The lane distribution at the location of the loop can thus be calculated for the same periods. Now an indicator of the cooperative lane change is gathered for a range between 500 and 190 meters before the merging point.

The lane distributions per minute at the three mentioned locations are shown and compared in Figure 4.6 and in Table 4.2.

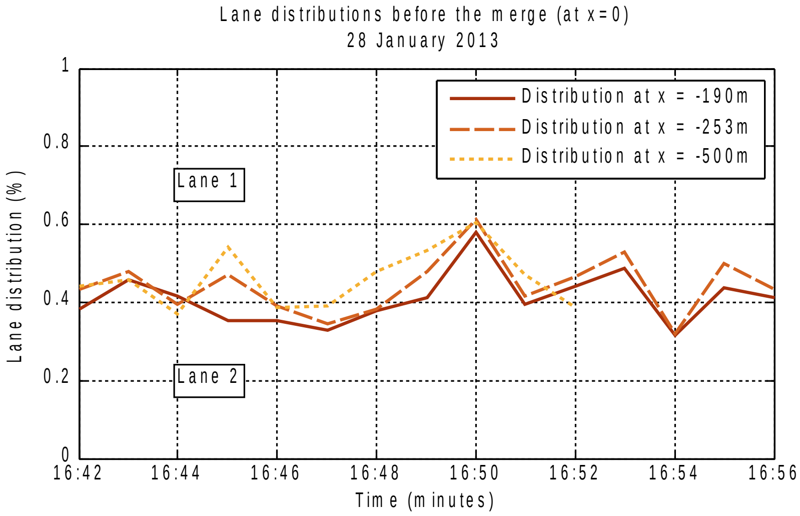


Figure 4.6: Lane distributions per minute compared at three successive locations

The increase of the number of vehicles at lane one can also be observed. The yellow dotted line (from x = -500 m) is on average higher than the straight lines (at x = -253 and x = -190 m). This makes the cooperative lane change manoeuvres more clear.

The figures in Table 4.2 show the sight of the drivers at each measurement location, which makes the lane change behaviour more understandable.

Table 4.2: Lane distributions between 28 January 2013 16:42 and 16:52 compared.

Lane	***Detection loop data***			***Video dataset 2***					
	500 m			***253 m***			***190 m***		
Lane 2	228	(1244 veh/h)	44%	222	(1211 veh/h)	42%	205	(1118 veh/h)	39%
Lane 1	295	(1609 veh/h)	56%	302	(1647 veh/h)	58%	317	(1729 veh/h)	61%
Total	523	(2853 veh/h)	100%	524	(2858 veh/h)	100%	522	(2847 veh/h)	100%
View	 *Figure 4.7: View 500 m before merging point**			 *Figure 4.8: View 253m before merging point**			 *Figure 4.9: View 190 before merging point**		

* *Source: Google* (2013)

The continuous line continued...

4.5 Adjacent road network

An increased highway performance should provide an increased on-ramp flow, and thus eventually an increased throughput at the adjacent network. This section provides a situation analysis of the adjacent road network, such that a reference is created for the effect of the continuous line.

Figure 4.10 shows the adjacent road network and the estimated two-hour intensities from the traffic that uses the on-ramp in that period. The figure is a selected link analysis from the regional traffic model of the Twente region (Goudappel Coffeng, 2013). The figure shows that most traffic to the A35 comes from the Diamantstraat and Haaksbergerstraat. Most delay is experienced here.

Figure 4.10: Origin traffic at on-ramp over a 2-hour interval. (Goudappel Coffeng, 2013)

With the data from the regional traffic model, traffic light log files and traffic counts is observed that a large amount of traffic does not head for the A35. A large traffic flow is coming from Hengelo for example, though only a fraction of this traffic flow heads for the A35. Congested patterns at the adjacent road network could hinder traffic flow that does not head for the A35. It is beyond the scope of this research to quantify this effect, though it is important to notice that congestion due to the merge affects multiple traffic flows at the adjacent road network.

Section 4.3.2 described that congestion normally starts between 17:00 and 17:15, and takes approximately 30 minutes. During this period, only 1345 veh/h can enter the highway instead of 1664 veh/h. This is a reduction of 160 vehicles, and results in a delay up to 6 minutes.

4.6 Network approach

The weakest link in the network determines its capacity. Increasing the capacity at Hengelo Zuid is only useful if the network downstream can process this flow. This aspect is elaborated in this section.

Figure 4.11 shows the current A35 traffic situation, and Figure 4.12 shows the dispersion of traffic that enters the on-ramp at Hengelo-Zuid (1 in the figure).

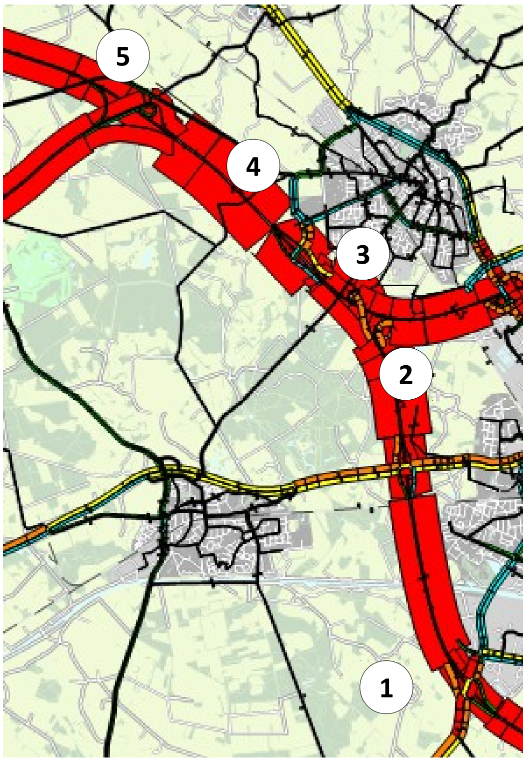


Figure 4.11: A35 traffic flow 2012 (Goudappel Coffeng, 2013)

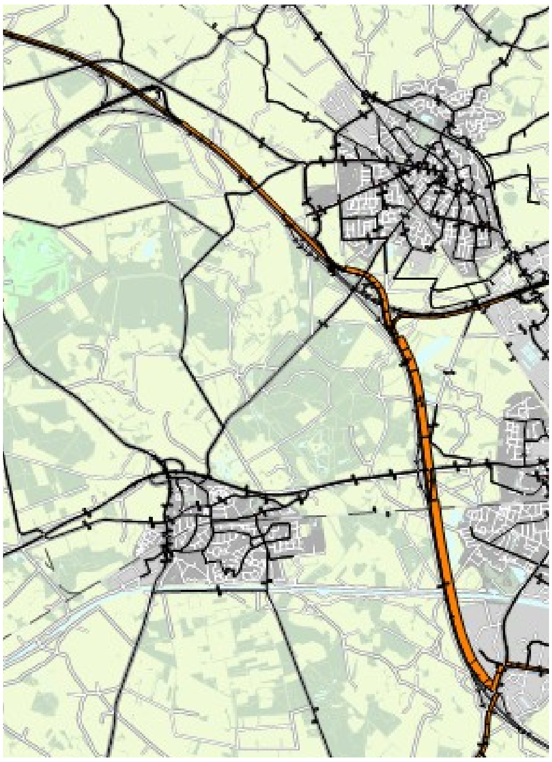


Figure 4.12: Dispersion of traffic from access Hengelo-Zuid 2012 (Goudappel Coffeng, 2013)

The figures above show potential bottlenecks at the A1/A35:

1. the on-ramp Hengelo-Zuid
2. the weaving section between Delden and Buren;
3. the sharp corner at interchange Buren;
4. the tapered merge and weaving section with RW1; and
5. intersection Azelo.

To determine the load on the network downstream, the origin destination should be known. The origin-destination pattern of the traffic from Hengelo Zuid is estimated for 2012 and 2020 with the Regio Twente Model (Goudappel Coffeng, 2013). The maximum extra flow that can enter the highway is 319 vehicles per hour (which is the difference between a free on-ramp flow and a congested on-ramp flow).

Table 4.3 shows the I/C ratio's for bottlenecks at the A35 for 2012 and 2020 for the current situation (reference), and for the situation with an improved on-ramp flow. The flows are the average flows during the busiest evening peak hour. The data is gathered from (DVS, 2011; Goudappel Coffeng, 2013; Rijkswaterstaat, 2012, 2013).

Table 4.3: Estimated I/C ratio's for bottlenecks at the A35 for 2012 and 2020, in the reference situation (Ref) and with an improved on-ramp flow (*).

Bottleneck location	*Capacity (veh/h)*	*Flow 2012*	*Flow 2020*	*O/D (2012)*	*O/D (2020)*	*I/C Ref 2012*	*I/C Ref 2020*	*I/C * 2012*	*I/C * 2020*
1. Hengelo-Zuid	4620	4350	+9%	100%	100%	0.94	1+	1+	1+
2. Weaving section	6149	4450	+5%	90%	89%	0.72	0.84	0.84	0.89
3. Corner Buren	3927	3167	+7%	58%	61%	0.81	0.86	0.85	0.91
4. Weaving section	5590	4874	+12%	43%	45%	0.87	0.98	0.90	1+
5. Interchange Azelo	4200	3218	+14%	31%	32%	0.77	0.87	0.79	0.90

The table shows that Hengelo-Zuid is an active bottleneck at the A35. Increasing the capacity here increases thus the capacity of the whole wider network. Though, the I/C ratios at the weaving section A1/A35 are also relatively high. Increasing capacity at Hengelo-Zuid enlarges thus the congestion probability at this section slightly.

4.7 Summary

Literature showed the importance of macroscopic traffic characteristics (capacity distributions, intensities and variances) and microscopic traffic characteristics (headway distributions, lane changing behaviour and lane distributions). This chapter aimed to evaluate these traffic characteristics in the study area with observed data. This resulted in an overview of the situation and performance indicators.

The capacity at the A35 highway access Hengelo-Zuid is estimated to be 5016 veh/h. This value is relatively high, though plausible. The discharge capacity is 4080 vehicles per hour. This indicates a capacity drop of almost 19%, which shows the importance decreasing the congestion probability. At the highway, congestion occurs on average at an upstream flow q_{up} of 3352 veh/h. During congestion, the on-ramp flow q_{on} decreases from 1664 veh/h to 1345 veh/h.

Observed headway distributions match with the theory. Cooperative lane changing behaviour is observed, which influences the lane distribution ratio to 39% at lane two and 61% at lane one.

Congestion starts usually between 17:00 and 17:15, and lasts for approximately 30 minutes. The average delay at the highway is estimated to be 3 minutes. The extra waiting time at the adjacent road network is maximum 6 minutes. Most delay is experienced at the Haaksbergerstraat and Diamantstraat. The bottleneck at Hengelo-Zuid is the largest bottleneck within the wider network of the A35. Increasing the throughput here has no direct consequences for the traffic situation downstream.

In this research, capacity is used as the performance indicator. The capacity value is relatively easy to observe. Estimations of travel time delays can eventually be used to translate the increase of capacity into an indicator which is better understandable for the end user.

5. MODEL CALIBRATION AND VALIDATION

The effect of the continuous line is evaluated with a traffic simulation model. This chapter introduces the modelling process. The first section underpins the choice for the traffic simulation model, followed by a section that describes the calibration method. The third section shows the design of the simulation, followed by sections that describe the calibration process. Sections 5.8 and 5.9 show the validation process. The chapter ends with a summary about the reliability and constraints of the model.

5.1 Traffic simulation model

5.1.1 Aim of the simulation

The effect of the continuous line is determined using a traffic simulation model. The model must be calibrated and validated to represent the traffic flow characteristics as good as possible, and to improve the correctness of capacity calculations. After the calibration and validation, a pronouncement about the reliability of the model can be made. The eventual aims are 1) to estimate whether the line has a significant effect or not; and 2) to perform a qualitative pronouncement about the effect of the line.

5.1.2 Criteria for traffic simulation software

The traffic simulation should be able to simulate the current traffic situation, and simulate the effect of the continuous line. Several criteria can be set up for choosing traffic simulation software.

The model:

1. must be able to simulate the Dutch highway on-ramp geometry;
2. must be able to simulate the continuous line;
3. must be able to determine capacity distributions;
4. should be microscopic (such that the model can be calibrated with observed data), and be able to simulate the following quantities representatively:
   a) headway distributions;
   b) lane distributions;
   c) (cooperative) lane changing;
5. should be easy to use and low in costs due to the relative short lead time of the research.

Studies from Grontmij (2002) evaluated 9 traffic simulation models which are considered to be suitable for local analyses and goal-oriented research. These models are matched to the criteria in Appendix III. This matching is done according to studies from Grontmij (2002), trial software and personal communication with model experts (Van Velzen & De Jong, 2012).

The traffic simulation software Fosim (acronym for Freeway Operations SIMulation) came out best with above criteria. In addition to that, most capacity studies in The Netherlands are conducted with Fosim (DVS, 2011). Figure 5.1 shows an example of Fosim.

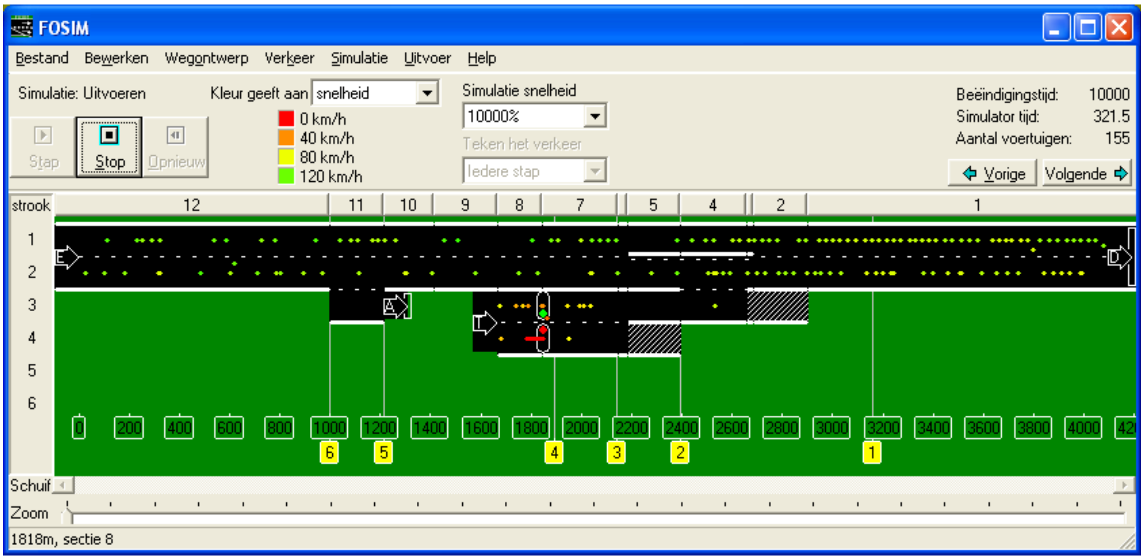


Figure 5.1: Example layout of Fosim

5.1.3 Description of traffic simulation software

Fosim is a microscopic simulation model, specific designed and calibrated for Dutch highways. It simulates the behaviour of individual drivers (Dijker & Knoppers, 2006). During the development of Fosim, emphasise was laid on estimating capacities.

Basically, the driving behaviour in Fosim is based on:

- a desired speed of drivers;
- attempts to change lane if drivers are not able to drive their desired speed;
- a desired headway of drivers, obtained from speed adjustments if lane changing is not possible;
- mandatory lane changes for a driver to reach their destiny.

Simulating congested conditions with Fosim should be done with some caution. Furthermore, Fosim assumes ideal circumstances during simulations (dry and daylight) and considers a maximum speed of 120 km/h (Dijker & Knoppers, 2006). Cooperative lane changing is not implemented in the model. However, it appears to be that the lane distribution at merge already includes the effect of cooperative lane changing. This effect is elaborated further in this chapter.

Calibrating Fosim

Fosim is calibrated for 5 different vehicle types, which can manually be increased up to 7. Each vehicle type has its own parameters (called 'vehicle parameters', see Appendix IV.I). The vehicle parameters include for example a desired speed, car following parameters and acceleration parameters. Other input values for Fosim are the geometry, vehicle distributions, the intensity pattern over time, and an origin-destination matrix (Dijker & Knoppers, 2006).

Usually, Fosim only needs to be calibrated by adjusting vehicle distributions. If these settings are represented correctly, Fosim usually provides reliable results. If not, the road layout and traffic load can be adjusted. Changing vehicle parameters is only an option at worst (Dijker & Knoppers, 2006).

5.2 Calibration method

For the calibration process, a balance should be found within three constraints: 1) the goal of the simulation, 2) possibilities of the model, and 3) availability of source data for reference.

1. The goal of the simulation is to model the effect of the continuous line as good as possible. For this, the microscopic lateral flow characteristics (lane changing behaviour and vehicle interactions) are the most important aspects. Next to these, the microscopic longitudinal traffic characteristics (headway distribution) and macroscopic traffic characteristics (flow, speed, and lane distributions) are important.

2. The model provides no options to influence lane change behaviour and vehicle interactions directly, or the developers advice against this. Variables that initially should be adjusted for calibration are vehicle distributions, intensities, road geometry, and origin-destination matrix.

3. The source data delivers intensities and variances for each traffic flow, headway distributions per lane, capacities, percentages heavy vehicles, and derivatives of this information, such as a fundamental q/v diagram.

An assessment is made between above three constraints, fixing on the aim of the simulation which resulted in the following method. The model is calibrated with source data for the traffic conditions during u_{Line}. The calibration is done with parameters and indicators. Parameters are the input for the simulation. Indicators are the output of the simulation. The calibration and validation process is shown in Figure 5.2.

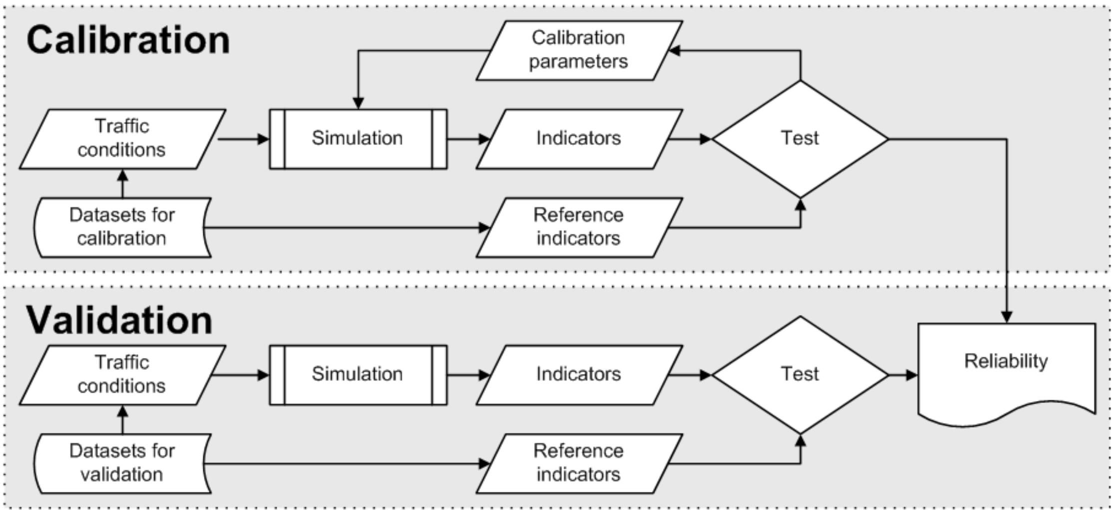


Figure 5.2: Calibration and validation process

In the end, the model is validated. The calibration and validation is done with different datasets.

- The calibration is done with detection loop data from September – November 2012 and the video observations from 10 January 2013.
- The validation is done with data from January – March 2013 and the video observations from 28 January 2013.

5.2.1 Traffic conditions

The traffic conditions are gathered directly from the data sources, and are:

1. vehicle distributions for heavy vehicles;
2. minute-intensities upstream (q_{up});
3. intensity at the off-ramp (q_{off}).

The traffic conditions are described in Section 5.4.

5.2.2 Calibration indicators

Calibration indicators are indicators which are gathered from the source data, which must be adjusted before entering them into the model. The model provides eventually the calibration indicators as output, and the indicators from the model and source data are compared. The calibration parameters are adjusted iteratively until the indicators from the model and source data correspond sufficiently.

The calibration (and validation) process uses the following indicators:

1. intensities and variances of the highway flow (q_{up});
2. intensities and variances of the on-ramp flow (q_{on});
3. headway distributions;
4. the fundamental q/v diagram.

The calibration indicators are described in Section 5.5.

5.2.3 Calibration parameters

Calibration parameters are parameters in the model, which must be adjusted such that the model matches with the calibration indicators. Some parameters are different for each model, and other parameters are the same for each model. For example, the headway model has other vehicle distributions and other intensities than the q/v model. The calibration parameters are listed below and described in Section 5.6.

1. Vehicle parameters (desired speed per vehicle category)
2. Length of the first section in the model (the location where the simulation begins)
3. Intensity raise factor
4. Traffic light settings
5. Vehicle distributions per vehicle type.

Appendix IV. shows the settings of the model, including input and output, into more detail.

5.3 Simulation design

The layout of Hengelo-Zuid is rebuilt in the traffic simulation model. The dimensions of the study area are measured and rounded off in meters. The start of the model is a variable, which is described in Section 5.6.2. The off-ramp is included since this can influence the lane distribution. See Figure 5.3.

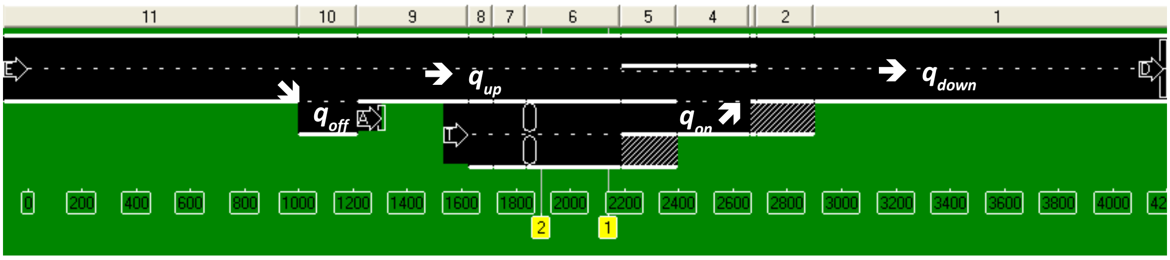


Figure 5.3: Design of the simulation in Fosim. Location (1) represents the video measurement location, and location (2) represents detection loop HR 61.095 L.

Four indicators are used. The source data must be selected carefully to quantify these indicators:

1. The **fundamental q/v diagram** requires low and high intensities. Otherwise, the fundamental q/v diagram can not be created. Therefore, data from whole evening peak periods are used as reference, according to $u_{qv} \in \{16:00 < t < 19:00\}$. Source data from the detection loop RW35 HR 61.095 L (500 meters before the merging point) is used.
2. The quantification of **highway and on-ramp flows** requires only data during u_{Line} . The sample size must be representative, and therefore source data from the detection loop RW35 HR L 61.095 (500 meters before the merging point) is used.
3. The **headway distributions** require data during u_{Line} . The headway distributions are measured with the video observations. The video observations provide thus the input for the calculation of headway distributions.

   The simulations stopped after congestion is detected at the measurement location in the model. The traffic flow during congested periods gives namely a wrong image of the headways.

Above is summarised in Table 5.1. Furthermore, the simulation is based on the Monte-Carlo method. The number of simulation runs is 50. Each simulation run has slightly different starting conditions which corresponds to daily and coincidentally differences in the traffic flow.

Table 5.1: Selection criteria and sources for quantification of the indicators

Indicator	*Data selection criteria*	*Source*
Fundamental q/v diagram	$u_{qv} \in \{16:00 < t < 19:00\}$	Detection loop
Highway flow	u_{Line}	Detection loop
On-ramp flow		
Headway distributions	u_{Line}	Video measurements

5.4 Traffic conditions

This section describes the traffic conditions, which form the input for the model.

5.4.1 Vehicle distributions for heavy vehicles

Vehicle distributions for heavy vehicles can be derived directly from the source data, and be entered in the traffic simulation model.

In the source data, the vehicle categories are classified according to Dutch standards (Geearts & Van Bergen). The distribution of these vehicles is calculated for u_{Line} and for free flow periods u_{FF} (Equations 19 and 20). This is necessary, because the fundamental q/v diagram indicator requires data from the free flow periods, and the other indicators require data from u_{Line}.

$$u_{FF} \in \begin{Bmatrix} 16:00 < t < 19:00 \\ u \neq u_{Line} \end{Bmatrix} \tag{20}$$

The vehicle categories are:

- Category 1 (CAT I): <5.6 meter
- Category 2 (CAT II): 5.6 – 12.2 meter
- Category 3 (CAT III): > 12.2 meter

The vehicle distributions are shown in Table 5.2.

Table 5.2: Vehicle distributions in the different data sources.

Source data	%HGV for u_{Line}	%HGV for u_{FF}
Detection loop (2012)	CAT II: 3% CAT III: 2%	CAT II: 5% CAT III: 3%
Detection loop (2013)	CAT II: 3% CAT III: 2%	CAT II: 5% CAT III: 3%
Video (10 jan)	CAT II: 2% CAT III: 3%	-
Video (28 jan)	CAT II: 3% CAT III: 2%	-

The table above shows that the amount of heavy vehicles is clearly larger for u_{FF}. A main reason for this, is that 1) local freight trips are planned outside the peak periods, and 2) traffic peak hours are mainly caused by home-work trips, which indicates a larger percentage of passenger cars.

5.4.2 Minute-intensities upstream (q_{up})

The minute intensities measured upstream (either with detection loop or video measurements) are used as input for the model. The intensities are however raised with an intensity raise factor, a calibration parameter which is described in Section 5.6.3.

5.4.3 Intensity at the off-ramp (q_{off})

The mean intensity at the off-ramp is for both u_{Line} and u_{FF} 10% from the total intensity at the highway (highway and off-ramp flow together). This value can be entered in the model.

5.5 Calibration indicators

This section describes the calibration indicators, which are derived from the model output.

5.5.1 Intensities and variances of the highway flow (q_{up})

To represent the highway traffic flow at a macroscopic scale, the intensities and variance from the highway flow are used as an indicator.

The observed values are shown in Table 5.3. The size of interval n for the detection loop data differs for the intensity and variance calculation. The variance is calculated subject to to the 15-minute central moving average. Thus, only successive intervals of at least 15 minutes could be used for this calculation.

Table 5.3: Intensities and standard deviations for the source data

Source data	*Main carriageway*		
	Mean	Variance	n
Detection loops 2012	2813	454^2	260 / 106
Detection loops 2013	2626	490^2	446 / 138
Video 10 January	2684	406^2	15
Video 28 January	2748	561^2	15

The minute-intensities and standard deviations from the source data eventually compare with those from the simulation.

5.5.2 Intensities and variances of the on-ramp flow (q_{on})

The intensities and variance from the on-ramp flow is used as an indicator to represent the on-ramp traffic flow at a macroscopic scale. The merge of on-ramp and highway flows together are eventually namely the cause of congestion.

The mean intensity at the on-ramp is estimated with the video measurements. The intensity here seems to be fluctuating due to the traffic lights. The intensities and variances of the on-ramp flow are shown in Table 5.4. The percentage of heavy vehicles is 4%.

Table 5.4: Intensities and variances for the video dataset.

Source data	*On-ramp*		
	Intensity	Variance	n
Video 10 January 2013	1664	545^2	30
Video 28 January 2013	1599	352^2	17

After analysing the data, it seems that there is a peak in the traffic demand every 2.5 - 3 minutes. The width of the peak is approximately 60 seconds. The pattern of the on-ramp flow is sketched and compared with the pattern of the on-ramp flow in the model. The chosen time step here is 10 seconds, which is sufficient to group individual vehicles, and also sufficient to recognize peaks in the traffic flow. The pattern is shown in Section 5.7.2.

5.5.3 Headway distributions

The headway distributions are an indicator for the microscopic longitudinal flow characteristics and macroscopic flow characteristics. The mean headway indicates the lane distribution of the traffic flow. The standard deviation and shape of the distributions are an indicator of the distribution of headways.

The video measurements are used to register headways per lane on two successive locations. With this data, the passing time, time headway, speed and vehicle length can be calculated. The minimum size of the sample is calculated with Equation 21:

$$n=\left(\frac{2\cdot z\cdot \sigma}{\omega}\right)^2 \tag{21}$$

where n is the minimum sample size, z is the right critical value for the confidence interval, σ is the standard deviation of the sample and w is the width of the confidence interval. Two datasets were gathered, both over a 15-minute interval. Both datasets contain about 700 headways, which is sufficient for a confidence interval of 95% (z = 1.96) and a confidence interval width w of 2 · 0.25 = 0.5.

The mean and standard deviations of the samples are eventually compared with those from the model using a T- and F-test. A Kolmogorov-Smirnoff-test (KS-test) is performed to evaluate the difference between the distributions (see Appendix I.).

Difference between headways

Not only the headway distribution is created, but also the distribution of the difference in two successive headways, which indicates the distribution of the headways on the road. The difference between headways indicates to what extent headways are randomly distributed, which can also be classified under the microscopic longitudinal traffic characteristics.

The difference between two successive headways is calculated with Equation 22.

$$\Delta h=h_n-h_{n-1} \tag{22}$$

The means and standard deviations of the difference between headways from the sample is eventually compared with those from the model using a T- and F-test. A KS-test is performed to evaluate the difference between the distributions.

5.5.4 Fundamental q/v diagram

The fundamental q/v diagram is used to calibrate the speeds and vehicle distributions in the model, which represent the macroscopic flow and speed characteristics and microscopic lateral flow characteristics. The diagram namely indicates the interaction between vehicle types at different flows and speeds. Because each vehicle type has an own desired speed, the shape of the q/v diagram depends on the speeds and the share of each vehicle type, and the model can be calibrated by adjusting these parameters.

The q/v diagram can only be made if the sample contains low and high intensities, which means that data from the whole peak period should be used. Section 5.4.1 showed that the percentage of heavy vehicles during u_{FF} (Equation 20) is different than during u_{Line} (Equation 19). In the model, only one value for heavy vehicles can be entered. The difference in vehicle distributions between u_{Line} and u_{FF} can thus not be modelled. This would result in a biased view in the q/v diagram, because the small and large trucks have a large influence in the speed. Therefore, only category 1 vehicles are considered in the fundamental q/v diagram. This minimises the effect of small and large trucks on the speed in the diagram.

The percentage heavy vehicles however must be entered in the model, and are the same as during u_{Line}. The assumption is made here that the speeds of category 1 vehicles outside u_{Line} does not depend on the presence of heavy vehicles. This is plausible, since the intensities are then lower and there is sufficient space for overtaking.

The fundamental q/v diagram is thus set up with minute-intensities of category 1 vehicles, which is shown in Figure 5.6. The selected time period is $u_{qv} = \{16:00 < t < 19:00\}$. The harmonic mean speed is calculated between the arithmetic speeds of the two lanes, until I_{min} = 3000 veh/h. Then, linear regression is applied using the least square fit method, assuming that the fundamental q/v diagram is linear until at least 3000 vehicles/hour. Outliers $e > 3 \cdot s$ are filtered, which matches with a confidence interval of 99%.

5.6 Calibration parameters

This section describes the used calibration parameters, which are adjusted iteratively.

5.6.1 Vehicle parameters

The vehicle parameters in the model contain settings about the desired speeds of each vehicle type. The standard settings in the model contain five different vehicle types. This can be expanded up to seven. The desired speeds must be calibrated, such that the reference and model indicators match.

The model is designed for a maximum speed limit of 120 km/h. Since September 2012, the maximum speed at the study area is 130 km/h. The speeds in the model must thus be changed. Recent studies showed that the average speed on highways where the speed limit was raised from 120 km/h until 130 km/h, increased with three km/h. The average speed limit for freight traffic did not increase (ARCADIS & Bureau Onderweg, 2011).

With the source data, the desired speed for each vehicle type is estimated. This is done by selecting minute-intensities under 1500 vehicles per hour, and calculating the (harmonic) mean speeds of the vehicles. After filtering outliers (larger than 2·S), the desired speed for category 3 vehicles (large trucks) is estimated to be 85 km/h, and for category 2 vehicles (small trucks) 98 km/h. The desired speed for category 1 vehicles is 117 km/h. Though, the modelling software considers multiple vehicle types as category 1 vehicles, which makes it hard to derive the desired speed for each vehicle type. These speeds must thus be calibrated.

The calibration for category 1 vehicles is done using the following assumptions:

1. a first group of drivers which aims to drive slightly harder than the speed limit;
2. a second group of drives which aims to drive slightly slower than the speed limit;
3. a third group of drivers aims to drive its own desired safe speed, regardless the speed limit.

This division causes large differences between the desired speeds, which was visible in the headway distributions. Therefore, vehicle types 6 and 7 are added (see Table 5.5). Vehicle types 2 and 6 together now form the second group, and vehicle type 7 is used to fill the large gap between vehicle types 2 and 3 (Table 5.5).

The mean speeds are chosen such, that the differences in desired speeds are more or less equally distributed, but also such that the calibration results match as good as possible with the reference frame. During the calibration the desired speed of large trucks was increased with 1km/h, such that an optimal calibration result was reached.

Table 5.5: Calibration results for desired speeds in the model

Vehicle type	*Desired speeds (km/h)*	
	Fosim standard settings (120 km/h)	Used settings (130 km/h)
1	125	135
2	115	120
3	100	102
4 (small trucks)	95	98
5 (large trucks)	85	86
6	-	128
7	-	110

5.6.2 Length of first section

The position from where the simulation begins has an influence on the traffic flow pattern in the model. This initial length is shown in Figure 5.4 and must be calibrated.

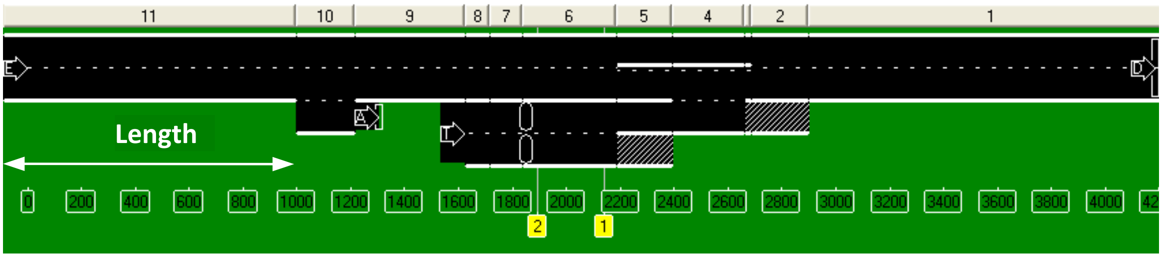


Figure 5.4: Section 11 in the model is a calibration parameter.

Increasing this length increases the standard deviation of the intensities, though it also increases the platooning (grouping) of vehicles, which disturbs the headway distribution. If the length is too short, the standard deviation of the intensities is too small. In addition to that, there must be sufficient space for vehicles to pre-sort for the off-ramp.

A first-section length of 1000 meters is advised by the Fosim support. This length is also required to provide sufficient length of the lane changing areas for the off-ramp. The length of 1000 meters is used in the model, though it does not provide the required variance in the traffic flow. Therefore, a variance in the traffic flow is used as input using the intensity raise factor (Section 5.6.3).

5.6.3 Intensity raise factor

The intensity raise factor is used to raise the variance in the intensities before the observed variance is entered in the model. This provides a better corresponding traffic flow in the model.

The intensity that is found with the detection loops is exclusive the off-ramp flow. Since the measured traffic flow is 90% from the original traffic flow (coming from Enschede), the intensities measured in the datasets should be divided by 0.9 to determine the input value.

Then the observed variance in the traffic flow must be increased such that the model produces a representative traffic flow. The variance in the traffic flow should be raised with an intensity raise factor F_{Raise}.

To calculate the headway distributions, the measured minute intensities from the video observations were used as input. The minute-intensities must therefore be multiplied with the intensity raise factor, according to Equation 23.

$$\{I_{Model\ In}\}=[I_{Model,1}\dots I_{Model,n}]=\frac{\bar{I}_{Video}}{0.9}+([I_{Video,1}\dots I_{Video,n}]-\bar{I}_{Video})\cdot F_{raise} \tag{23}$$

The calculation of the fundamental q/v diagram and intensities require detection loop data as input. Here, the minute-intensities are generated with a random generator, using the product of the standard deviation of the sample and the intensity raise factor as input. The precondition here is that the random generator generates the values such that the mean and standard deviation of the output are equal to that of the input, according to Equation 24.

$$\{I_{Model\ In}\}=[I_{Model,1}\dots I_{Model,n}]=\frac{\bar{I}_{Ref}}{0.9}+randn[1\dots n]\cdot S_{I_{Ref}}\cdot F_{Raise} \tag{24}$$

Where $\bar{I}_{Model,out}\approx\bar{I}_{Ref}$, and $S_{I_{Model,oud}}\approx S_{I_{Ref}}$.

The intensity raise factor turned out to be $F_{Raise}=1.5$.

5.6.4 Traffic light settings

The on-ramp traffic behaviour is simulated using traffic lights at the on-ramp. Two traffic lights are used. One traffic light simulates the large platoons with a 170 seconds interval. The other traffic light releases the vehicles more randomly. The settings are shown in Table 5.6, and were calibrated such that the mean intensity is about 1650 in situations without congestion.

Table 5.6: Traffic light settings at the on-ramp in the model

Lane	Traffic light settings			
	Green time	Amber time	Cycle time	Offset
Lane 3	12	2	15	0
Lane 4	70	3	170	0

The traffic flow from the model is compared with the observed traffic flow in Section 5.7.2.

5.6.5 Vehicle distributions

The vehicle distributions show the division of vehicles within the model, and are mainly calibrated with the fundamental q/v diagram. The percentage of large and small trucks is given by the datasets. For the other vehicle types, the values are not fixed and need to be calibrated. The model manual describes that, if the vehicle distributions are represented sufficient in the simulation, the assumption can be made that the model gives reliable results (Dijker & Knoppers, 2006).

Table 5.7 shows the result of the calibration of the vehicle distributions. The percentage heavy vehicles depends on the source data used, according to Section 5.4.1.

Table 5.7: Result vehicle distributions

Distribution per vehicle type (%)						
1	**6**	**2**	**7**	**3**	**4**	**5**
25%	15%	15%	25%	15%	Fixed	

5.7 Calibration results

This section shows the results of the calibration. The values of the indicators from the model and source data are compared.

5.7.1 Intensity and variance from the highway flow (q_{up})

The intensities and variances from the highway flow from the model are compared with the observed values. This comparison is done with the following hypotheses (Equations 27 and 28). The T-test is used from Equation 31, and the F-test is used from Equation 32.

The results of the tests are shown in Table 5.8. The intensities of both the intensity model and headway distribution model are tested. The intensity for the headway model is tested to check whether the model represents the observed data in a sufficient way. The intensity for the intensity model is tested to check whether the random generator is reliable for providing intensities. The random generator is also used for analysing the continuous line.

Table 5.8: Result calibration with an intensity raise factor of 1.5

Model	*Test*	*Reference*		*Model*		*Test value*	*Critical value*	*Accept H_0?*
		Value	**n**	**Value**	**m**			
Intensity model	Mean (T)	2813	260	2836	1705	$T_{DF}=0.65$	$T_{0.05}=1.64$	Yes
	Variance (F)	454^2	106	478^2	1705	$F_{n-1}^{m-1}=1.11$	$F_{0.05}=1.22$	Yes
Headway model	Mean (T)	2684	15	2682	658	$T_{DF}=0.02$	$T_{0.05}=1.64$	Yes
	Variance (F)	406^2	15	409^2	658	$F_{n-1}^{m-1}=1.01$	$F_{0.05}=1.67$	Yes

The tests show that the model represents the intensities from the source data within a confidence interval of 95%.

5.7.2 Intensity and variance from the on-ramp flow (q_{on})

The intensities and variances from the highway flow from the model are compared with the observed values. The on-ramp flow in the model is, as explained in Section 5.5.2, controlled by traffic lights. Samples of the flows from the model and observed flows are compared in Figure 5.5.

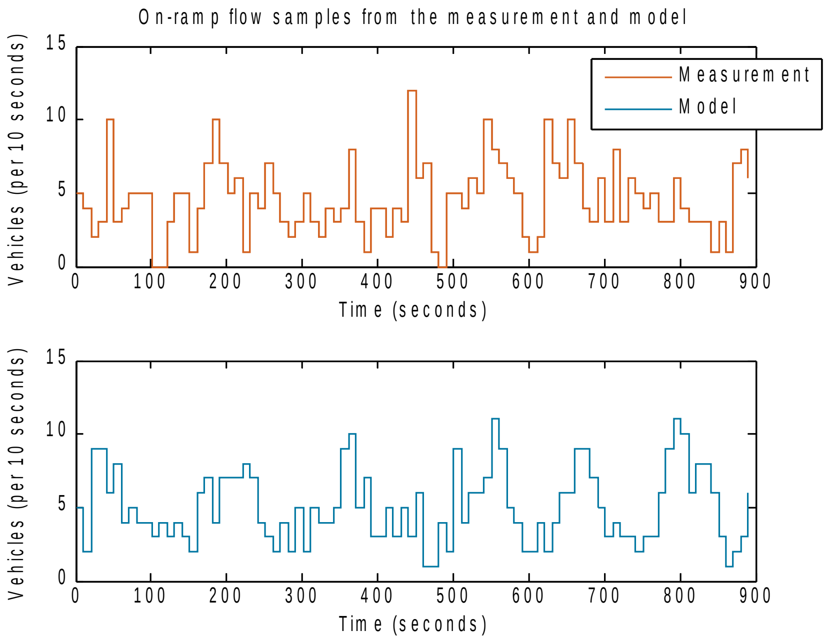


Figure 5.5: On-ramp flow observed from the measurements compared to the on-ramp flow in the model (during free flow conditions)

The peaks in Figure 5.5 have approximately the same height. This indicates that the peak flows have approximately the same density. The width of the peaks is an indicator for the size of the platoons. The flows in the observations and model seem to match.

The on-ramp flows during free flow and congested conditions are tested in Table 5.9, according to hypotheses 27 and 28, and Equations 31 and 32. Here, a main characteristic of the free-flow conditions is that the average speed is approximately 70 km/h. A main characteristic of the congested flow conditions is that the average speed is approximately 40 km/h.

Table 5.9: On-ramp flows during congestion and free flow.

Condition	Test	Reference		Model		Test value	Critical value	Accept H_0?
		Value	n	Value	n			
Free flow	Mean (T)	1664	30	1649	49	$T_{DF}=0.12$	$T_{0.05}=1.64$	Yes
	Variance (F)	515^2	30	524^2	49	$F_{n-1}^{m-1}=1.04$	$F_{0.05}=1.65$	Yes
Congested flow	Mean (T)	1345	12	1533	75	$T_{DF}=1.75$	$T_{0.05}=1.64$	No
	Variance (F)	227^2	12	740^2	75	$F_{n-1}^{m-1}=10.6$	$F_{0.05}=1.75$	No

Table 5.9 shows that the average on-ramp intensity and variance during free flow conditions in the model matches with the observed data. The table also shows that on-ramp flows during congestion do not match. The model is thus not able to represent congested flows at the on-ramp.

5.7.3 Headway distributions

The headway distributions from the model are compared with the observed headway distributions. Here, only traffic situations during free flow conditions are considered (with speeds above 70 km/h).

Figure 5.6 displays the headway distribution. The headway distributions for lane two seem to match well. The headway distributions have a mismatch for short headways on lane one. The figure shows namely that headways under 1 second are under-represented in the model, and that headways between 1 and 2 seconds are over-represented. This is mainly due to the prescribed minimum headways per vehicle type in the model. The model slightly overestimates short headways in lane one, which causes more unstable traffic patterns and thus earlier congestion at lane one.

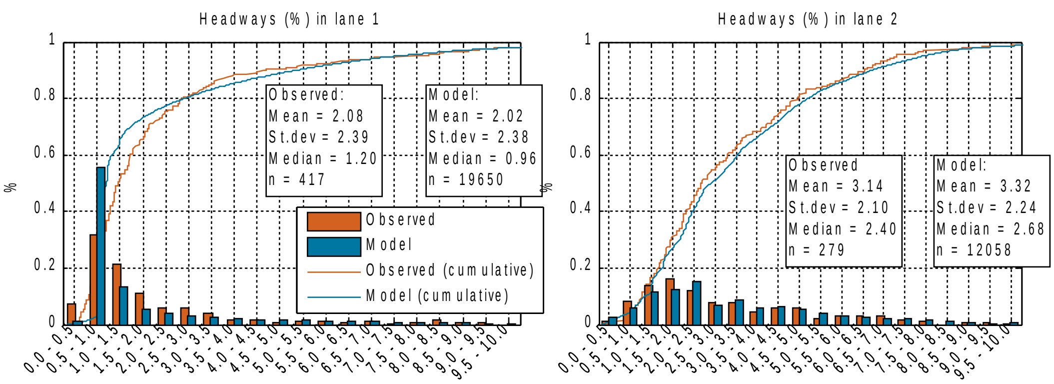


Figure 5.6: Headway distribution and occupancy after calibration

The mean and variances of the headway distributions can be tested with a T-, F- and K-test according to hypotheses 27, 28, 29 and Equations 31, 32 and 33. The results are shown in Table 5.10.

Table 5.10: Statistical tests for the headway analysis

Lane	Test	Reference	Model	Test value	Critical value	Accept H_0?
Lane 1	Mean headway (T)	2.08	2.02	$T_{DF}=0.51$	$T_{0.05}=1.64$	Yes
	Headway variance (F)	2.39^2	2.38^2	$F_{n-1}^{m-1}=1.01$	$F_{0.05}=1.22$	Yes
	Distribution (KS)	-	-	$D_n=0.20$	$D_{0.05}=0.07$	No
Lane 2	Mean headway (T)	3.14	3.32	$T_{DF}=1.41$	$T_{0.05}=1.64$	Yes
	Headway variance (F)	2.10^2	2.24^2	$F_{n-1}^{m-1}=1.14$	$F_{0.05}=1.22$	Yes
	Distribution (KS)	-	-	$D_n=0.07$	$D_{0.05}=0.08$	Yes

The lane distribution ratio in the model is 62% / 38% at the location of the video measurements. During observations, this was 60% / 40%. The mean headways in the model, and thus the lane distribution, are representative for the observed mean headways.

Though, the lane distribution ratio in the model is in general different than observed. Section 4.4.2 described changing lane distribution ratios before the merge. This behaviour is not observed in the model. In the whole pre-merging section (before the start of the continuous line), the lane distribution is equal. The share of vehicles on lane one is in the model higher than observed. On the other hand, cooperative lane changing does not occur in the model. Apparently the model already included the effect of cooperative lane changing.

The headway distributions also seem to be representative, though only the KS-test for the distribution in lane one shows a statistical difference. This is caused by constrains within the model, as explained above. The headway distributions in the model are considered to be representative for the observed headway distributions.

Difference between headways
The distribution of Δh shows to what extent a successive headway deviates from a subject headway (Figure 5.7).

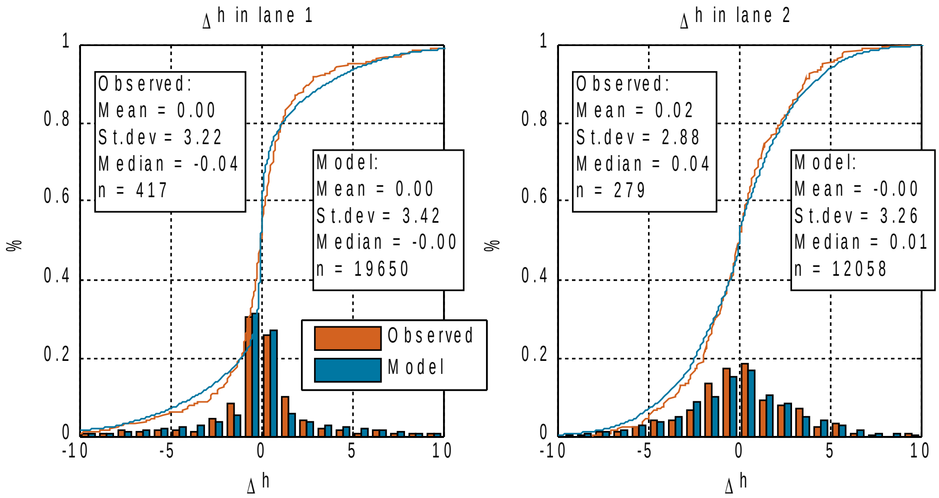


Figure 5.7: Difference between successive headways after calibration

The results from the model seem to match with the observed values. For lane two, the observed differences in the model are slightly smaller than in the model. This is an indication for more platooning in observations than in the model.

The mean, variance and distributions of Δh are tested with T-, F- and KS-tests according to hypotheses 27 - 29, and Equations 31 - 33. The results are shown in Table 5.11.

Table 5.11: Statistical tests for headway difference

Lane	Test	Reference	Model	Test value	Critical value	Accept H_0?
Lane 1	Mean Δh (T)	0.00	0.00	$T_{DF}=0.00$	$T_{0.05}=1.64$	Yes
	Δh variance (F)	3.22^2	3.42^2	$F_{n-1}^{m-1}=1.13$	$F_{0.05}=1.22$	Yes
	Distribution (KS)	-	-	$D_n=0.12$	$D_{0.05}=0.07$	No
Lane 2	Mean Δh (T)	0.02	-0.00	$T_{DF}=0.11$	$T_{0.05}=1.64$	Yes
	Δh variance (F)	2.88^2	3.26^2	$F_{n-1}^{m-1}=1.28$	$F_{0.05}=1.22$	No
	Distribution (KS)	-	-	$D_n=0.05$	$D_{0.05}=0.08$	Yes

The mean headway deviations are in the model within a 95% confidence interval the same as for the observations. For lane one, the distribution is not statistically the same, though the standard deviation is. The difference in the KS-test is mainly caused by a slight difference in the steep part of the probability distribution.

For lane one, the standard deviation is not statistically the same, though the distribution is the same. This difference is caused by a slight over-representation of small headway differences. The probability distribution shows that the effect of this is only small.

The mean in headway differences and distribution in headway differences in the model are therefore considered to be representative for the observations.

5.7.4 Fundamental q/v diagram

The fundamental q/v diagrams from the model are compared with the fundamental q/v diagram from the observed data. The result is shown in Figure 5.8.

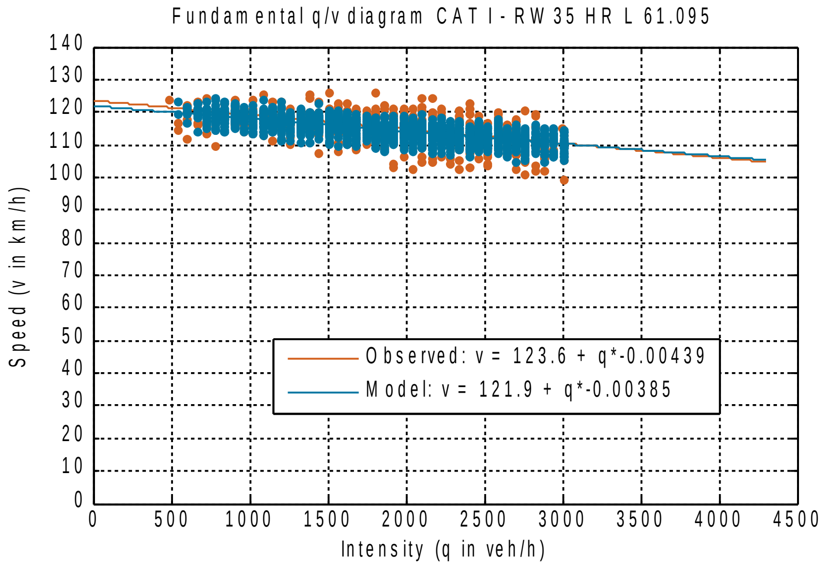


Figure 5.8: Fundamental q/v diagram after calibration

The diagrams seem to match. The slope of the line is mainly caused by the speed differences between the vehicles. A large difference between desired speeds of vehicles leads to a steeper slope.

The source data shows more deviation in the q/v plot: this is partly due to the deviation in de vehicle distributions. The desired speeds for vehicles in the model are fixed. For the source data, this is not the case. For this reason, the model simulates less variance in speeds, while during observations, coincidentally lower and higher speeds can be observed due to a group of slow or fast drivers.

Whether the lines from the model and the source data are the same or not can be tested with hypotheses 25 and 26.

$$H_0: a_1 = a_2 ; H_1: a_1 \neq a_2 \text{ ; Accept } H_0 \text{ if } |a_1 - a_2| < 2 \cdot SE_{a_1 - a_2} \quad (25)$$

$$H_0: b_1 = b_2 ; H_1: b_1 \neq b_2 \text{ ; Accept } H_0 \text{ if } |b_1 - b_2| < 2 \cdot SE_{b_1 - b_2} \quad (26)$$

The hypotheses are tested in Table 5.12.

The continuous line continued...

Table 5.12: Statistical test for the fundamental q/v diagram

Line property	Value	Reference	Model	Difference	Accept H_0?
Slope	a	a_1 = -0.00438	a_2 = -0.00385	$\lvert a_1 - a_2 \rvert$ = 0.00054	Yes
	SE_a	SE_{a1} = 0.00033	SE_{a2} = 0.00008	SE_{a1-a2} = 0.00034	
Constant	b	b_1 = 123.6	b_2 = 121.9	$\lvert b_1 - b_2 \rvert$ = 1.70	No
	SE_b	SE_{b1} = 0.66	SE_{b2} = 0.17	SE_{b1-b2} = 0.68	

The slope of the graph is representative for the observations within a 95% confidence interval (which matches with 2 · SE). The hypothesis for the constant is however not accepted within a 95% confidence interval.

The average speed at low intensities is thus slightly lower in the model. It was however not possible to let the constant values match. Adjusting the calibration parameters such that the constants match resulted in a mismatch in headways and lane distributions. The result such as it is shown is the most optimal.

The slope is also lower, however not significant. This means that the graphs at higher intensities are more similar, which is also visible in the figure. Since only situations with high intensities are relevant in this research, the effect of the mismatch in constants is estimated to be small in this research.

5.8 Validation of the model

The validation tests the reliability for the model, by comparing the indicators with indicators from another dataset than used for calibration. The model is designed for the highway access at Hengelo-Zuid, thus the model is validated with source data from this location. The validation process is done with the calibrated model; traffic conditions and indicators are gathered from detection loop source data from 2013 and the video observations from 28 January 2013.

5.8.1 Intensity and variance on the highway flow

The validation results of the highway flow are shown in Table 5.13.

Table 5.13: Validation of the highway flow

Model	*Test*	*Reference*		*Model*		*Test value*	*Critical value*	*Accept H_0?*
		Value	**n**	**Value**	**m**			
Intensity model	Mean (T)	3201	724	3200	1464	$T_{DF}=0.04$	$T_{0.05}=1.64$	Yes
	Variance (F)	490	463	536	1464	$F_{n-1}^{m-1}=1.20$	$F_{0.05}=1.22$	Yes
Headway model	Mean (T)	2748	15	2779	468	$T_{DF}=0.21$	$T_{0.05}=1.64$	Yes
	Variance (F)	561^2	15	504^2	468	$F_{n-1}^{m-1}=1.23$	$F_{0.05}=1.66$	Yes

The intensities are thus calibrated sufficient. This indicates that the macroscopic flow characteristics in the model are representative. This is important for the recovery of small shock waves or the occurrence of congestion.

5.8.2 Intensity and variance of the on-ramp flow

From the video measurements of 28 January 2013, only free flow conditions could be observed. The number of observations is 15, which is relative short. The validation results of the on-ramp flow are shown in Table 5.14.

Table 5.14: Validation of on-ramp free flow.

Condition	*Test*	*Reference*		*Model*		*Test value*	*Critical value*	*Accept H_0?*
		Value	*n*	*Value*	*n*			
Free flow	Mean (T)	1599	15	1649	49	$T_{DF}=0.42$	$T_{0.05}=1.64$	Yes
	Variance (F)	352^2	15	524^2	49	$F_{n-1}^{m-1}=2.21$	$F_{0.05}=1.84$	No

The average on-ramp flow in the model is representative for the reference dataset, though the variance not. This is mainly due to the low sample size of the reference dataset (the sample size for the calibration was 30). The variance in on-ramp flow may also be overestimated in the model. An overestimation of this variance probably leads to an increased probability for congestion. This effect should thus be taken into account.

5.8.3 Headway distributions

The headways from the video measurements of 28 January 2013 are compared with those from the model. The results are shown in Figure 5.9.

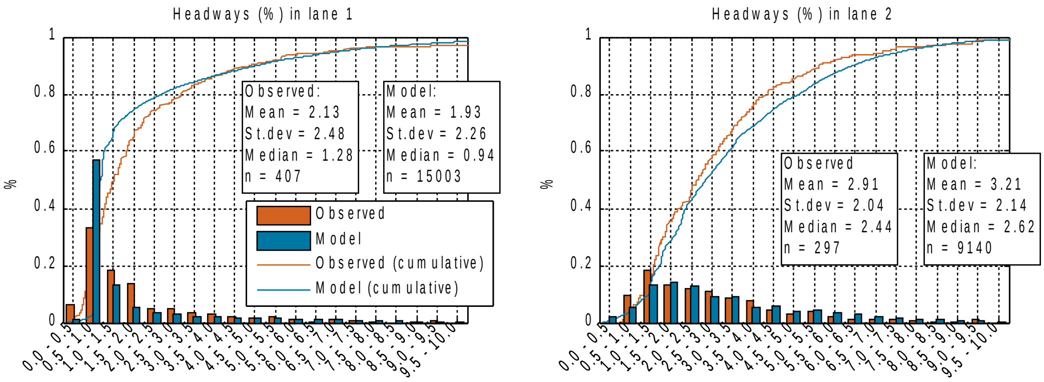


Figure 5.9: Validation of the model with headways

The figure shows that the headways generated in the model are shorter for lane one. This indicates that the lane distribution is different in the model than observed. Relatively more vehicles drive on lane one in the model. For lane two, the whole headway probability distribution from the model is slightly lower than observed. The means, standard deviations and distributions are compared in Table 5.15.

Table 5.15: Mean and standard deviation test for headways

Lane	Test	Observed	Model	Test value	Critical value	Accept H_0?
Lane 1	Mean headway (T)	2.13	1.93	$T_{DF}=1.60$	$T_{0.05}=1.64$	Yes
	Headway variance (F)	2.48^2	2.26^2	$F_{n-1}^{m-1}=1.20$	$F_{0.05}=1.22$	Yes
	Distribution (KS)	-	-	$D_n=0.21$	$D_{0.05}=0.07$	No
Lane 2	Mean headway (T)	2.91	3.21	$T_{DF}=2.45$	$T_{0.05}=1.64$	No
	Headway variance (F)	2.04^2	2.14^2	$F_{n-1}^{m-1}=1.10$	$F_{0.05}=1.22$	Yes
	Distribution (KS)	-	-	$D_n=0.09$	$D_{0.05}=0.08$	No

The lane distribution ratio in the model was 62% / 38% at the location of the video observations. For the video observations this was 58% / 42%.

The tests in Table 5.15 show that there is some mismatch between the observed and modelled headway distributions. At first, the model shows a (statistical relevant) higher mean headway at lane two, and a (statistical not-relevant) lower mean headway at lane one. This means that the average share of traffic at lane one in the model is higher. This matches with the assumption that the model already includes the effect of cooperative lane changing in the lane distributions.

The variance in headways is in both distributions statistically the same, though the KS-tests show that the distributions are different. For lane one, the share of headways between approximately one and three seconds are over-represented in the model, and headways shorter than one second are under-represented. This is partly due to the minimum headway per vehicle type, though the main reason of this difference is the higher share of vehicles on lane one in the model. This causes apparently more platoons (a more constant headway distribution), which results in more short headways. On lane two, the opposite occurs. Less vehicles on lane two cause a more random headway distribution (see the literature in Section 2.2.1), which results in a lower probability function.

The main conclusion after this headway analysis is that the model tends to overestimate the amount of vehicles on lane one. Section 5.1.3 already introduced the effect of cooperative lane changing in the model. The higher share of vehicles on lane one could refer to the effect of cooperative lane changing, which is apparently already included.

Headway deviation
The validation result for the distribution of successive headways is shown in Figure 5.10.

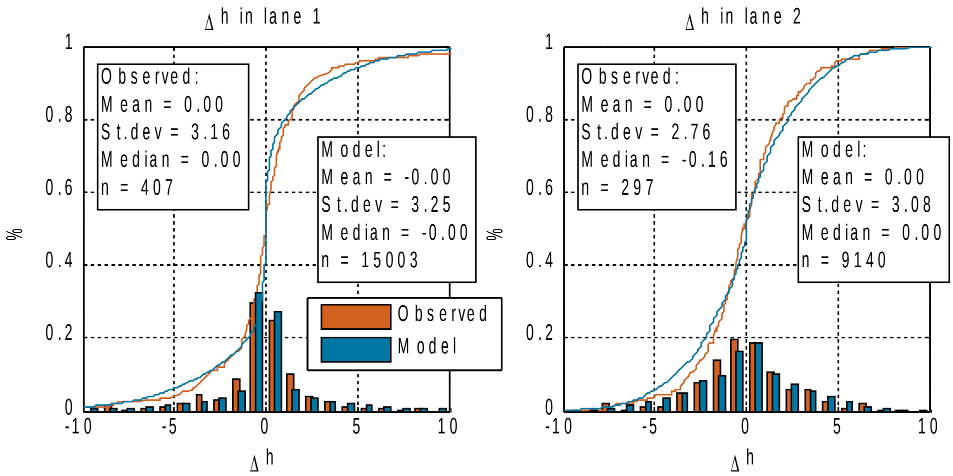


Figure 5.10: Validation results headway difference

The distributions on lane one seem to match in the figure. The distributions on lane two seem to mismatch for headway differences between approximately 2 and 5 seconds, which refers to a more random headway distribution on lane two in the model. The results are tested in Table 5.16.

Table 5.16: Validation – statistical tests for headway difference

Lane	Test	Observed	Model	Test value	Critical value	Accept H_0?
Lane 1	Mean Δh (T)	0.00	0.00	$T_{DF}=0.04$	$T_{0.05}=1.64$	Yes
	Δh variance (F)	3.16^2	3.25^2	$F_{n-1}^{m-1}=1.06$	$F_{0.05}=1.22$	Yes
	Distribution (KS)	-	-	$D_n=0.12$	$D_{0.05}=0.07$	No
Lane 2	Mean Δh (T)	0.00	0.00	$T_{DF}=0.00$	$T_{0.05}=1.64$	Yes
	Δh variance (F)	2.76^2	3.08^2	$F_{n-1}^{m-1}=1.25$	$F_{0.05}=1.22$	No
	Distribution (KS)	-	-	$D_n=0.06$	$D_{0.05}=0.08$	Yes

For lane one, the mean and variance of the headway difference in the model are statistically representative for the observations; the distribution however not. This difference is mainly caused by the steep slope at Δh = 0. A small difference here causes the statistical difference. Though, the distributions for lane 1 are considered to be relevant for the reference frame.

For lane two, the distribution from the model is representative for the observations, though the variance is not. As expected, the headways are more random in the model than observed. Headways here are thus less correlated to each other.

5.8.4 Fundamental q/v diagram

The validation result for the fundamental q/v diagram is shown in Figure 5.11.

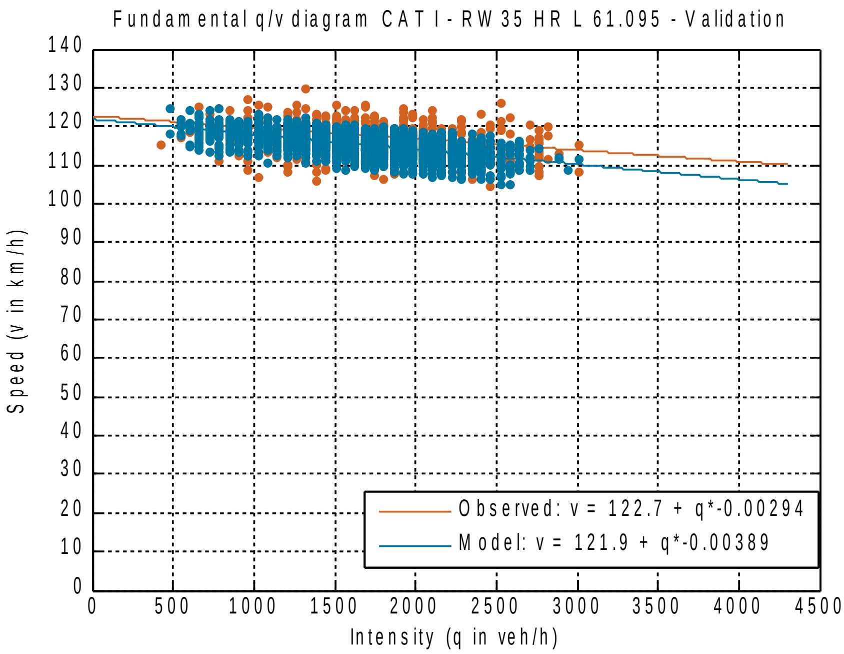


Figure 5.11: Validation q/v diagram

The figure shows that the calculated q/v diagrams mismatch for high intensities. This indicates that the average speed in the model is lower than in reality. Though, it was not possible to increase the slope of the diagram any further without negatively affecting the other calibration indicators. The results are tested in Table 5.17.

Table 5.17: Validation - statistical test for fundamental q/v diagrams

Line property	Value	Reference	Model	Difference	Accept H_0?
Slope	a	a_1 = -0.00294	a_2 = -0.00389	$a_1 - a_2$ = 0.00095	No
	SE_a	SE_{a1} = 0.00020	SE_{a2} = 0.00011	SE_{a1-a2} = 0.00022	
Constant	b	B_1 = 122.7	B_2 = 121.9	$b_1 - b_2$ = 0.84	No
	SE_b	SE_{b1} = 0.34	SE_{b2} = 0.20	SE_{b1-b2} = 0.40	

Both hypotheses are not accepted. Since it was not possible to increase the slope of the line any further, this indicates that the limits of the traffic simulation model are reached. The speeds at high intensities in the model are apparently lower than in reality, which could be clarified by more vehicle interactions in the model. More vehicle interaction refers to a larger congestion probability.

The mismatch in the fundamental q/v diagram probably thus means that the capacity is underestimated.

5.9 Capacity validation

The used traffic simulation software is designed to simulate multiple traffic situations on Dutch highways, though the estimating the capacity is emphasised (Dijker & Knoppers, 2006). In the model, the capacity is based on a probability distribution. A value from a probability distribution is the maximum amount of vehicles that passed a highway section, before congestion is detected in the model. The median of the distribution is the estimated capacity. Though, also probability distributions can be set up for other indicators than the capacity.

In this section, the probability distributions from several indicators from the model are compared with observed distributions, which eventually results in a pronouncement of the suitability of the model. The probability distributions indicated with the model (a.o. capacity distribution) are compared with the probability distributions from the handbook CIA and observed data.

Three indicators are used to validate the probability distributions from the model:

1. the capacity, which is the maximum intensity at the highway downstream (q_{down}) before congestion upstream (q_{up}) occurs (used in the Handbook CIA);
2. the maximum intensity at the highway upstream (q_{up}) before on-ramp congestion (q_{on}) occurs;
3. the maximum intensity at the highway upstream (q_{up}) before congestion at that location (q_{up}) occurs.

The values of indicators 2) and 3) were during observations equal to each other. The measurement locations for the indicators are visualised in Figure 5.12.

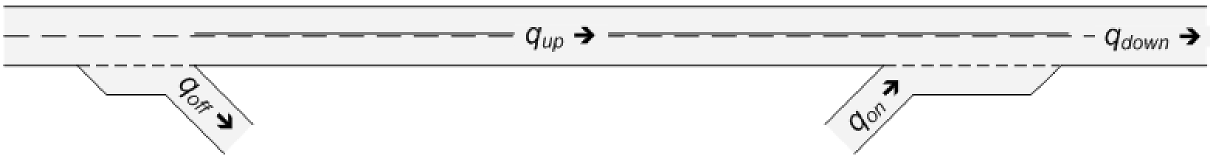


Figure 5.12: Measurement locations for congestion indicators

To estimate the probability distributions in the model, the intensity on the main carriageway slowly increased with 1000 vehicles/hour. The simulation stopped if the speed on the measurement location dropped below a certain threshold: for 1) the measurement location of the detection loops HR 61.095 L, and speed threshold is 40 km/h; equal to empirical capacity analysis. For 2) the measurement location is the on-ramp, and speed threshold is also 40 km/h. For 3) the measurement location is the same as for 1). The measured intensity was the 5-minute intensity.

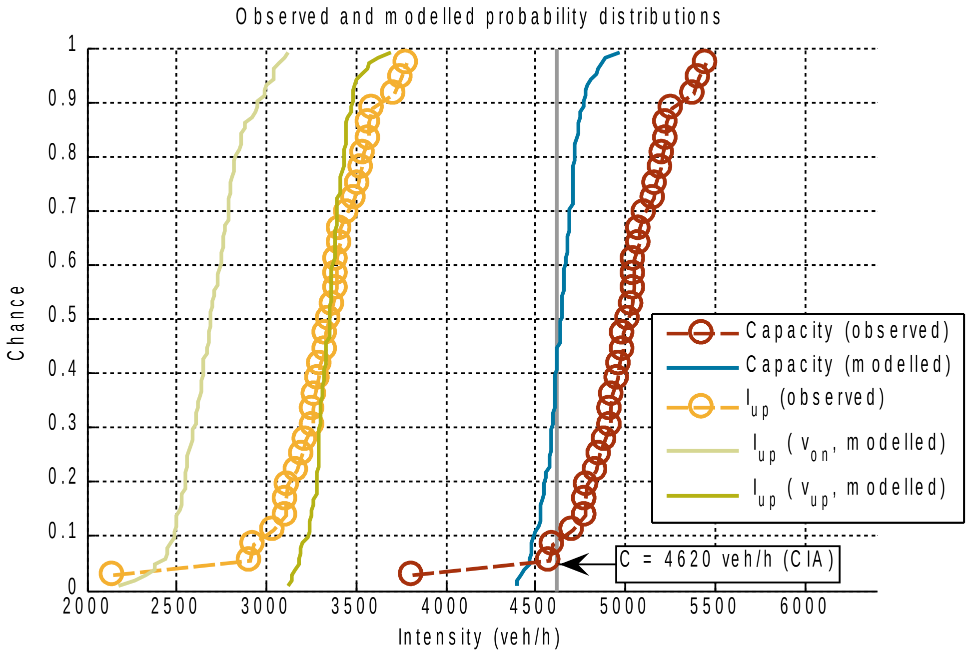


Figure 5.13: Probability distributions from the model compared with empirically measured probability distributions.

In the figure, the empirically measured capacity seems to be much higher than the modelled capacity. This applies also for the highway intensity at which on-ramp congestion occurs. The moment at which highway congestion occurs seems to be correct. The values are tested in Table 5.18 according to Equations 28 and 31.

Table 5.18: Statistical tests for probability distribution values

Indicator	Measurement location	Observed value			Modelled value			Test value	Critical value	Accept H_0?
		Mean	S^2	n	Mean	S^2	n			
Capacity	Highway downstream	5016	523^2	45	4638	107^2	100	3.73	1.64	No
Intensity upstream	On-ramp	3352*	755^{2*}	45	2688	183^2	100	5.82	1.64	No
Intensity upstream	Highway upstream	3352*	755^{2*}	45	3348	102^2	100	0.03	1.64	Yes

* *the intensity upstream at which congestion occurs is calculated as the capacity (5016 ± 523) minus the on-ramp flow (1664 ± 545), which is 3352 ± 755.*

The empirically measured capacity is significant higher than the capacity estimated in the model. The difference is about 8%. Research from Grontmij (2009) has shown that the Fosim method differs in most cases between -10% and 10% from the empirical distribution method. An average is that the capacity values from the Fosim method are 2% lower than the capacity values calculated empirically. Though, the value from the model matches with the capacity in the Handbook CIA.

The difference in capacity could be explained by the explanations mentioned in Section 4.2, which are 1) local conditions; 2) the inaccuracy of capacity calculation with Fosim; 3) the relative low upstream flow; and 4) relaxation. This chapter showed that Fosim overestimates vehicle interactions, as concluded in Sections 5.7 and 5.8. The value from the Handbook CIA is also calculated with Fosim, which means that also here the vehicle interactions are overestimated.

The indicated moment that on-ramp congestion occurs is also significantly lower in the model. This is mainly due to constraints in the model. Fosim uses a critical gap based on a relative speed. If a vehicle is unable to merge in this critical gap, the speeds of the vehicle can drop to zero, which quickly causes congestion at the on-ramp. Research has shown that the merging process in Fosim does not represent observed behaviour (Loot, 2009), which can explain the overestimation of the occurrence of on-ramp congestion.

Highway congestion is predicted more accurate. This indicator seems thus to be reliable. Noticeable is that modelled indicator for highway upstream congestion (3348 veh/h) plus the observed congested on-ramp flow (1345 veh/h) is approximately equal to the modelled capacity. The model apparently underestimates the on-ramp flow in the capacity calculation which is plausible considering the findings of Loot (2009).

Though, since the model is calibrated to estimate the capacity, we assume that a difference in the capacity measured by the model also indicates a difference in the capacity in the studied road section. The indicator for highway congestion could be used for evaluating effects of the continuous line at separate lanes, especially lane one.

5.10 Summary

During the calibration was aimed to let the simulation match with the observations as good as possible, such that correctness of capacity calculations can be improved. The calibration is done by matching indicators from the model and observations. Indicators were intensities and variances of the upstream highway flow and on-ramp flow, headway distributions, and the fundamental q/v diagram. With these indicators, the macroscopic speed and flow characteristics, and microscopic longitudinal and lateral flow characteristic were tried to simulate as good as possible. During the calibration, it was not possible to let all the indicators match. The most optimal result is tried to achieve, which is eventually used for analysis.

Macroscopic flow characteristics

The macroscopic flow characteristics are calibrated sufficient. For free flow conditions, modelled highway intensities and variances represent the observed traffic flows. The variance in on-ramp flow may be slightly overestimated. The model is not able to represent congested traffic flows.

Macroscopic speed characteristics

The macroscopic speed characteristics are calibrated with the fundamental q/v diagram. The most optimal result was tried to achieve, though the fundamental q/v diagrams do not match sufficient. Apparently, vehicle interactions are heavier in the model than observed. The presence of heavy vehicles decreases the average speed in the model. During observations, this happened to a less extent.

Microscopic lateral flow characteristics

Cooperative lane changing behaviour is not implemented in the model. Though, the effect of cooperative lane changing seems to be included in the lane distribution ratio. In the pre-merging section, the share of vehicles on lane one is higher in the model. At merge the distributions may be equal.

As described above, the fundamental q/v diagram obtained from the model does not represent the observations, though the most optimal result was tried to achieve. Lateral flow characteristics are in the model heavier than observed

Microscopic longitudinal flow characteristics

The headway distributions and distributions of difference between headways match sufficient. Restrictions in the model cause slight differences in the distributions, though in general the microscopic longitudinal traffic flow is modelled sufficiently.

Capacity

The capacity is underestimated by the model with approximately 8%. Recent research confirms that observed and modelled capacities can differ this much. The overestimated lateral flow characteristics in the model could also be a reason for underestimating of the capacity.

The moment at which upstream highway congestion occurs can reliably be predicted by the model. This indicator can be used to evaluate effects of the continuous line at separate lanes. On-ramp congestion cannot be modelled precisely due to restrictions in the model.

6. MODELLING

The calibrated traffic simulation model is used to evaluate the effect of the continuous line. At first, the evaluation method and indicators to analyse the effect are elaborated. In the sections thereafter, the effect of the line is evaluated by comparing indicators.

6.1 Evaluation method

The model is calibrated for the traffic situation during evening peak hours at Hengelo-Zuid. Here, the characteristics of the upstream traffic flow match sufficiently with the observations. This is necessary because effects of the continuous line are evaluated at the upstream flow (with a.o. the capacity distribution as indicator, see next section). Concretely, this means that the vehicle distributions and percentage of freight traffic at the upstream flow are fixed within this evaluation framework. The on-ramp flow is used as variable.

Not only the demand at the on-ramp, but also the pattern of the on-ramp flow could influence the capacity distribution. Therefore, multiple on-ramp flows with and without traffic lights are evaluated.

- Traffic lights release a flow with an intermediate headway state (see Section 2.2.1). At high intensities, the traffic lights release large platoons of vehicles, which makes the headway state more constant. After a platoon, the headway state is more random.
- Evaluating without traffic lights means that the model releases vehicles more constantly (DVS, 2011). Though, the long and double on-ramp lane give the headway distributions of the constant vehicle release a very random character (see Section 6.3). This flow pattern occurs for example is traffic comes from a roundabout.
- A ramp meter has a relative constant on-ramp flow. Vehicles are released within a constant time interval. The random on-ramp flow is also released more or less constantly, though for a ramp meter vehicles are released per lane at the same time, and vehicles have less space to accelerate. The conditions for a ramp meter make that other effects of the continuous line can occur.

These three flow patterns are theoretically and practically relevant and form therefore a good framework for evaluation.

Traffic lights are often installed if the on-ramp demand during peak hours is higher than approximately 1000 veh/h. Random on-ramp flows normally occur at low intensities (see Section 2.2.1). Though, on-ramp flows are hardly completely random due to grouping of vehicles (platooning). Both random and signalised flows are evaluated within a range between 500 and 1664 vehicles per hour, such that a comparison between both flows can be made. Practice has shown that the minimum and maximum ramp meter release rates are 500 and 1450 veh/h (Ministerie van Verkeer en Waterstaat, 2007). Table 6.1 shows the evaluated on-ramp flows.

Table 6.1: Evaluated on-ramp flows

On-ramp demand (veh/h)	Signalised flow (Intermediate)	Random flow	Metered flow (Constant)
1664	x	x	-
1500	x	x	x*
1250	x	x	x*
1000	x	x	x*
750	x	x	x*
500	x	x	x*

* the metered on-ramp flow could only be approached, see Table 6.3.

6.2 Simulation settings

6.2.1 Simulate continuous line

The length of the continuous line is a variable in this study. Therefore the effect of the line is tested for different lengths (see Table 6.2). The continuous line prohibits that vehicles change lane from lane one to lane two. This means that none of the vehicles negotiates the line, and eventually leads to an overestimation of the effect of the line.

Table 6.2: Evaluated line lengths

Name	Length	Extra length	Description
Reference	498 m		Current line length
Line 1	793 m	+295 m	Detection loops HR L 61.095
Line 2	1018 m	+225 m	
Line 3	1243 m	+225 m	
Line 4	1468 m	+225 m	End deceleration lane off-ramp

The line lengths are visualised in Figure 6.1.

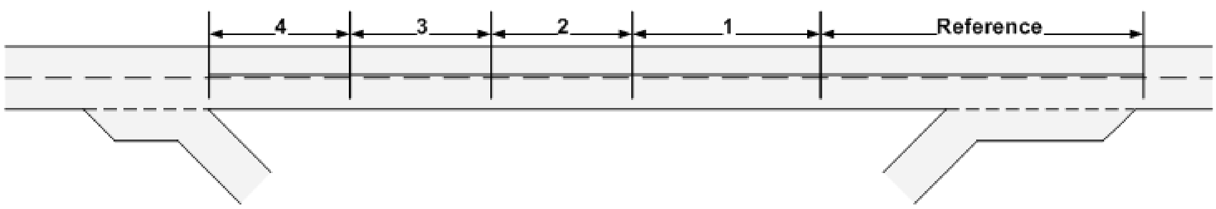


Figure 6.1: Line types

The simulation settings from the evaluation are shown in Appendix IV.

6.2.2 Traffic light settings

The used traffic light settings are as described in Section 5.6.4.

6.2.3 Ramp meter settings

The used traffic simulation software does not simulate ramp meters. Traffic lights can be added though. If the 5-minute intensity can be kept constant, the traffic lights can function as a ramp meter. In the model, the intensities are entered such that the 5-minute-intensities are equal. An inaccuracy here is that the model releases the vehicles with a certain random distribution, which means that the output intensities are not exactly the same each 5 minutes. The simulated ramp meter is not able to react on these deviations in the traffic demand. Though, a ramp meter smoothens the intensities which reduces this inaccuracy.

The release rate from the ramp meter (on-ramp flow) is simulated with traffic lights. The cycle time calculated according to Equation 11. The following settings are therefore used in the model (see Table 6.3).

Table 6.3: Simulated ramp meter settings and flows in Fosim

Release rate (veh/h)	***Vehicles per green time (vehicles · lanes)***	***Cycle time (seconds)***
480	1 · 2	15
720	1 · 2	10
960	2 · 2	15
1200	2 · 2	12
1440	2 · 2	10

The simulation layout matches with the situation at Hengelo-Zuid. Here, the ramp meter is located 242 m before the merge, which is done according to the manual ramp metering (Ministerie van Verkeer en Waterstaat, 2007). For comparison: the traffic lights at the adjacent road network are located 542 m before the merge. The distance between the ramp meter limit line and merge is thus relatively short. Figure 6.2 shows the layout of the ramp meter in the simulation.

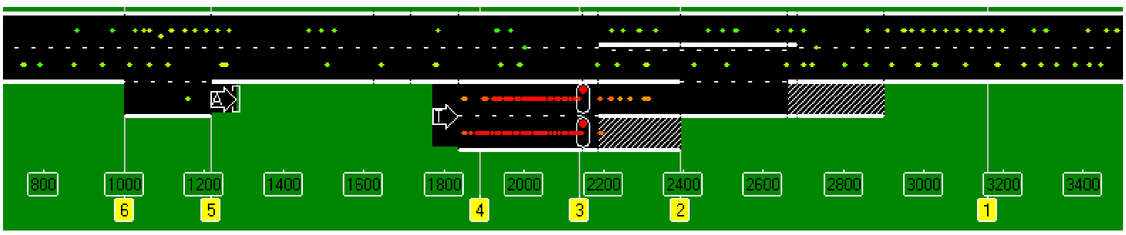


Figure 6.2: The simulation layout for a ramp meter.

6.2.4 Indicators

The following indicators evaluate the effect of the continuous line:

1. the capacity of the highway (measured at q_{down}, congestion detected at q_{up});
2. the average upstream highway intensity at which congestion occurs at lane one (measured at q_{up}, congestion detected at lane one at q_{up});
3. the lane distributions at merge;
4. the lane changes before, during and after the merge; and
5. the estimated effect on travel time delay at the highway and on-ramp.

The locations of the indicators (1) and (2) are shown in Figure 6.3.

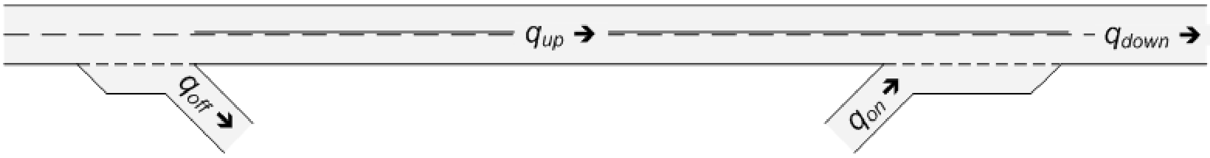


Figure 6.3: Measurement locations for congestion indicators q_{up} and q_{down}.

The capacity value is calculated at the same way as described in Section 5.9.

Congestion at lane one or lane two upstream is defined as the 5-minute average speed lower than 70 km/h. The threshold of 70 km/h is chosen because a lot of small backward shock waves at lane one occur, which are not desirable, and are not detected with a threshold of 40 km/h. A threshold of 40 km/h only detects complete congestion. The difference between effects on both lanes are better visible with a threshold of 70 km/h, which is shown in Figure 6.4 a and b.

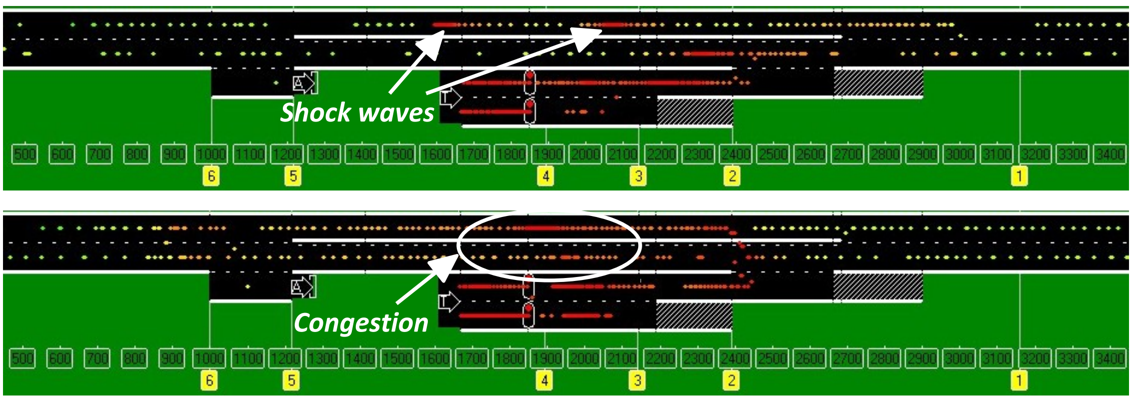


Figure 6.4 a) backward shock waves at lane one are detected with a speed threshold of 70 km/h; and b) congestion is detected with a speed threshold of 40 km/h.

6.3 Headway distributions of on-ramp patterns

Figure 6.5 shows the (cumulative) headway distributions for the different on-ramp flow patterns. In the figure, the curve of the signalised flow crosses the curve of the random flow. This means that the signalised flow clearly has a mix between the constant and random headway state. Short headways are combined with long headways. Signalised on-ramp flows with a lower demand clearly have less platoons.

The distributions from the constant on-ramp flow are usually normal distributed, recognised by a steep slope that does not directly start at a short time headway. This is good recognisable for low on-ramp flows. The random headway distributions are in the figure more equally divided, which is also good recognisable for low on-ramp flows.

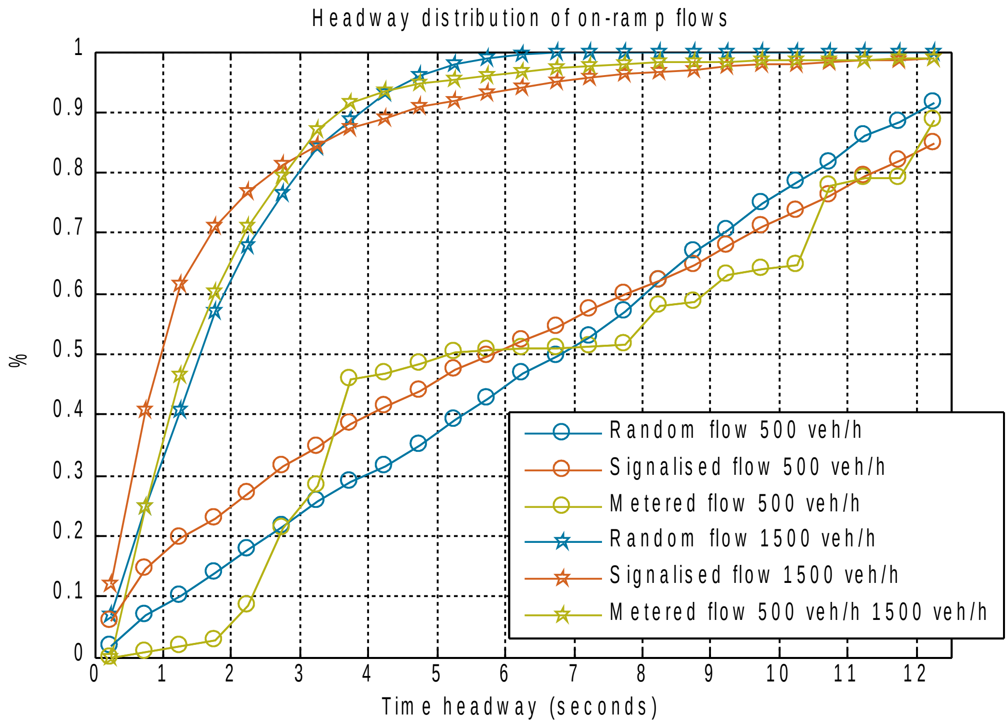


Figure 6.5: Headway distributions of on-ramp flows (cumulative), measured at the merge

Appendix V. shows the headway distributions of the other on-ramp demands.

6.4 Simulation results

This section shows the simulation results for signalised, random and metered on-ramp flows.

6.4.1 Effects on capacity with signalised on-ramp flow

Table 6.4 shows the simulation results for effects of the continuous line on the capacity distributions for a signalised on-ramp flow.

Table 6.4: Effects continuous line with signalised on-ramp flow on capacity

On-ramp demand (veh/h) (signalised)		***Capacity value per line length (veh/h)***				
		Reference (498 m)	**Line 1 (793 m)**	**Line 2 (1018 m)**	**Line 3 (1243 m)**	**Line 4 (1468 m)**
1664	**$P_{50\%}$**	4638	4686	4704	4758	4764
	%	*100%*	*+1%*	*+1%*	*+3%*	*+3%*
1500	**$P_{50\%}$**	4608	4668	4716	4746	4740
	%	*100%*	*+1%*	*+2%*	*+3%*	*+3%*
1250	**$P_{50\%}$**	4524	4632	4656	4674	4692
	%	*100%*	*+2%*	*+3%*	*+3%*	*+4%*
1000	**$P_{50\%}$**	4452	4530	4542	4596	4632
	%	*100%*	*+2%*	*+2%*	*+3%*	*+4%*
750	**$P_{50\%}$**	4524	4614	4620	4692	4722
	%	*100%*	*+2%*	*+2%*	*+4%*	*+4%*
500	**$P_{50\%}$**	4992	4998	4998	5100	5100
	%	*100%*	*n.s.*	*n.s.*	*+2%*	*+2%*

n.s. = no significant difference within a 95% confidence interval.

The table shows that the capacity increases slightly up to 4% for all different on-ramp demands. Another observation is that the capacity depends on the on-ramp flow. According to the Newell-Daganzo model (Section 2.1.2), low on-ramp flows are coupled with a higher capacity. The simulation results shows the same, though they also show a higher capacity for high on-ramp flows. Latter is mainly due to the fact that congestion at lane one already occurs, though the capacity still increases. The effect of the line at lane one is evaluated in the section below.

6.4.2 Effects on the upstream flow with signalised on-ramp flow

Table 6.5 shows the simulation results for effects of the continuous line on lane one for a signalised on-ramp flow. The indicator used is the maximum observed upstream intensity until the moment that the 5-minute average intensity at lane one dropped below 70 km/h.

Table 6.5: Maximum observed intensities before shock waves at lane one occur

On-ramp demand (veh/h) (signalised)		***Maximum observed upstream flow per line length***				
		Reference (498 m)	**Line 1 (793 m)**	**Line 2 (1018 m)**	**Line 3 (1243 m)**	**Line 4 (1468 m)**
1664	**$P_{50\%}$**	3288	3372	3192	3120	3072
	%	*100%*	*+3%*	*−3%*	*−5%*	*−7%*
1500	**$P_{50\%}$**	3288	3360	3192	3162	3210
	%	*100%*	*+2%*	*−3%*	*−4%*	*−2%*
1250	**$P_{50\%}$**	3258	3354	3300	3312	3336
	%	*100%*	*+3%*	*+1%*	*+2%*	*+2%*
1000	**$P_{50\%}$**	3444	3522	3498	3540	3594
	%	*100%*	*+2%*	*+2%*	*+3%*	*+4%*
750	**$P_{50\%}$**	3840	3924	3960	3978	4002
	%	*100%*	*+2%*	*+3%*	*+4%*	*+4%*
500	**$P_{50\%}$**	4428	4446	4566	4656	4632
	%	*100%*	*n.s.*	*+3%*	*+5%*	*+5%*

n.s. = no significant difference within a 95% confidence interval.

The table shows that negative effects of the continuous line can occur at lane one. For high on-ramp flows, there seems to be an optimal line length. For lower on-ramp flows, there seem to be no negative effects at lane one. Then, a long line seems to be optimal.

It seems strange that the capacity increases while negative effects occur at lane one. Though, this difference is due to the different speed thresholds for the indicators: a speed threshold of 40 km/h does not show any negative effects at lane one.

6.4.3 Effects on capacity with random on-ramp flow

Table 6.6 shows the simulation results for effects of the continuous line on the capacity distributions for a random on-ramp flow.

Table 6.6: Effects continuous line with constant on-ramp flow on capacity

On-ramp demand (veh/h) (random)		*Capacity value per line length*				
		Reference (498 m)	**Line 1 (793 m)**	**Line 2 (1018 m)**	**Line 3 (1243 m)**	**Line 4 (1468 m)**
1664	**$P_{50\%}$**	4686	4758	4764	4788	4824
	%	*100%*	*+2%*	*+2%*	*+2%*	*+3%*
1500	**$P_{50\%}$**	4692	4740	4728	4782	4776
	%	*100%*	*+1%*	*n.s.*	*+2%*	*+2%*
1250	**$P_{50\%}$**	4746	4770	4794	4812	4818
	%	*100%*	*n.s.*	*+1%*	*+1%*	*+2%*
1000	**$P_{50\%}$**	4788	4872	4872	4830	4902
	%	*100%*	*+2%*	*+2%*	*n.s.*	*+2%*
750	**$P_{50\%}$**	4986	4974	4992	4974	5022
	%	*100%*	*n.s.*	*n.s.*	*n.s.*	*n.s.*
500	**$P_{50\%}$**	5160	5148	5148	5178	5178
	%	*100%*	*n.s.*	*n.s.*	*n.s.*	*n.s.*

n.s. = no significant difference within a 95% confidence interval.

Here, the table shows different results than for a signalised on-ramp flow. The capacity increases for high on-ramp flows. For low on-ramp flows, the capacity does not increase significantly. The capacity here does increase according to the Newell-Daganzo model. The modelled capacity is higher for low on-ramp flows.

6.4.4 Effects on the upstream flow with random on-ramp flow

Table 6.7 shows the simulation results for effects of the continuous line on lane one for a random on-ramp flow. The indicator used is the maximum observed upstream intensity until the moment that the 5-minute average intensity at lane one dropped below 70 km/h.

Table 6.7: Maximum observed intensities before shock waves at lane one occur

On-ramp demand (veh/h) (random)		Maximum observed upstream flow per line length				
		Reference (498 m)	Line 1 (793 m)	Line 2 (1018 m)	Line 3 (1243 m)	Line 4 (1468 m)
1664	$P_{50\%}$	3312	3384	3336	3288	3282
	%	*100%*	*+2%*	*n.s.*	*n.s.*	*n.s.*
1500	$P_{50\%}$	3348	3438	3372	3402	3396
	%	*100%*	*+3%*	*n.s.*	*+2%*	*+1%*
1250	$P_{50\%}$	3540	3588	3558	3612	3612
	%	*100%*	*n.s.*	*n.s.*	*+2%*	*+2%*
1000	$P_{50\%}$	3876	3930	3954	3906	3984
	%	*100%*	*n.s.*	*+2%*	*n.s.*	*+3%*
750	$P_{50\%}$	4320	4320	4320	4302	4362
	%	*100%*	*n.s.*	*n.s.*	*n.s.*	*n.s.*
500	$P_{50\%}$	4758	4746	4764	4788	4752
	%	*100%*	*n.s.*	*n.s.*	*n.s.*	*n.s.*

n.s. = no significant difference within a 95% confidence interval.

The table shows that the effects of the continuous line on lane one are relatively small, or not significant. For low on-ramp flows, the effects are not significant. This is obvious, since the capacity did also not increase. For the higher on-ramp flows, also no negative effects can be observed.

6.4.5 Effects on capacity with metered on-ramp flow

Table 6.8 shows the effect of the continuous line in combination with a ramp meter on the capacity.

Table 6.8: Effect continuous line with ramp meter on capacity

Release rate (veh/h)		***Capacity per line length (veh/h)***				
		Reference	**Line 1**	**Line 2**	**Line 3**	**Line 4**
480	$P_{50\%}$	5202	5220	5202	5256	5250
	%	*100%*	*n.s.*	*n.s.*	*+1%*	*+1%*
720	$P_{50\%}$	4656	4740	4800	4836	4902
	%	*100%*	*+2%*	*+3%*	*+4%*	*+5%*
960	$P_{50\%}$	4488	4560	4602	4650	4644
	%	*100%*	*+2%*	*+3%*	*+4%*	*+3%*
1200	$P_{50\%}$	4560	4632	4644	4692	4752
	%	*100%*	*+2%*	*+2%*	*+3%*	*+4%*
1440	$P_{50\%}$	4596	4668	4710	4710	4740
	%	*100%*	*+2%*	*+2%*	*+2%*	*+3%*

n.s. = no significant difference within a 95% confidence interval.

The table shows that the capacity for an on-ramp equipped with a ramp meter depends on the release rate of the ramp meter. If the release rate is low (480 vehicles per hour), the estimated capacity is relatively high. If the release rate increases (up to 960 vehicles per hour), the capacity decreases. For higher on-ramp flows (1200 and 1440 vehicles per hour), the capacity increases again.

This effect also occurred for the signalised on-ramp flow. The increase of capacity for higher on-ramp flows does not match with the Newell-Daganzo model. Latter is mainly due to the fact that congestion already occurs at lane one, though capacity still increases. A lower on-ramp flow causes less disruptions at the merge, simply because there are less merging manoeuvres.

6.4.6 Effects on the upstream flow with metered on-ramp flow

Table 6.9 shows the results for the effects of the continuous line with a ramp meter on lane one, for different release rates. The indicator used is the maximum observed upstream intensity until the moment that the 5-minute average intensity at lane one dropped below 70 km/h.

Table 6.9: Maximum observed intensities before shock waves at lane one occur

Release rate (veh/h)		***Maximum observed upstream flow per line length***				
		Reference	**Line 1**	**Line 2**	**Line 3**	**Line 4**
480	**$P_{50\%}$**	4668	4740	4764	4824	4800
	%	*100%*	*+2%*	*+2%*	*+3%*	*+3%*
720	**$P_{50\%}$**	3960	4050	4050	4116	4200
	%	*100%*	*+2%*	*+2%*	*+4%*	*+6%*
960	**$P_{50\%}$**	3468	3504	3528	3552	3564
	%	*100%*	*n.s.*	*+2%*	*+2%*	*+3%*
1200	**$P_{50\%}$**	3288	3360	3330	3372	3384
	%	*100%*	*+2%*	*n.s.*	*+3%*	*+3%*
1440	**$P_{50\%}$**	3276	3384	3228	3192	3144
	%	*100%*	*+3%*	*−1%*	*−3%*	*−4%*

n.s. = no significant difference within a 95% confidence interval.

For high on-ramp flows, there seems to be an optimal length of the line. For lower on-ramp flows, a long line seems to be the most effective. This effect is mainly due to the fact that low on-ramp flows correspond with less merging manoeuvres, and thus a lower probability for disruptions at the merge. For higher on-ramp flows, there seem to occur negative effects at lane one. This is probably due to the fact that the merging speed for a metered on-ramp flow is lower. The speed differences at merge are this higher, which results into a larger probability for disruptions, and congestion.

6.5 Summary

In this chapter, the modelling of the continuous line is performed for three different on-ramp flows: a signalised on-ramp flow (representing an intermediate headway state), a random on-ramp flow (representing a more random headway state) and a metered on-ramp flow (with RWS-algorithm, representing a more constant on-ramp flow). The result is that a continuous line can increase the capacity.

- For high signalised on-ramp flows, there seems to be an optimum in line length regarding negative effects (backward shock waves) at lane one. For lower signalised on-ramp flows, a long line seems to be the most effective.
- For high random on-ramp flows, a long continuous line also seems to have an effect on the capacity, though no negative effects are observed.
- For high metered on-ramp flows, there seems also to be an optimum in line length regarding negative effects at lane one. For lower metered on-ramp flows, a long line seems to be the most effective.

The pattern of the on-ramp flow apparently influences the effect of the continuous line. The effect of the line is for random on-ramp flows less large than for signalised and metered on-ramp flows. This is mainly due to the fact that capacities for random on-ramp flows are beforehand higher. This is obvious since a merging vehicle has both time and space for finding a gap. For platoons of vehicles this is much harder and disruptions are more likely to occur. For a metered on-ramp flow, the difference is mainly due to the slower speed at merge (Wu et all, 2007; see Section 2.5). Especially freight vehicles arrive with a relative low speed at the merge and earlier cause disruptions. Literature showed however that in practice more cooperative lane changing occurs with a ramp meter, and therefore the modelled capacities are in reality higher than for a random on-ramp flow (Middelham & Taale, 2006; Wu et al., 2007; see also Section 2.5).

Note is that a ramp meter aims for a downstream flow equal to the discharge flow, which makes increasing the capacity not directly useful. Though, the aimed downstream flow could with a line be increased without influencing the congestion probability. This is described in the next chapter.

The effect of the continuous line for a signalised and random on-ramp flow is sketched in Figure 6.6. The figure is based on the Newell-Daganzo-model from Section 2.1.2.

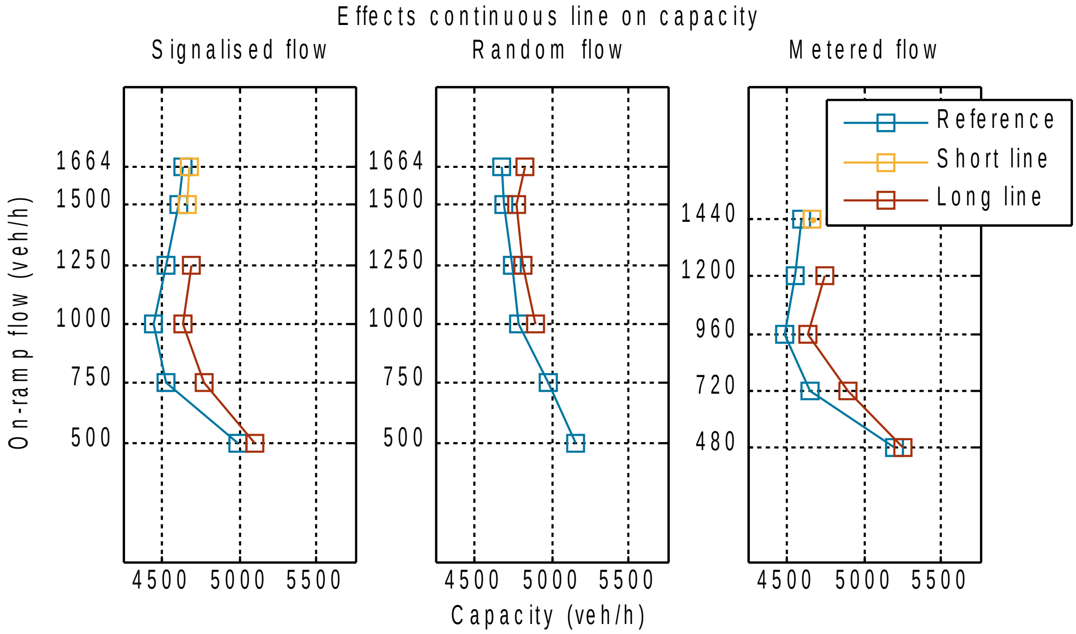


Figure 6.6: Visualisation of effects continuous line on capacity

The figure does not completely match with the Newell-Daganzo model from Section Figure 2.2. According to the Newell-Daganzo model, the capacity decreases if the on-ramp flow increases. Though, this is not the case for the modelled signalised on-ramp flows. This is mainly due to the fact that for high signalised and metered on-ramp flows congestion at one lane already occurred, though the capacity was still able to increase. The main message in the figure is however to visualise the increase in capacity by using the continuous line. The capacity increase for low random on-ramp flows are not significant and therefore not shown in the figure.

7. Evaluation

This chapter describes the evaluation of the modelled results. The first section describes the lane distributions where should be aimed for. The second section describes the influence on lane change manoeuvres. Also the influence on travel time delay and external validity of the results are evaluated. The chapter ends with a summary.

7.1 Lane distributions

In the model, the length of the line does not determine the effect of the line. It causes a certain lane distribution, where should be aimed for. Figure 7.1 shows the lane distributions at different highway intensities just before the merge (q_{up}), for different line lengths.

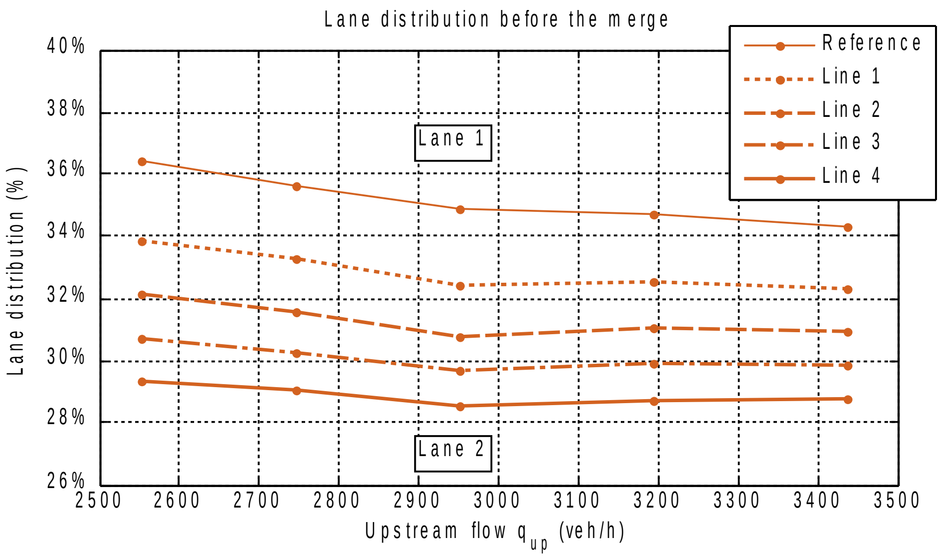


Figure 7.1: Lane distributions

The figure shows that the line length influences the lane distribution to a large extent. The upstream flow q_{up} hardly influences the lane distribution. Since the model does not include cooperative lane change behaviour (but the effect of cooperative lane changing at merge is included), the length of the line in the model is not representative for the effect on lane distribution. However, the lane distributions determine the highway performance. In other words, the real relation between lane distribution and length of the line could in reality be different than represented in the model. Test pilots can show this.

The optimal lane distributions can be derived from the figure. For example: in situations where a long line is considered to be optimal, there should be aimed for a lane distribution of 71% at lane one, and 29% at lane two (expressed in % of the flow in vehicles per hour).

7.2 Lane change analysis

In the simulation, traffic at lane one is forced to stay at that lane. Merging traffic has less space to manoeuvre to lane one. After the merging section and end of the continuous line, vehicles from both lanes are free to change lane, and traffic can weave. This section evaluates the effect of this weaving behaviour.

The lane changing behaviour is in reality mainly due to aggressive drivers (rabbits) and less aggressive drivers (slugs) (Daganzo, 2002). In Fosim, the behaviour of aggressive drivers is not represented sufficiently, since only available gaps are taken into account for a lane change.

Figure 7.2 shows the lane change behaviour in the model for a highway demand of 2500 vehicles per hour, which represent conditions without congestion.

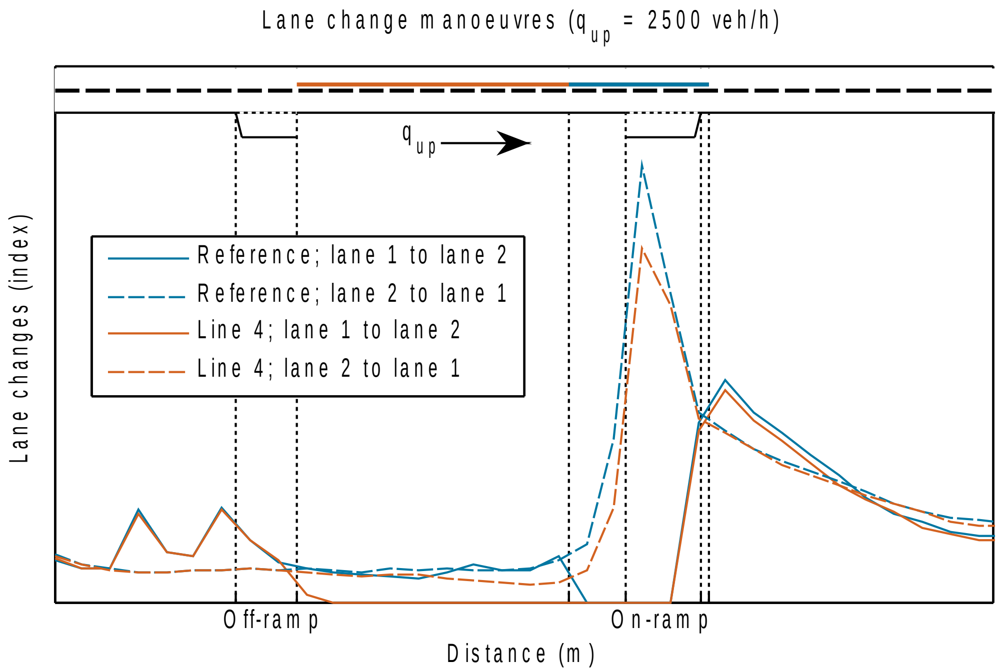


Figure 7.2: Lane change behaviour

The figure shows that there are less lane changes over the whole section if a long continuous line is implemented. This behaviour seems logical, because vehicles on lane one are not able to change lane. The available gaps at lane one for vehicles on lane two are thus reduced.

The amount of lane changes at the merge is also lower, as well due to less available gaps on lane one. The amount of weaving traffic after the merge seems to be less with a line than without a line. The behaviour can be explained by the fact that aggressive drivers do not necessarily have the aim to drive right; less aggressive drivers, or slower drivers do not necessarily have the aim to change lane to lane one. Otherwise, they would have changed lane before the merge already.

Next to that, without continuous line vehicles are able to change lane multiple times. With the continuous line, vehicles are able to change lane only once.

7.3 Effect on travel time delay

Effects on travel time delay are evaluated in general, for the case Hengelo-Zuid, and for the ramp meter.

7.3.1 Qualitative evaluation on travel time delay

The continuous line could increase the capacity. This means that congestion duration could be reduced: the probability for congestion is namely reduced since the probability distribution has shifted to the right.

The modelled capacity increase is 1-4% for signalised on-ramp flows, 0-2% for random on-ramp flows and 2-5% for metered on-ramp flows. The extent of this increase is questionable, though the assumption can be made that there is a significant increase, and that the increases are for signalised and metered on-ramp flows are higher than for random on-ramp flows.

7.3.2 Case Hengelo-Zuid

Returning to the case at Hengelo-Zuid, the average maximum upstream intensity (q_{up}) per day is approximately 3500 veh/h (see Figure 4.3). With an on-ramp flow of 1664 veh/h, the maximum demand per day at the merge is approximately 5164 veh/h. The current capacity is 5016 veh/h, which means 50% probability that congestion occurs at this demand. A slight increase of the capacity refers thus to a reduced probability for congestion. Looking at the probability distribution in Figure 5.13, an intensity of 5164 veh/h refers to a congestion probability of approximately 70%. A capacity increase of 1-4% could reduce the probability to approximately 50-60%. The probability that congestion does not occur increases with 10-20 percent points. Latter effect is shown in Figure 7.3.

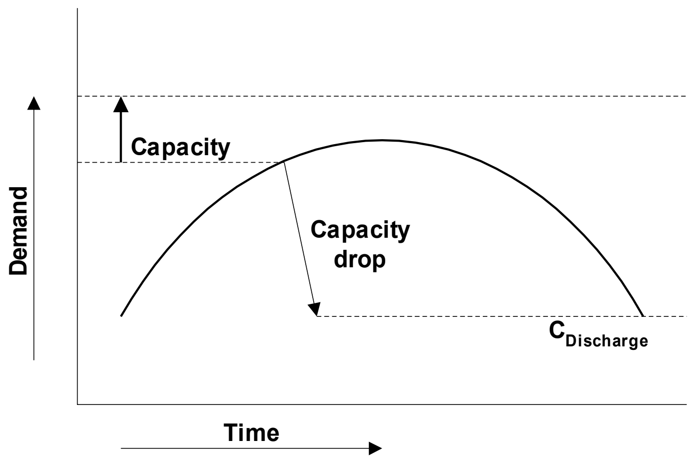


Figure 7.3: Possible effect of a capacity increase (outline).

If congestion can be prevented, the effect of the line is relatively large. The daily extra on-ramp delay of 160 vehicles could be prevented. If congestion only can be postponed, the estimated gain in time is approximately 0 – 5 minutes (derived from Figure 4.3). If congestion has approximately 5 minutes less to grow, congestion requires also less time to solve. A rough estimation is a reduction in on-ramp delay of 1.5 minutes (45 vehicles).

For 2020, the highway flow is expected to increase. Then, congestion can probably not be prevented. Though congestion can be postponed which results in a reduction of on-ramp delay.

7.3.3 Travel time delays for a ramp meter

As shown in Chapter 2, a ramp meter (with an RWS algorithm) reduces the risk for congestion at the highway. The maximum allowed downstream flow is equal to the discharge capacity. Then, congestion at the highway cannot occur due to a too high demand, because no backward shock waves will occur. This means that an active ramp meter also (de facto) involves a capacity drop. To increase traffic throughput, there should be aimed for a higher throughput than the discharge capacity.

The continuous line reduces the probability for congestion with a metered on-ramp flow. The capacity is thus higher. This implies that the release rate of the ramp meter could be increased. The increase is equal to the absolute increase of the capacity, and varies between approximately 50 and 250 vehicles per hour (which can be derived from Table 6.8 and Table 6.9). This is however an estimation, and a better indication can be done after field capacity studies with continuous line. And if congestion already occurs, increasing the release rate has no use.

The reduce in travel time delay is explained with the following example.

> ***Example:*** if an average metered on-ramp flow of 750 veh/h could be increased with 100 veh/h up to 850 veh/h over a 30 minute period of on-ramp congestion, the delay could be decreased with 50 vehicles. The travel time saving at the on-ramp would be 4 minutes.

7.4 External validity of the results

Section 3.4 described the conditions which must apply to provide the external validity of the results. These were:

- the merge is an active bottleneck;
- the highway has the layout of a two-lane highway with a speed limit of 130 km/h;
- the vehicle distributions and driver behaviour are comparable.

The results are applicable for locations which meet these conditions. Though, the results may also apply if some of these conditions differ.

If a merge is no active bottleneck, increasing the capacity at the merge has negative effects for the bottlenecks downstream which makes a continuous line superfluous.

This is for example the case of situations where the speed limit is lower than 130 km/h. Knoop et al. (2010) showed that for slower speeds (60 km/h) the share of vehicles at lane two is higher. This means that the continuous line should be longer to reach the optimal lane distribution.

Effects for highways with more lanes are not evaluated, since a reference frame is missing. Effects of other vehicle distributions and different amounts of freight traffic are not evaluated.

Furthermore, only a RWS algorithm is evaluated. Though, the other common used ramp meter strategy (ALINEA) uses the same release rate formula and effects are assumed to be the same.

7.5 The continuous line under different circumstances

The evaluation of the continuous line showed that the optimal length of the line depends on the on-ramp flow. In practice, multiple on-ramp demands and patterns can occur at an on-ramp. Demands can vary in case of road works, accidents, events, or just coincidentally. A ramp meter can be broken down (which is the case in Hengelo-zuid), which also refers to a different on-ramp flow.

Next to that, the effect of the continuous line is evaluated for non-congested situations. If congestion occurs, the capacity will drop, and the line has no effect on the capacity anymore.

A short version of the continuous line is therefore the most ideal for all situations. Then, the line causes no negative effects at lane one, and increases the capacity for all situations from 0% up to 2%.

7.6 Summary

In this chapter, the effects of the continuous line are evaluated into more detail.

As shown in the previous chapter, the continuous line can increase the capacity of the highway slightly but significantly. Aim is however to create an optimal lane distribution at the merge. The relation between the optimal lane distribution and the length of the line could not be determined by the model. Test pilots could evaluate this. The amount of lane changes does not increase with a line: neither before the merge, nor after the merge.

The modelled capacity increase is 1-4% for signalised on-ramp flows, 0-2% for random on-ramp flows and 2-5% for metered on-ramp flows. The effect for metered on-ramp flows is however overestimated, as described in Chapter 6. The continuous line has thus the largest effect on the capacity for signalised on-ramp flows. For a ramp meter, a continuous line enables a higher release rate without increasing the congestion probability.

Looking at the case Hengelo-Zuid, a capacity increase of 1-4% could reduce the congestion probability with approximately 10-20 percent points in the current situation. If congestion can be prevented, on-ramp delay could be reduced up to 6 minutes. On average, congestion duration can be reduced up to 10 minutes. Then, a rough estimation of reduction in on-ramp delay is 1.5 minutes.

The external validity of the model is relatively low. The effects of the continuous line are applicable for situations where the merge is an active bottleneck, with a two-lane highway, and approximately the same vehicle distributions (percentage freight traffic). Effects of the line are applicable for locations with the same or lower speed limits. In latter situations, a longer line is required to get the optimal lane distribution.

8. FINDINGS

This research is conducted by performing a literature study, a single case study at Hengelo-Zuid, and by simulating this case in a traffic simulation model. The findings which are done during the research are presented below. Conclusions are presented in the next chapter.

The literature study showed relations between traffic characteristics around a merge. On a macroscopic level these are capacity distributions, congestion, traffic flows and shock waves. At a microscopic scale, also headways, lane distributions, and lane changing play an important role. Headway distributions can be distinguished into a constant, intermediate and random headway state. Only few studies focused on influencing lane distributions for merging traffic. Ramp metering is usually used to control on-ramp flows and to prevent highway congestion. The effect of a continuous line on lane distributions is not studied before. Shorter versions of the line are implemented for safety reasons and refer to a slight speed increase.

Findings from the literature are studied in the study area at Hengelo-Zuid, which consist of a two-lane main carriageway. Both detection loop data and video observations at the merge are used. The detection loop data is only 50% of the time reliable, which restricted the possibilities during data analysis.

After a literature study and situation analysis, the capacity value is found to be the most suitable highway performance indicator. Capacity values in the study area were relatively easy to observe, and used modelling software is found suitable to evaluate effects on capacity. The observed capacity is relatively high but plausible (5016 veh/h for a two-lane highway). The observed capacity drop is 19%. Cooperative lane changing is observed, which influences the lane distribution significantly.

With the findings from the situation analysis, the highway access at Hengelo-Zuid is rebuilt in a traffic simulation model. Fosim is found to be the most suitable simulation software. During the calibration was aimed to simulate the microscopic traffic characteristics as good as possible, with the eventual aim to create a model that reliably can determine capacity values. Eventually, a model is created that is found to be suitable to show capacity increases.

With the calibrated model, the effect of the continuous line is evaluated for three different on-ramp flows: 1) a signalised on-ramp flow, representing an intermediate headway state; 2) a random on-ramp flow, representing a more random headway state; and 3) a metered on-ramp flow (with RWS-algorithm), representing a more constant on-ramp flow. All on-ramp flows are theoretically and practically relevant.

Capacities for random and metered on-ramp flows are on average higher than capacities for signalised on-ramp flows. Most effects of the line are therefore expected for signalised on-ramp flows. The capacity for signalised on-ramp flows is lower due to platoons of vehicles that merge simultaneously.

The effect of the continuous line must be related to the lane distribution. The relation between the optimal lane distribution and the length of the line could not be determined by the model. Test pilots could evaluate this.

9. CONCLUSION

The aim of this research is to evaluate to what extent the highway performance at a merge could be increased, by analysing the effect of a continuous line in the pre-merging area on the highway and give possible implications for a ramp meter. This chapter presents the conclusions.

The main conclusion is that a continuous line can increase the capacity of the highway slightly but significantly. For high signalised on-ramp flows and high metered on-ramp flows, the maximum length of the line is constrained by negative effects (backward shock waves) at lane one. For all other on-ramp flows, a long continuous line increases the capacity without creating negative effects at lane one. The amount of lane changes does not increase with a line: neither before the merge, nor after the merge.

Most effects of the line are expected for signalised on-ramp flows. The modelled increase in capacity is 1-4%. For a ramp meter, a continuous line enables a higher release rate without increasing the congestion probability.

The continuous line aims for an optimal lane distribution at the merge. If the optimal length of the line is not constrained by negative effects at lane one, the optimal lane distribution ratio is at least 71% (lane one) and 29% (lane two). Higher ratios (a higher share of vehicles at lane one) are not tested. If negative effects at lane one can occur, the share of vehicles at lane one should be lower.

The results apply for each merge which is an active bottleneck location with approximately the same traffic conditions and road geometry. If lower speed limits apply, a longer line is required to get the optimal lane distribution.

Looking at the case Hengelo-Zuid, a capacity increase of 1-4% could reduce the probability for congestion with approximately 10-20 percent points in the current situation. On-ramp delay could then be reduced up to 6 minutes. On average, the congestion duration could be reduced up to 10 minutes. A rough estimation of reduction in on-ramp delay is then 1.5 minutes.

Recommendations to USE

This research came into being after a product development of USE system engineering, namely dynamic road markings. In reference to that, this research evaluated the effect of a continuous line on the highway performance. From a traffic engineering point of view, a dynamic road marking has definitely potential for realising the continuous line. Variations in on-ramp demands and flow patterns make the optimal length of the continuous line variable, which could be regulated with a dynamic road marking.

10. DISCUSSION

This research is performed using a single case study in combination with a simulation study. This strategy has some limitations, which are discussed in this section.

The single case study implies that the effect of the continuous line is only evaluated for the situation at Hengelo-Zuid. The effect of the line applies for all other active bottlenecks with identical characteristics. Different on-ramp demands and patterns are evaluated, though locations with more lanes, different amount of freight traffic or other ramp meter strategies are not evaluated. A sensitivity analysis about effects of different input parameters is not performed.

The traffic simulation model is calibrated and validated for capacity calculations for Dutch highways. Effects could only be evaluated on a more macroscopic level. Effects on microscopic level are hard to evaluate. This is however the case for the most microsimulation models. Effects on safety, or automated driving are also not taken into account. Cooperative driving could for example influence traffic stability – and thus the congestion probability. Though, such adaptive systems could also aim for longer headways – and thus less capacity.

Safety issues are also not taken into account. Though, a continuous line increases the share of traffic at lane one. In The Netherlands, a minimum headway of two seconds is considered to be safe. Though, in peak hours this is not considered to be realistic (Rijksoverheid, 2013). But with a continuous line, the average headways drop further below these two seconds (see Appendix VI.). The reduction in lane changes could increase safety.

The effect of the continuous line is tested for different on-ramp demands and flow patterns. Results are different for each situation. In reality, several situations can apply. The on-ramp demand can be variable for example, and a ramp meter can be broken down which results in an other flow pattern at the same location. Then, a dynamic road marking could be a solution. Implementation of the line, cost-effectiveness and other (dis)advantages of implementing the line are not studied.

Another subject of discussion is that a continuous line is just an example of influencing lane distribution. Alternatives of influencing lane distribution could also be effective. After this research, no pronouncement can be made about the most effective way to influence lane distribution.

The traffic simulation model is calibrated for the case Hengelo-Zuid. The model can be used for further research, for example about further improving efficiency at a merge. Appendix VII. presents a foundation for further research about adjusting the on-ramp flow and the upstream flow to each other, such that the congestion probability is minimised.

11. REFERENCES

Appert-Rolland, C., & Du Boisberranger, J. (2011). Macroscopic relaxation after on-ramps in real data and in cellular automata simulations. *Transportation Research Part C, In Press*.

ARCADIS, & Bureau Onderweg. (2011). Evaluatie experimenten Dynamax 130 km/h.

Bie, J. (2011). Ramp Metering *Operations & Management 2*. Enschede: University of Twente.

Cassidy, M. J., & Rudjanakanoknad, J. (2005). Increasing the capacity of an isolated merge by metering its on-ramp. *Transportation Research Part B: Methodological, 39*(10), 896-913.

Chen, C., Jia, Z., & Varaiya, P. (2001). Causes and cures of highway congestion. *IEEE Control Systems Magazine, 21*(6), 26-32.

Daamen, W., Loot, M., & Hoogendoorn, S. P. (2010) Empirical analysis of merging behavior at freeway on-ramp. (pp. 108-118).

Daganzo, C.F. (1995). The cell transmission model, part II: network traffic. *Transportation Research Part B* (2), 79-93.

Daganzo, C. F. (2002). A behavioral theory of multi-lane traffic flow. Part II: Merges and the onset of congestion. *Transportation Research Part B: Methodological, 36*(2), 159-169.

Dijker, T., & Knoppers, P. (2006). Fosim 5.1 Gebruikershandleiding: TU Delft.

DVS. (2011). Capaciteitswaarden Infrastructuur Autosnelwegen (3 ed.).

Eurosense. (2008). Wegbeheerders. Retrieved from http://www.rijkswaterstaat.nl/geotool/wegbeheerders.aspx?cookieload=true

Geerarts, J. F. H. G., & Van Bergen, J. N. (2003). Project Voertuig Categorie Indeling: Adviesdienst Verkeer en Vervoer.

Gemeente Hengelo. (2013). Loggegevens VRI Diamantstraat - Haaksbergerstraat.

Google. (2013). Google Maps, from http://maps.google.com/

Goudappel Coffeng. (2013). Regio Twente model. In Goudappel Coffeng (Ed.). Hengelo: Gemeente Hengelo,.

Grontmij. (2002). Leidraad model- en evaluatiestudies benuttingsmaatregelen: Adviesdienst Verkeer en Vervoer.

Grontmij. (2009). Capaciteitsmetingen autosnelwegen - Samenvatting van de resultaten voor 75 locaties.

Hidas, P. (2005). Modelling vehicle interactions in microscopic simulation of merging and weaving. *Transportation Research Part C: Emerging Technologies, 13*(1), 37-62.

Kesting, A., Treiber, M., & Helbing, D. (2007) General lane-changing model MOBIL for car-following models. (pp. 86-94).

KNMI. (2013). Etmaalgegevens Twenthe. De Bilt: Royal Netherlands Meteorological Institute (KNMI),.

Knoop, V. L., Duret, A., Buisson, C., & Van Arem, B. (2010). *Lane distribution of traffic near merging zones influence of variable speed limits*.

Laval, J. A., & Leclercq, L. (2008). Microscopic modeling of the relaxation phenomenon using a macroscopic lane-changing model. *Transportation Research Part B: Methodological, 42*(6), 511-522.

Leclercq, L., Laval, J. A., & Chiabaut, N. (2011). Capacity drops at merges: An endogenous model. *Transportation Research Part B: Methodological, 45* `(9), 1302-1313.

Liu, R., & Hyman, G. (2012). Modelling motorway merge: The current practice in the UK and towards establishing general principles. *Transport Policy, 24*, 199-210.

Loot, M. (2009). Empirical analysis of merging behaviour. Delft: TU Delft.

May, A. D. (1990). *Traffic Flow Fundamentals*. New Jersey: Prentice-Hall.

Middelham, F., & Taale, H. (2006). *Ramp metering in The Netherlands: An overview.*

Middelham, F., Taale, H., & Wang, T. C. (2001). Syllabus FLASH/FLEXSYT-II-. Rotterdam: Adviesdienst Verkeer en Vervoer.

Ministerie van Infrastructuur en Milieu. (2011). *Mobiliteitsbalans 2011*.

Ministerie van Verkeer en Waterstaat. (2006). *Evaluatie proef samenvoegstrook A12*. Rijkswaterstaat.

Ministerie van Verkeer en Waterstaat. (2007). *Handleiding verkeerskundig beheer toeritdoseerinstallaties*. Rijkswaterstaat, Dienst Verkeer en Scheepvaart.

Ministerie van Verkeer en Waterstaat. (2008). *Eindboek FileProof*. Rotterdam.

Molenkamp, L. (2008). *FileProof: Fileaanpak op de korte termijn*. Ministerie van Verkeer en Waterstaat.

Newell, G.F. (1982). *Applications of queueing theory,* second ed. New York: Chapman & Hall.

Nokia & Microsoft Corporation. (2013). Bing Maps, from http://maps.bing.com/

Noordmans, M. (personal communication, 2012). [Meeting at BAM Infrasolutions bv, Culemborg].

Papageorgiou, M., Hadj-Salem, H., & Middelham, F. (1997) ALINEA local ramp metering: Summary of field results. (pp. 90-98).

Papageorgiou, M., & Kotsialos, A. (2002). Freeway Ramp Metering: An Overview. *IEEE Transactions on Intelligent Transportation Systems, 3*(4), 271-281.

Provincie Overijssel. (2006). Visuele kruispunttellingen N739 Haaksbergerstraat - Op- en afritten A35.

Rijksoverheid. (2013). Wat is de 2 seconden regel en hoe pas ik deze toe? Retrieved 23 May, 2013, from http://www.rijksoverheid.nl/onderwerpen/verkeersveiligheid/vraag-en-antwoord/wat-is-de-2-seconden-regel-en-hoe-pas-ik-deze-toe.html

Rijkswaterstaat. (2012). Visuele 24-uurstellingen A1 Deventer-Oost - Azelo.

Rijkswaterstaat. (2013). Monica data A35: Grontmij, Arcadis, Rijkswaterstaat.

Sarvi, M., & Kuwahara, M. (2008). Using ITS to improve the capacity of freeway merging sections by transferring freight vehicles. *IEEE Transactions on Intelligent Transportation Systems, 9*(4), 580-588.

Schultz van Haegen, M. H. (2012). *Eindbeeld verhoging maximumsnelheid*. Den Haag.

Taale, H., Slager, J., & Rosloot, J. (1996). The Assessment of Ramp Metering Based on Fuzzy Logic: RWS - AVV, Heidemij Advies.

Traag, W. (personal communication, 2012). [Meeting at Rijkswaterstaat Oost Nederland, Arnhem].

Van Berkum, E. C., Thomas, T., Telgen, M. G., & Buyck, S. C. P. (2010). *Statistic Tools for Data Analysis in Traffic Engineering*. Enschede: Universiteit Twente.

Van der Kuil, J.H.M. (personal communication, 2012). [Meeting at USE System Engineering, Haaksbergen].

Van Toorenburg, J. A. C. (1988). Toepasbaarheid van toeritdosering. *Verkeerskunde, 40 (1988) Nr. 6*, 284-289.

Van Velzen, B., & De Jong, F. (2012). [Conversation about traffic simulation models, Grontmij, Utrecht].

VID. (2012). Fileproblematiek Retrieved 30 October 2012, 2012, from http://www.vid.nl/fileproblematiek.html

Vlek, E. (personal communication, 2012). [Phone conversation with Eric Vlek, Vlek Verkeerslichten, Oosterhout.].

Wessel, P. (2003). Critical Values for the Two-sample Kolmogorov-Smirnov test (2-sided).

Wikipedia. (2013). Kolmogorof-Smirnov Test, from https://en.wikipedia.org/wiki/Kolmogorov%E2%80%93Smirnov_test

Wu, J., McDonald, M., & Chatterjee, K. (2007). A detailed evaluation of ramp metering impacts on driver behaviour. *Transportation Research Part F: Traffic Psychology and Behaviour, 10*(1), 61-75.

Appendix I. Formulas for statistical tests

Hypothesis for testing the mean:

$$H_0: \bar{x}_{Ref} = \bar{x}_{Model}\,;\ H_1: \bar{x}_{Ref} \neq \bar{x}_{Model}\ \text{; Accept } H_0 \text{ if: } T_{DF} < T_{0.05} \tag{27}$$

Hypothesis for testing the standard deviation:

$$H_0: S_{x,Ref} = S_{x,Model}\,;\ H_1: S_{x,Ref} \neq S_{x,Model}\ \text{; Accept } H_0 \text{ if: } F_{n-1}^{m-1} < F_{0.05}\,. \tag{28}$$

Hypothesis for testing the shape of the distribution:

$$H_0: F_{Ref} = F_{Model}\,;\ H_1: F_{Ref} \neq F_{Model}\ \text{; Accept } H_0 \text{ if: } D_n < D_{0.05}\,. \tag{29}$$

One sided T-test:

$$T_{n-1} = \frac{\bar{X} - a}{S_{\bar{X}}} \tag{30}$$

Two-sided T-test:

$$T_{DF} = \frac{\bar{x} - \bar{y}}{\sqrt{\frac{S_x^2}{n} + \frac{S_y^2}{m}}} \tag{31}$$

F-test

$$F_{n-1}^{m-1} = \frac{S_x^2}{S_y^2} \tag{32}$$

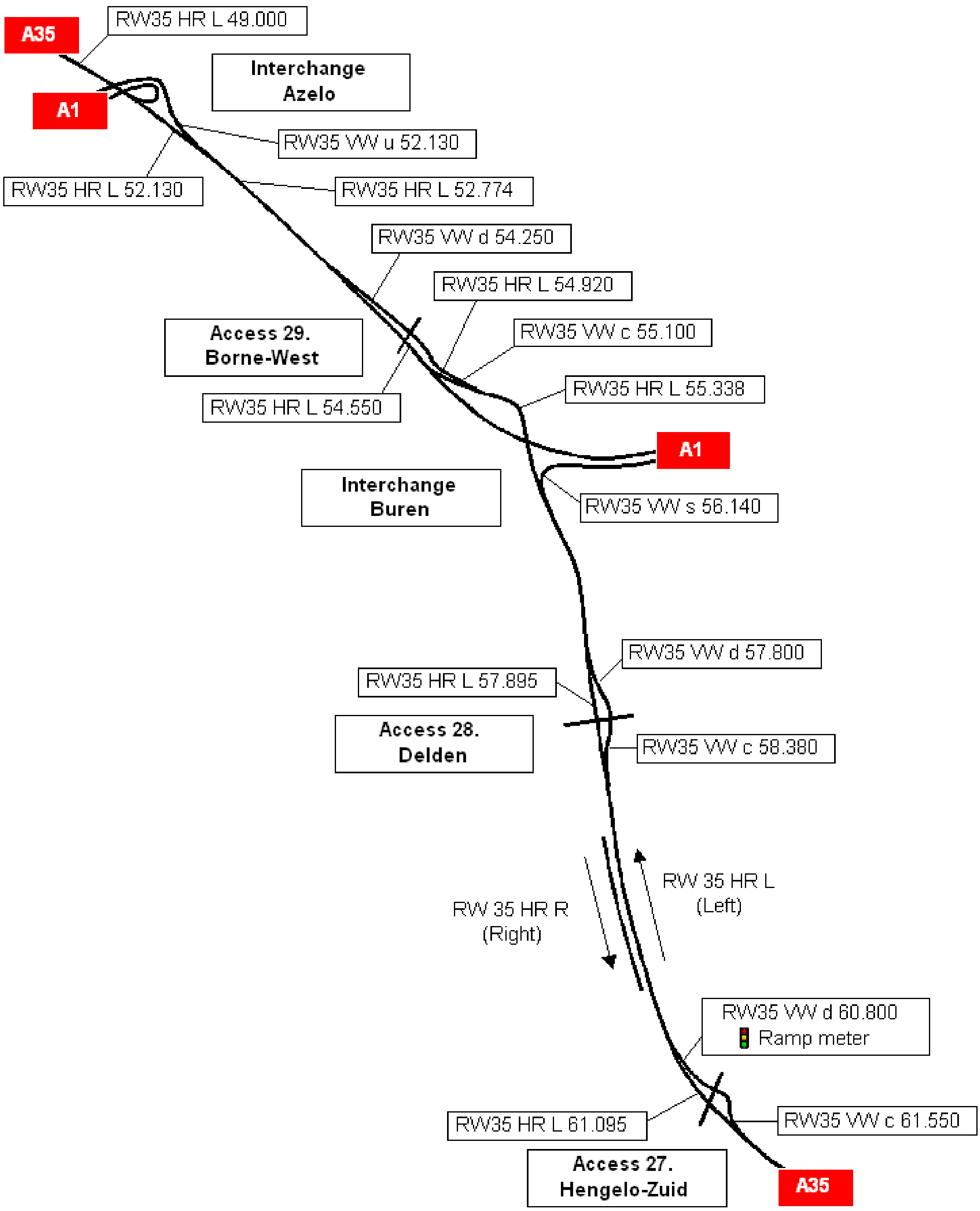
KS-test:

$$D_n = \sup \left| F_{X,n}(x) - F_{Y,m}(x) \right| \tag{33}$$

$$D_{0.05} = c_{0.05} \cdot \sqrt{\frac{n_X + n_Y}{n_X \cdot n_Y}} \tag{34}$$

(Sources: Van Berkum, Thomas, Telgen, & Buyck, 2010; Wessel, 2003; Wikipedia, 2013)

APPENDIX II. DETECTION LOOPS IN STUDY AREA



The continuous line continued...

Appendix III. Multi criteria analysis for simulation software

Model	*Dutch Highway*	*Continuous line*	*Microscopic LC / CF*	*Capacity*	*Ease*	*Costs*	*Assessment*
AIMSUN	+/-	-	+	+	+/-	€ 5950,-	
FLEXSYT-II	+/-	-	+	+		Free	
FlowSimulator	+	?	-	+		?	
FOSIM	+	+	+	+	+	Free	1
Integration	+	?	+	+		€ 1000,-	3
MIXIC	+	-	+	+/-		€ 2250,-	
Paramics 2000	+	-	+	+	-	Free	
TRANSYT	-	-	-	-		€ 50,-	
VISSIM	+/-	+	+	+	+/-	€ 4950,-	2

N.B. None of the models was able to simulate cooperative lane changing. The ease of the model is based on personal experience.

(Sources: Dijker & Knoppers, 2006; Grontmij, 2002; Middelham, Taale, & Wang, 2001; Van Velzen & De Jong, 2012)

Appendix IV. Fosim settings

IV.I Vehicle parameters in Fosim

Parameters		*Vehicle types*						
		Passenger cars					Freight traffic	
		1	6*	2	7*	3	4	5
Desired speed								
At 120 km/h	km/h	**135**	**128**	**120**	**110**	**102**	**98**	**86**
At 70 km/h	km/h	95	90	85	80	75	75	75
Max. acceleration jump	m/s^3	1	0.8	0.6	0.6	0.6	0.5	0.4
Car following factor z2	s	0.56	0.6	0.72	1	1.28	2.08	2.23
Max. acceleration	m/s^2	4	2.8	2.4	2.4	2.4	1	0.4
Max. following deceleration	m/s^2	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Max. lane change deceleration	m/s^2	-3	-2.6	-2.4	-2.4	-2.4	-2	-1.6
Max. deceleration	m/s^2	-7	-7	-7	-7	-7	-6	-6
Vehicle length	m	4.5	4.2	4	4	4	8	14
Car following factor z1	m	3	3	3	3	3	3	3
Car following factor z3	s^2/m	0.005	0.005	0.005	0.005	0.005	0.005	0.005
Average specific power	kW/ton	80	65	50	43	35	12	9
Standard deviation specific power	kW/ton	0	0	0	0	0	0	0
Air resistance cooefficient	km^{-1}	0.6	0.6	0.5	0.5	0.4	0.2	0.1
Maximum deceleration at traffic lights	m/s^2	-3.5	-3.5	-3.5	-3.2	-3	-3	-2.5
General parameters								
Lane change time	s	3						
Reaction time for acceleration	s	0.30						
Reaction time for deceleration	s	0.20						

* Vehicle types 6 and 7 are added, values are estimated.
The **bold** values are calibrated.

The continuous line continued...

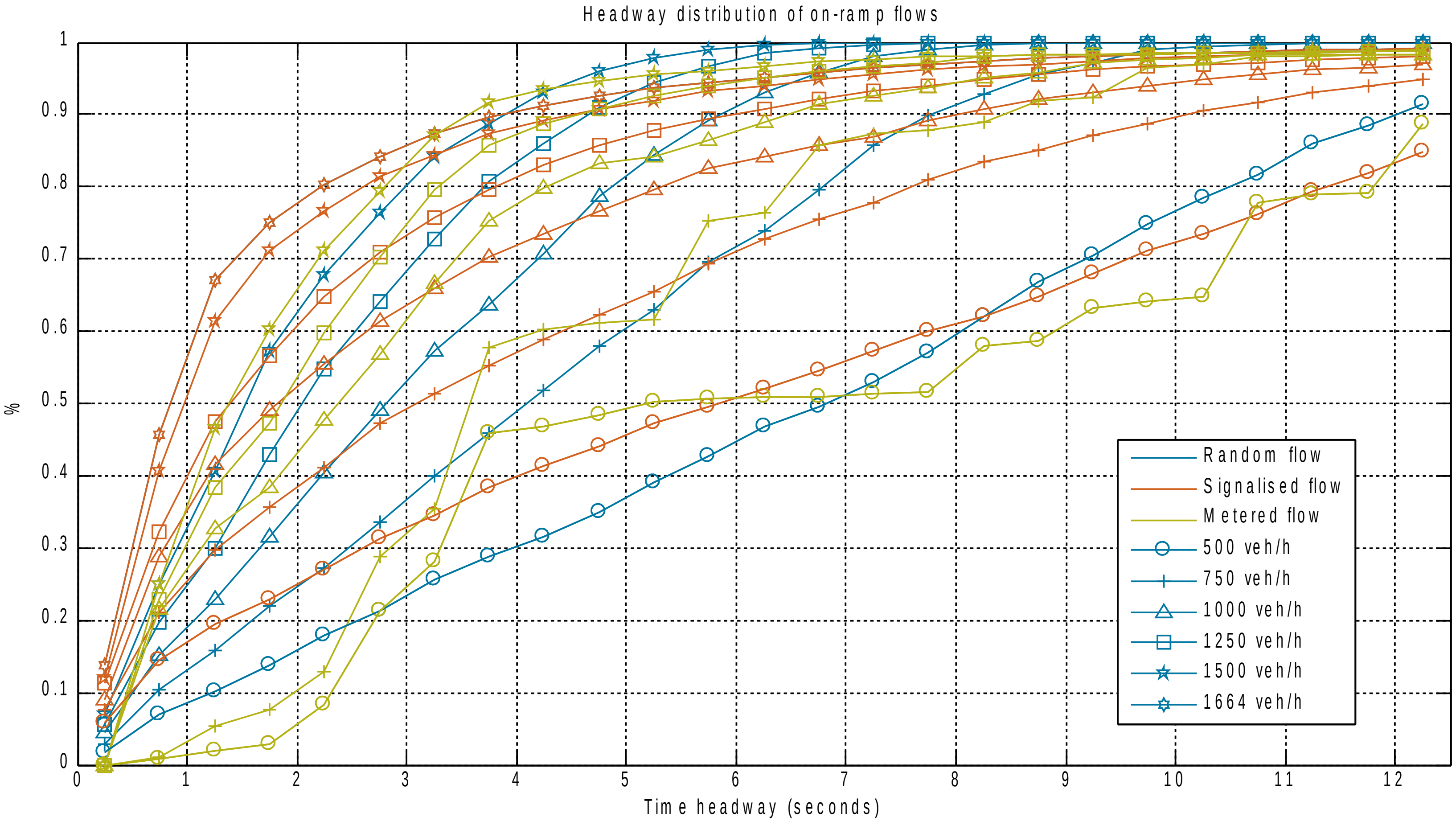
IV.II Fosim settings for calibration and validation

<table>
<tr><th colspan="2">Indicator</th><th>Headway distributions</th><th>Fundamental q/v diagram</th><th>Intensity and variances</th><th>Capacity distribution</th></tr>
<tr><td colspan="2">Calibration parameters</td><td colspan="4">- Vehicle parameters (desired speeds per vehicle category)
- Length of first section in the model
- Traffic light settings
- Vehicle distributions per vehicle type
- Intensity raise factor</td></tr>
<tr><td rowspan="3">Input</td><td>Intensity</td><td>Measured minute intensities (15 min, video)</td><td>Averages from u_{qv}; Random generator * St.dev * F_{Raise}</td><td>Average from u_{Line}; Random generator * St.dev * F_{Raise}</td><td>Increasing with 1000 veh/h</td></tr>
<tr><td>%HGV</td><td>As measured with video</td><td>As measured with loop detector</td><td>As measured with loop detector</td><td>As measured with loop detector</td></tr>
<tr><td>On-ramp</td><td>10%</td><td>10%</td><td>10%</td><td>10%</td></tr>
<tr><td rowspan="3">Simulation</td><td>Runs</td><td>50</td><td>50</td><td>50</td><td>100</td></tr>
<tr><td>Duration</td><td>20 minutes (1200 seconds)</td><td>45 minutes (2700 seconds)</td><td>+1 hour (3800 seconds)</td><td>Infinite</td></tr>
<tr><td>Stop at</td><td><70 km/h at q_{up}</td><td>-</td><td><70 km/h at q_{up}</td><td>< 40 km/h at q_{down}</td></tr>
<tr><td colspan="2">Output</td><td>Micro detector</td><td>Micro detector</td><td>Micro detector</td><td>Capacity distribution</td></tr>
</table>

IV.III Fosim settings for modelling results

Indicator		*Capacity distribution*	*Max. upstream intensity*	*On-ramp headway distributions*	*Lane distributions and lane change*
Calibration parameters		- Vehicle parameters (desired speeds per vehicle category) - Length of first section in the model - Traffic light settings - Vehicle distributions per vehicle type - Intensity raise factor			
Input	*Intensity*	Increasing with 1000 veh/h	Increasing with 1000 veh/h	- Random - Signalised - Ramp meter	Average from u_{Line}; Random generator * St.dev * F_{Raise}
	%HGV	As measured with loop detector	As measured with loop detector	As measured with loop detector	As measured with loop detector
	On-ramp	10%	10%	10%	10%
Simulation	*Runs*	100	100	1	50
	Duration	Infinite	Infinite	10.000 seconds	1800 seconds
	Stop at	< 40 km/h at q_{down}	< 70 km/h at q_{up}	<70 km/h at q_{up}	< 40 km/h at q_{down}
Output		Capacity distribution	Capacity distribution	Micro detector	Micro detector

APPENDIX V. ON-RAMP HEADWAY DISTRIBUTIONS



APPENDIX VI. MODELLED HEADWAYS AND LANE DISTRIBUTIONS

Highway flow (q_{up})	*Line type*	*Lane 1*			*Lane 2*			*Lane 1 / Lane 2 (%)*
		Mean	Std	q_{lane1}	Mean	Std	q_{lane2}	
2500	Reference	2.06	2.45	1623	3.65	2.35	930	64% / 36%
	Line 1	1.98	2.33	1689	3.94	2.52	864	66% / 34%
	Line 2	1.92	2.28	1733	4.15	2.82	820	68% / 32%
	Line 3	1.88	2.25	1770	4.34	3.17	784	69% / 31%
	Line 4	1.84	2.23	1805	4.54	3.56	749	71% / 29%
2750	Reference	1.89	2.23	1767	3.46	2.24	979	64% / 36%
	Line 1	1.82	2.12	1832	3.71	2.43	914	67% / 33%
	Line 2	1.77	2.09	1879	3.92	2.73	866	68% / 32%
	Line 3	1.73	2.05	1915	4.09	3.05	831	70% /30%
	Line 4	1.70	2.01	1948	4.26	3.45	798	71% / 29%
3000	Reference	1.60	2.07	1923	3.14	2.24	1029	65% / 35%
	Line 1	1.52	1.97	1994	3.39	2.44	958	68% / 32%
	Line 2	1.48	1.93	2043	3.59	2.73	909	69% / 31%
	Line 3	1.45	1.90	2075	3.73	3.02	876	70% / 30%
	Line 4	1.56	1.85	2109	4.02	3.41	843	71% / 39%
3250	Reference	1.45	1.85	2084	2.89	2.13	1109	65% / 35%
	Line 1	1.39	1.74	2153	3.10	2.34	1040	67% / 33%
	Line 2	1.35	1.70	2201	3.26	2.63	992	69% / 31%
	Line 3	1.32	1.68	2237	3.40	2.95	955	70% / 30%
	Line 4	1.29	1.64	2276	3.55	3.30	917	71% / 29%
3500	Reference	1.31	1.65	2257	2.69	2.11	1179	66% / 34%
	Line 1	1.26	1.56	2325	2.87	2.32	1111	68% / 32%
	Line 2	1.23	1.51	2372	3.01	2.60	1064	69% / 31%
	Line 3	1.20	1.47	2410	3.13	2.89	1026	70% / 30%
	Line 4	1.18	1.43	2447	3.26	3.25	988	71% / 29%

APPENDIX VII. FOUNDATIONS FOR FURTHER RESEARCH

During the research the on-ramp at Hengelo-Zuid is modelled. On-ramp and highway flows in the model are representative for the observations. The model could be used for further investigations to improve the traffic performance at the merge. This chapter shows implications for further research at the case A35 on-ramp at Hengelo-Zuid.

VII.I Merge of on-ramp and highway flows

During observations, high variances in on-ramp and upstream flows are observed. During the observations an impression was obtained that on-ramp and upstream flows merge inefficiently, what results in a high congestion probability. This inefficiency is shown in Figure VII.1.

The figure shows the variances in the 10-second flows from the on-ramp and highway upstream, measured at the location of the video measurements (250 m before the merge). The interval of 10 seconds is chosen because a sufficient amount of vehicles can be grouped within an interval that is as small as possible.

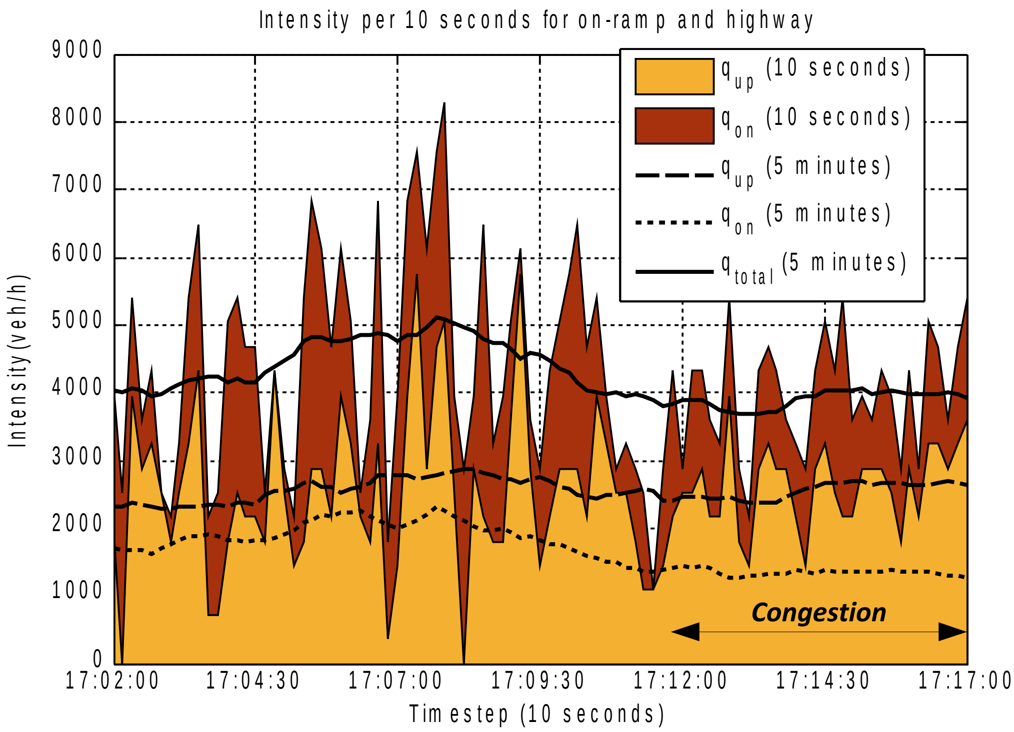


Figure VII.1: Intensities per 10 seconds interval, 250 m before merge. Congestion is observed at 17:12 (at 10 January 2013)

The figure shows that both on-ramp and highway flow have large peaks. These flows are uncorrelated to each other and the peaks can strengthen or weaken each other. Congestion is observed at 17:12 (at this time, congestion was visible within the range of the camera). It is thus possible that congestion started earlier, for example due to the peak at 17:08 or 17:10.

Hereby following hypothesis is formulated:

> *The merge of platoons in upstream and/or on-ramp flows increases the congestion probability.*

This hypothesis is strengthened by the simulation results of Chapter 6. The capacity values are for the signalised on-ramp flows (with platoons) lower than for random on-ramp flows. If this hypothesis is found to be true, a better match of upstream and on-ramp traffic flows could reduce the congestion probability. Ideally, the on-ramp flow is the inverse of the highway flow, such that the peaks in traffic flows neutralise each other.

A ramp meter matches the on-ramp and highway flows better, since the on-ramp flow is metered. Though, the release rate of the ramp meter anticipates on the average 5-minute on-ramp flow, as described in Chapter 2. This is visualised in Figure VII.2.

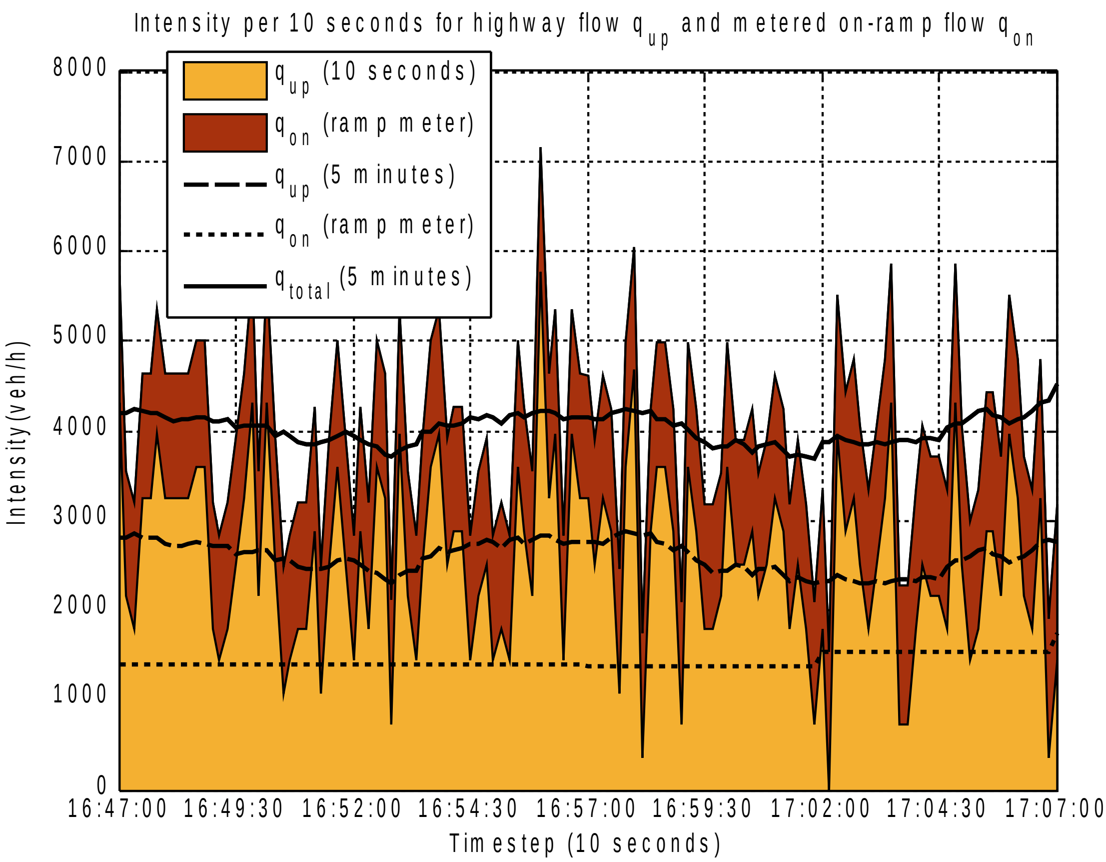


Figure VII.2: Merge of highway upstream flow (10 January 2013) and metered on-ramp flow (estimated).

Due to the variance in highway flow, the ramp meter is not able to neutralise the peaks in highway upstream flow. A ramp meter aims namely for a downstream flow (q_{down}) equal to the discharge capacity, and refers thus to an insufficient use of the highway. Also here, the hypothesis which is formulated above applies.

VII.II Predicting flows

If the hypothesis stated in Section VII.I is true, the merging process could further be optimised by letting a ramp meter anticipate on the traffic flow into more detail. Then, higher release rates could be achieved without increasing the risk for congestion. In order to do so, the traffic flow at merge must be known in advance. Since the traffic simulation model is able to represent the observed traffic flows and variances, the model could be used to analyse to what extent traffic flows could be predicted.

Microscopic results from the model are used to test whether traffic flows at the merge could be predicted. Detection loop data from a location upstream from the merge (1) and merge (2) are measured in the model (see Figure VII.3).

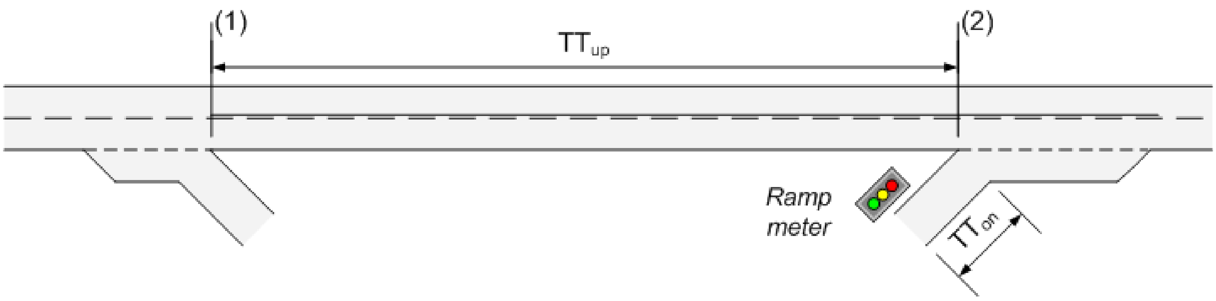


Figure VII.3: Measurement location for predicting (1) and observing (2).

For each vehicle at (1), the arrival time at (2) is estimated by dividing the speed measured at (1) by the distance between (2) and (1). The traffic flow can be predicted as far in advance as the minimum travel time between (1) and (2), called TT_{up}. For a speed of 130 km/h, and distance of 1175 m (between on- and off-ramp), the prediction horizon TT_{up} is 32 seconds.

The minimum predicting time is equal to the travel time of the merging vehicle from the ramp meter to the merge (250 m), called TT_{on}. In the model, this value varies between 16 and 23 seconds for passenger cars, and between 26 and 35 seconds for small and large trucks.

If predictions are made real time, a ramp meter could use actual predictions to determine how many vehicles can be released. The accuracy of predicting flows is shown in Figure VII.4 for a time horizon between 20 and 32 seconds.

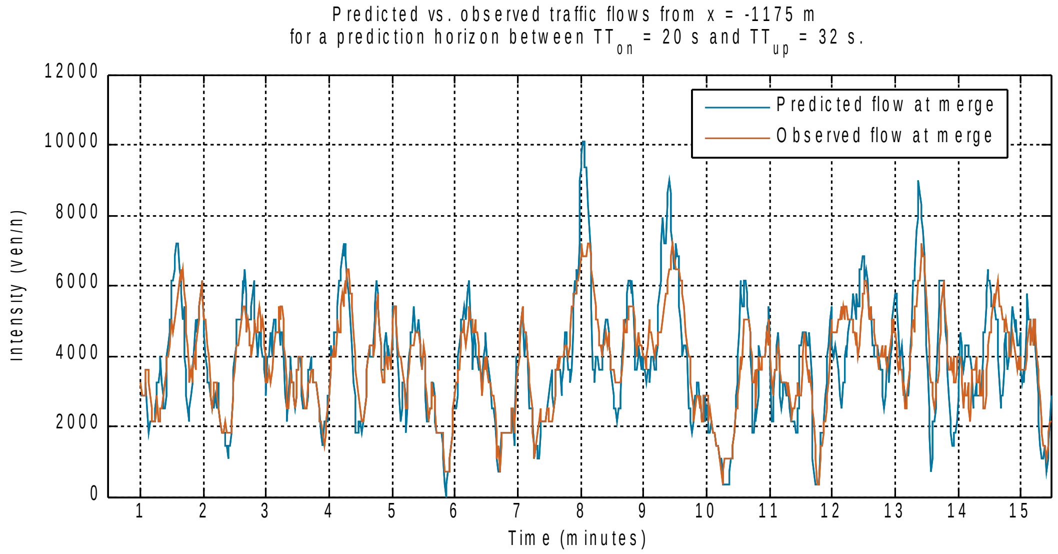


Figure VII.4: Predicting flows within a time horizon over 20 – 32 seconds.

The effectiveness of a ramp meter that anticipates on the traffic flow depends on the variance in the on-ramp travel time TT_{on}. A separate on-ramp lane for trucks could for example reduce the variance in this.

Further research can therefore evaluate the hypothesis:

An adaptive ramp meter can improve the on-ramp flow by not increasing the congestion probability.